GUIDELINES ON THE COST EFFECTIVENESS OF RURAL WATER SUPPLY AND SANITATION PROJECTS

CSIR • DWT

WRC Report No. 231/1/93

Water Research Commission

REPORT TO THE WATER RESEARCH COMMISSION

COMPRISING OF TWO PARTS

PART I:

GUIDELINES ON THE COST EFFECTIVENESS OF RURAL WATER SUPPLY AND SANITATION PROJECTS

- Executive Summary
- Cost-effectiveness and choice of technology

PART II:

GUIDELINES ON THE TECHNOLOGY FOR AND MANAGEMENT OF RURAL WATER SUPPLY AND SANITATION PROJECTS

- Technological aspects of rural projects

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ISBN 1 874858 47 0
WRC NO. 231/1/93

PRETORIA
DECEMBER 1991

PART I: GUIDELINES ON THE COST EFFECTIVENESS OF RURAL WATER SUPPLY AND SANITATION PROJECTS

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PART II: GUIDELINES ON THE TECHNOLOGY FOR AND MANAGEMENT OF RURAL WATER SUPPLY AND SANITATION PROJECTS

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TECHNOLOGICAL ASPECTS OF RURAL PROJECTS

ACKNOWLEDGEMENTS

These guideline documents emanate from a project funded by the Water Research Commission and undertaken by the CSIR through its Division of Water Technology. Inputs to the documents were also obtained from parties outside of this Division.

The project team wishes to thank the following people who were associated with the steering committee responsible for the project, and in particular for their valuable inputs and guidance leading to the final documents:

Mr H C Chapman	Water Research Commission
Mr D S van der Merwe	Water Research Commission
Mr D Huyser	Water Research Commission
Mr P Alcock	University of Natal, Pietermaritzburg
Mr M Muller	Development Bank of Southern Africa
Mr G A Norris	Division of Building Technology, CSIR
Mr I A Pearson	Division of Water Technology, CSIR
Mrs S B Ramahotswa (Mogane)	Division of Water Technology, CSIR
Mr J Rivett-Carnac	Appropriate Technology Information
Mr F E Solsona	Division of Water Technology, CSIR
Mr L Teubes	Division of Water Technology, CSIR

The Appropriate Technology Group would also like to give special thanks to the following people for their valuable contributions to the report documents:

Mr J Rivett-Carnac	Appropriate Technology Information
Mr D Reyneke	Ove Arup Incorporated
Mr C D Swartz	Division of Water Technology, CSIR

The willingness of the engineers, planners and developers operating in the Independent and National States and the rural areas of South Africa who agreed to an interview is gratefully acknowledged.

Lastly, this project would not have been initiated without the dedication and hard work of Mr P G Williams, formally of the National Institute for Water Research, CSIR (now living in Australia).

EXECUTIVE SUMMARY

1. INTRODUCTION

On 30 and 31 March 1987, the Water Research Commission arranged a workshop to co-ordinate the efforts of people and organisations involved in rural water development. This workshop was held at Espada Ranch outside Pretoria. The workshop highlighted the need of organisations and people working in the rural areas for guidelines on how to ensure the cost-effectiveness and appropriateness of their work.

Consequently the CSIR proposed the formulation of a set of guidelines to the Water Research commission, which would have the aims of establishing recommended procedures from lessons learnt in the past, and establishing guidelines for future projects on rural water supply and sanitation.

The project proposed was of two years duration (1988, 1989). Unfortunately the project leaders who had formulated the project left the CSIR before the project commenced. This together with the difficulty of employing new staff, resulted in the project being delayed for a period.

It is hoped that the final product as presented in this report does justice to the initial aims and vision of the project as originally formulated. The final product consists of two reports as follows:

"Guidelines on the cost-effectiveness of rural water supply and sanitation projects", and

"Guidelines on the technology for and management of rural water supply and sanitation projects"

2. AIMS OF THE PROJECT

The aim of this project was to develop guidelines to assist development organisations (government and non-government) and rural communities themselves(e.g. tribal authorities and development committees) to achieve a higher rate of success in rural development programmes, with emphasis on the provision of basic water and sanitation services.

The objectives in the development of the guidelines were as follows:

- assess methods to optimize the cost-effectiveness of rural development programmes and to evaluate projects;

- develop practical guidelines on cost-effective rural water supply and sanitation projects;

- develop practical guidelines to assess the cost-effectiveness of projects and to evaluate their success; and

- to institute a system for regular review and improvement of the guidelines.

3. SUMMARY

The International Water Supply and Sanitation Decade declared by the United Nations for the 1980's, resulted in an increase in the emphases placed on rural water supply and sanitation in the developing world, particularly third world countries. Experience in many developing countries world wide has been accumulated during this period and a number of important lessons with respect to successful technologies and implementation processes have been learnt.

Lessons from development projects in the water supply and sanitation fields have emphasized the need for two main components to enhance the success of these projects. These are:

1. the vital necessity of community involvement in all phases of the project, including the planning and decision making processes; and

2. the importance of the use of appropriate technology solutions for the provision of water and the disposal of human wastes to ensure the long term sustainability of such systems.

An important finding, both locally and in other developing countries, was that the community does not always choose the lowest-cost option, but rather an affordable option which they feel will provide them with a service they are satisfied with and are capable of maintaining. Thus when the cost-effectiveness of schemes is evaluated, such schemes which satisfy the aspirations of the community, yet are affordable but not necessarily the lowest cost, are the schemes which will be the most cost-effective.

The first guideline document on the cost-effectiveness of rural water supply and sanitation projects focuses primarily on the methodologies for the selection of technologies for rural water supply and sanitation projects. The important factors of sustainability, cost recovery, and community participation are stressed. The related costs and benefits of various water supply systems are dealt with, and a general service level table is provided. This table indicates the costs and benefits of service levels from traditional sources to house connections, through the steps of improved traditional sources, hand pumps, standpipes, yardtaps and finally house connections. Estimates of capital and operation costs are given for the various components of water supply schemes. Finally the steps recommended in the selection of suitable technology for a particular application are as follows:

- i discussion with the community on their needs and aspirations;
- ii feasibility study (technical and social);
- iii assessment of support finances available;
- iv consultation with community to select preferred option;
- v detailed design;
- vi training of personnel;
- vii setting up of administrative and technical support functions;
- viii implementation of system with community support at various levels;
- ix commissioning of system; and
- x post monitoring and evaluation.

The second guideline document on the technologies and management aspects of rural water supply and sanitation projects has focused on the various appropriate technology solutions for the provision of water supplies and sanitation to rural areas. The document has focused on technologies which have been successfully applied in Southern Africa, and in some cases in other parts of the world but which would be applicable for use in Southern Africa. The main topics covered in this document are:

- general survey of the existing situation in the rural areas of South Africa
- water sources
- water quality
- water treatment
- pumps and pumping
- storage and distribution
- sanitation options
- management aspects

Interviews with various officials responsible for providing water and sanitation in rural areas were conducted to assess which technologies are being used in South Africa, and what approach is adopted for the supply of these services.

Some of the main findings coming from these interviews were as follows:

a. Poor performance of conventional water treatment plants and water supply systems in rural areas can be attributed to maintenance problems. The maintenance problems are related to over-sophistication and the problem of centralisation (consultants and contractors are located in the distant urban centres). Local community members have not in general been trained to maintain the supply systems.

b. In rural areas there is minimum cost-recovery, even of the operating costs. This leads to the situation where a large portion of the budget of relevant government departments is used to operate and maintain existing schemes, and communities perceive the system as being government and therefore take no responsibility for it.

c. Linked to (b), is the situation that user communities have not been consulted or included in the planning of the schemes designed to supply them with water. Small, often independent organisations have been able to do this more effectively, but have not generally been able to provide the longer term back-up support to the community.

d. With respect to sanitation, rural dwellers use pit latrines or the bush in the majority of situations. There has been a small but growing swing to the construction of VIP latrines. The problem of fouled latrines which become a health hazard needs to be addressed through appropriate eduction.

Most organisations and authorities are now aware of the philosophies of community involvement in rural projects. However, the problems of maintenance, appropriate technologies and cost recovery still require much attention. These aspects are dealt with to some extent in the guideline document.

4. CONCLUSIONS

Conclusions emanating from this project fall into two main categories, i.e. those relating to the technology used for rural water supplies and sanitation, and secondly those relating to the institutional aspects of rural projects.

4.1 Technologies used for rural water supply and sanitation projects:

In terms of water supply and sanitation technologies, the world has progressed a long way in the past decade in developing appropriate technologies for use in rural areas. In Southern Africa much progress has also been made, with certain pioneering work having been accomplished by dedicated people, often on an individual or small team basis.

Presently some of the more appropriate water supply systems being employed in rural areas are as follows:

- Spring protection, storage and limited distribution, where favourable springs exist. These systems offer the benefits of good quality water which can substantially reduce the water collection time for rural dwellers. The associated costs are also relatively low, and maintenance requirements are rudimentary.

- Handpump fitted onto a well or borehole. As with protected springs, in most cases a good quality of water is available. Associated costs are low to medium, and maintenance requirements can be low with village level technician training.

- Wind, solar, diesel or electric pumps fitted to boreholes are significantly more costly than handpumps, but may prove to be a more cost effective solution in many circumstances. Trained caretakers and operators would be required. The groundwater quality will generally be high, although additional treatment may be required in cases of high fluoride, nitrate, or salinity.

- Surface water treatment through slow sand filtration, followed by storage and limited distribution. Such systems are appropriate where surface water is the only or preferred source. A pumping stage may be necessary. Final water quality is usually acceptable and maintenance requirements are relatively low, although trained personnel would be required on a semi-continuous basis.

Promising sanitation technologies for use in rural areas are as follows:

- VIP (ventilated improved pit) and SanPlat latrines. These latrines can be erected at a low cost by individual households. They provide virtually odour and fly free on-site disposal of human wastes. Under most circumstances the effect on the underground water reservoir is minimal and limited to a longer term nitrate build-up. Maintenance is in terms of general hygiene and cleanliness.

- Low flush pedestal connected to a digester/septic tank and on-site soakaway. These latrines may be preferred by certain households, as well as for use at rural institutions (schools, clinics, etc.). A number of systems are available, but these are all more costly and require a higher level of maintenance than VIP latrines.

- In more dense rural settlements, low or medium flush pedestals connected to digesters and then to some sewerage system. The transported effluent may be treated in ponds, or in wetland systems. Pond and wetland systems offer the possibility of low maintenance and the potential for agricultural or aquacultural exploitation.

Despite the progress made, these methods are not being widely utilised and successfully employed in South Africa. In certain areas where dedicated support is available, some of these methods have been adopted. However, in other rural areas, regional or large schemes which are extremely costly and require skilled technical inputs on an ongoing basis, are still being installed. Some government departments involved with the supply of water and sanitation services are now moving more and more to the use of these appropriate technologies. Unfortunately certain of these ventures have failed, not necessarily because of the use of inappropriate technology, but as a result of shortcomings in the institutional arrangements.

The results of a survey of the current situation points to the need to evaluate more of these technologies in the field under supportive conditions, to modify and change them to suit the local conditions where necessary, and to assess and then make known their capabilities more widely.

4.2 Institutional aspects of rural projects:

As some of the main problems associated with rural water supply and sanitation projects have been associated with maintenance, cost recovery, and community participation, it can be concluded that institutional aspects are a key to ensuring the success of these projects in the future. Of importance here are the following:

- need for community involvement in all aspects of the projects, including decisions on financing and cost recovery;

- need for improved education on sanitation and domestic hygiene;

- need for adequate training of local personnel in the operation and maintenance of schemes;

- need for support and training of local community level management structures (e.g. water committees);

- need for district and regional structures to provide ongoing support to these local management committees;

- need for appropriate technologies which can be operated and maintained at a local level.

In order to achieve this, more projects need to be implemented as pilot projects where all these aspects receive attention. In particular the full co-operation and involvement of the relevant government departments, local institutions, community members, and other interested organisations, should be assured.

5. CONTRACT OBJECTIVES AND PROJECT CONTRIBUTION

While it could not be stated that a ranking of rural water supply and sanitation technologies in terms of cost effectiveness has been achieved, this was not an actual objective. Instead, what has been presented is some recommended processes for determining the cost-effectiveness of rural water supply and sanitation projects. In addition, some necessary steps to ensure the long term sustainability of such projects has been presented. Promising technologies which fit the criteria of appropriateness have been described, and their relative costs determined within reason.

It is believed that this report offers the reader a useful though not exhaustive handbook on the processes and technologies for rural water supply and sanitation projects. It summarises the technologies most commonly being used in small-scale rural water supply schemes, and discusses the steps which should be taken into consideration when undertaking such projects.

It is recommended that this document be distributed as widely as possible, and with sufficient feedback an updated version be produced at a later stage, or preferably, in sections as suggested in the recommendations below.

Finally, it is believed that the present document can make a contribution and help the people involved in development to be able to assess projects from a wider perspective, and to plan new projects with deeper insight.

6. RECOMMENDATIONS

The water supply and sanitation decade has ended, but Southern Africa, and indeed the rest of the world, will need to focus much attention in the present and forthcoming years at developing and implementing strategies which are appropriate for rural development. Since water supply and sanitation are two of the basic needs in this respect, emphasis will continue to be centred on the provision of these services in future development thrusts.

Aspects which require particular attention at this time with respect to the provision of water supply and sanitation services in rural areas, include the following:

* There is a need for more attention to be paid to the education and training aspects of these schemes. Regional centres, or preferably regional training teams who can move from district to district, need to be established to provide the training and follow-up support needed. Education needs to focus on sanitation and domestic health and hygiene, as well as creating an awareness of ways of achieving development successfully. Training needs include the following:

- committee functions, management and the building of administrative capacity;

- maintenance of water supply infrastructure;

- construction of simple water supply features (e.g. spring protection);

- construction of VIP latrines;

- sanitary surveillance.

* Appropriate technologies which have been successful in other rural areas need to be tested and evaluated, and where necessary modified for local conditions. The following technologies are among those which should be followed up:

  - rainwater harvesting and storage;

  - small dams (especially sand dams) for semi-arid areas;

  - disinfection methods for small water supplies;

  - desalination for remote areas;

  - water dispensers for distribution terminals;

  - water storage tanks (ferro-cement and other low cost construction techniques); and

  - village level operation and maintenance (VLOM) handpumps.

* The privatization of many of the functions related to water supply in rural areas, should be promoted. In particular the support framework for entrepreneurs to participate in the construction and implementation of schemes, as well as to set up semi-commercial activities in:

  - manufacturing components for or the complete construction of VIP latrines;

  - selling water and maintaining the system;

  - operating and monitoring the system.

* With respect to the updating of the guidelines, it is suggested that this is not done as a whole. Rather specific aspects should be allocated to the person or organisation who is both closely associated with the field, and is in a position to update the relevant section objectively. On a contractual basis such persons or organisations should be requested to produce an update once in 2 years. There are at least 8 subsections in the report which could be treated thus:

- situation in rural South Africa
- water sources
- water quality
- water treatment
- pumps and pumping
- storage and distribution
- sanitation systems
- management aspects

PART I:

GUIDELINES ON THE COST EFFECTIVENESS OF RURAL WATER SUPPLY AND SANITATION PROJECTS

1. INTRODUCTION

The decision by the international water supply and sanitation community to embark on an ambitious programme of providing adequate water supply and sanitation facilities to all people, particularly those in the developing countries, was a sound and commendable decision. This has resulted in a tremendous amount of resources and effort being invested in the programme world wide, and a high level of sharing of experiences during the past decade. One of the greatest challenges has been to find ways of undertaking projects in the rural and peri-urban areas which are both cost-effective and result in an improved quality of life in the long term.

The experience gained during the past 10 years in particular has pointed to the need of three key elements to ensure the cost effectiveness of the water supply and sanitation projects embarked upon. These are:

- involvement of the community in all aspects of the project;
- the use of appropriate technology; and
- the need for institution building and training activities in conjunction with the project.

The process involved in the implementation of water supply and sanitation projects therefore requires the inputs of a multi-disciplined team, or at least of individuals who understand all the elements of technology, community work, and training.

This section of the report deals with the selection of technology and approach for community based projects. The second part of the report deals in more detail with the different technologies and the management requirements.

2. CHOICE OF COMMUNITY WATER SUPPLY TECHNOLOGY

When deciding on a community water supply or sanitation option which will be the most cost-effective in a given environment, a number of factors must be considered. Technology choice cannot be made on the basis of costs alone. Technical, economic, institutional, nutritional, public health and environmental impact factors, among others, may also need to be considered. However, measuring non-financial benefits and costs are difficult, but an estimate of benefits and costs will normally need to be made in order to compare technologies and select the most appropriate. Water supply options in this context refer not only to the sources of water, but also to the degree to which a particular source can be developed, the quality improved if necessary, and then distributed to points closer to peoples homes. Sanitation options refer to individual or community ablution facilities, as well as individual or community wastewater treatment and disposal facilities.

2.1 Important overriding factors

Before discussing ways to realistically compare technology options, there are certain overriding factors which must be borne in mind when considering alternatives.

i. The first of these factors is the importance of the sustainability of a new water supply scheme. Assessments of why past projects have often failed to live up to expectations have pointed to the important factor that the technology selected was usually not sustainable with the resources available to the particular community. Thus an important principle for community water supply systems should be that:

> **"The technology chosen should give the community the highest level of service that it is willing to pay for, and which it has the institutional capacity and resources to sustain."**

Note that this does not imply that a community must have all the skills required to maintain and/or repair a water supply or sanitation system, although this may be desirable, but rather that they have the institutional capacity and resources to contract the necessary maintenance service when required.

In general, technologies offering higher service levels place correspondingly higher resource demands on the benefiting community. This refers to both the initial capital outlay as well as ongoing operation and maintenance costs. These latter costs include both technical skills and materials (fuel, chemicals, spares, etc.). This is an important consideration in the South African context where sponsorship or subsidies may be obtained for the initial capital investment, but not for the ongoing operational and maintenance requirements. In the light of this the following statement holds true:

> **"Success or failure depends primarily on one factor : can the new water or sanitation system be sustained ?"**

ii. A second important overriding factor is that community water supply and sanitation improvement projects for one community should not be seen in isolation from the communities surrounding them. This means that the initial capital and the ongoing recurrent costs of a project be considered within a framework of limited resources. Or, in other words,

> **"Only if a reasonable proportion of the total costs of a project are recovered from the benefiting community will it be possible to generate the funds needed to replicate projects on a scale which would enable all rural populations to obtain at least a minimum level of improved supply."**

This does not mean that governments or donor agencies should stop supporting investments in community water supply and sanitation projects, but rather that only through the combination of such investments and the mobilization of local cash reserves is there hope for widespread rapid and self-sustaining improvements in rural water supply and sanitation in South Africa as a whole. In the countries where a substantial amount of financing is made available for rural water supply and sanitation upgrading, cost recovery may not appear to be as relevant. However, cost recovery enhances community ownership and hence the long term sustainability of any particular system, provided the community was also involved in the selection of the technology in the initial phase. Subsidies or grants should, when available, be for water supply and sanitation improvements which are required to ensure a basic level of health. The cost of further improvements above this should be borne by the community as far as possible. Willingness to pay should help determine technology choices. Operation and maintenance costs should be recovered in full from the community, and this should be carefully explained when deciding on the level of technology to be implemented.. If operation and maintenance costs were not recovered, budget funds which could be used to supply new consumers, would instead be required to keep the existing systems in operation..

> **"The supply of free water to any given consumer implies that the service may not be able to be extended to others who have equal rights to it."**

iii <u>A third factor of overriding importance</u> is that a high degree of community involvement or participation in the development of the water supply system is a prerequisite for success. Evidence from all over the world has shown that schemes developed by the community (with technical assistance from outside) stand the greatest chance of being sustained in the longer term. Hence the choice of technology must take into consideration the aspirations and preferences of the community itself. But the community must also be able to operate and maintain the selected system in the long term.

In summary, before a technology option can be considered for use in a rural area, it should satisfy at least the three conditions :

- the technology should be sustainable by the community with minimum external support;
- the community should be willing to pay for all operation and maintenance costs, as well as contribute to the capital costs, especially for investments above that required to meet basic health requirements; and
- the community should be involved in the selection of the technology, as well as its implementation and management.

NOTE : The above factors do not preclude the use of modern more sophisticated technology, or the construction of large regional schemes. Provided the community makes the decision as to what system of distribution and management should be used within their own village, such systems could be totally compatible with the principles stated above. However, the associated capital and maintenance costs associated with such schemes must be taken into consideration, and should preferably be discussed with the communities before implementation. The approach whereby large regional schemes are constructed, and then communities are given a "take it or leave it" choice, does not comply with the above approach.

2.2 Technology options

2.2.1 Water supply

Table 2.1 shows different community water supply options as a series of steps. In general, provided satisfactory reliability can be achieved, the level of service rises with each step, as do the total costs, the complexity of the system, and the difficulty of maintaining it.

Table 2.1 Options for Community Water Supply

Step	Type of service	Water source	Quality protection	Water use lpcd	Energy source	Operation and maintenance needs	Costs	General remarks
5	House Connections	Groundwater Surface water Spring	Good, no treatment May need treatment Good, no treatment	100 to 150	Gravity Electric Diesel	Well trained operator; reliable fuel and chemical supplies; many spare parts; wastewater disposal	High capital cost. High O & M costs, except for gravity schemes	Most desirable service level, but high resource needs
4	Yardtaps	Groundwater Surface water Spring	Good, no treatment May need treatment Good, no treatment	50 to 100	Gravity Electric Diesel	Well trained operator; reliable fuel and chemical supplies; many spare parts	High capital cost. High O & M costs, except for gravity schemes	Very good access to safe water; fuel and institutional support critical
3	Standpipes	Groundwater Surface water Spring	Good, no treatment May need treatment Good, no treatment	10 to 40	Gravity Electric Diesel Wind Solar	Well trained operator; reliable fuel and chemical supplies; many spare parts	Moderate capital and O & M costs. But low O & M cost on gravity schemes Collection time	Good access to safe water; cost competitive with hand pumps at high pumping lifts
2	Hand pumps	Groundwater	Good, no treatment	10 to 40	Manual	Trained repairer; few spare parts	Low capital and O & M costs; collection time	Good access to safe water; sustainable by villagers
1	Improved traditional sources (partially protected)	Groundwater Surface water Spring Rainwater	Variable Poor Variable Good, if protected	10 to 40	Manual	General upkeep	Very low capital and O & M costs; collection time	Improvement if traditional source was badly contaminated
0	Traditional sources (unprotected)	Surface water Groundwater Spring Rainwater	Poor Poor Variable Variable	10 to 40	Manual	General upkeep	Low O & M costs (buckets, etc.); collection time	Starting point for supply improvements

This step analogy, however, should not be taken too literally, as the situation will vary from region to region, or even from village to village. For example in South Africa it has often been found that a gravity piped scheme from a protected spring is of a lower cost and requires less maintenance than a hand pump installation. Aspects of the different technologies are described in considerably more detail in the part two of this report.

Choice of technology may be related to a step-wise improvement as in table 2.1. From this table, the steps in water supply improvement are as follows:

step 1 - *improvement of traditional sources.* This offers benefits in terms of time savings for drawing of water, improved water quality and a more secure water source. It does not reduce the distance to the source, or the drudgery associated with carrying containers of water over long distances, but it may be the only choice available to communities who have very little financial or organisational resources. Types of schemes which fit into this category are spring protection and well protection.

step 2 - *hand pumps.* This option will usually have a clear advantage over step 1, particularly since it may be possible to locate the borehole or well at a convenient position within the village and hence reduce the time and energy required for the collection of water. If a good local maintenance programme is implemented, this option may provide a reliable source of water even in times of drought. Handpumps may be installed on existing wells or boreholes which were previously fitted with another pump type (e.g. bucket systems or even motorised pumps which could not be sustained); or on new wells/boreholes specifically sited in consultation with the community and within the physical limits of the aquifer.

steps 3 and 4 - *standpipes and yardtaps.* Unless these options can be fed by a gravity system, they usually imply a significant increase in complexity because of the requirement for motorised pumping. In such cases this improvement could result in a less reliable supply unless the skills to sustain the system are developed within the community. It is then important that the combination of management and technical skills, financial resources, and logistic and technical support are available to ensure sustainability of the system. However, where these options can be supplied the result is usually a vast improvement in the community's water supply. When a clean water source which does not require treatment, and which can be gravity fed is available (e.g. a protected mountain spring), these options can be provided with little increase in complexity and with low operation and maintenance costs. This principle also applies to regional schemes where the community obtains its water under pressure from a regional pipeline which is operated and maintained by a central water supply authority, although the costs in this case will be higher. Other systems which offer a similar convenience to street and yard taps are rainwater tanks, water supplied by tanker, and small scale water vending.

step 5 - *house connections.* This is a step beyond yardtaps and will usually imply the need for proper wastewater disposal facilities in the home. In a rural village the costs for both the house connection and the wastewater removal facilities should be fully borne by the homeowner on a once-off basis. However, it is seldom that off-site wastewater disposal will be appropriate, and the need for adequate on-site wastewater disposal facilities should be stressed to homeowners who obtain house connections. House connections may also be made from rainwater collection systems, although it is seldom that all domestic water needs can be met from this supply. As with standpipes and yard taps, the water supply distribution system may be complex and require moderate to high operation and maintenance supervision.

It may often be found that a combination of various technologies is required to provide optimum improvement in an area. For example in a semi-arid region where the ground water is brackish it may be found that the following combination of technologies provides the best solution:

- rainwater collection for drinking and cooking;
- hand pump on a borehole for domestic hygiene and laundry;
- small dam for livestock watering.

Hence the water uses and associated quality requirements should also be considered when selecting an appropriate water supply. In general, the selection of a community water supply system is based on analysis of the overall costs and benefits associated with each water supply option.

2.2.2 Sanitation

Choice of sanitation technology may be related to the type of excrement disposal system employed. It is debatable whether these represent a step-wise improvement in each case, as some more sophisticated systems may be less suitable and give more problems than a simpler system under the specific conditions prevalent in rural areas. The types of sanitation systems are as follows:

type 1 - *on-site dry latrines*. These systems, when properly constructed, may provide a safe, odour and fly free latrine which is totally acceptable to most uses. Examples of such systems are the SanPlat latrine, the VIP latrine and the composting latrine. Such latrines can be constructed by individuals at a relatively low cost, and are reliable and simple to maintain even when water supply is limited.

type 2 - *on-site wet latrines*. These systems provide a latrine with some form of solids trap (digester) with partial treatment of the solids before the liquid fraction is discharged into the ground through subsurface drains. Such latrines are usually safe for all users, and may have a water seal which ensures an odour and fly free environment. Some systems may have slight odour problems, especially those with only a rough water seal. Examples of these systems are the aquaprivies, tipping tray latrines, anaerobic digesters, pour-flush latrines, and conventional flush latrines with septic tank. All of these systems may give rise to problems should water shortages be experienced, or if the subsurface drain should be inadequate to absorb the volume of water from the latrine. The subsurface tanks (digesters, septic tanks, etc.) will need to be emptied from time to time, depending on the retention capacity of the tank. These systems usually need to be purchased from a supplier, and may be costly. Maintenance requirements may be high, especially when the digester tank is small. Despite these few potential drawbacks of such latrines, they are usually considered by the users as a better system than the pit latrines. If properly installed and maintained, they do provide a very acceptable household or rural institution sanitation system.

type 3 - *off-site disposal with partial on-site treatment.* These systems provide a latrine with an on-site solids trap and partial solids treatment, but off-site treatment and disposal of liquids. This removes the possibility of on-site subsurface drains clogging and seeping out on the surface, and is recommended when water is supplied to each erf with subsequent higher volumes of waste water to dispose of. It also makes possible a significantly lower cost pipe network for transporting the liquid wastes to the disposal site than the conventional water borne systems. Similar pedestals and tanks to those in type 2 above are used, and regular emptying of the solids storage tanks will need to be scheduled. As with type 2 systems, the tank and pedestal must usually be purchased from a dealer at some cost. In addition the pipe network for transporting the liquid wastes and the subsequent liquid treatment and disposal facilities will increase the costs of these systems. Operation and maintenance requirements are high, and skilled personnel are required to operate and maintain the treatment and disposal systems. Simplified treatment systems, such as ponds and reed beds, could reduce these requirements and even result in revenue being generated through aquaculture programmes.

type 4 - *off-site treatment and disposal.* Systems of this type are such that both solid and liquid wastes are removed from site to be treated at an external treatment facility. This means that the on-site facility is simply the pedestal and the pipe or bucket for transporting the excrement to the treatment plant. Examples of this type of system are the bucket latrine and the conventional flush latrine with waterborne sewerage. The external treatment facility must be able to handle both liquid and solid wastes. Bucket systems are not usually recommended because of the health hazards associated with the regular collection and removal of buckets. Waterborne systems provide the most acceptable latrine to most users, but costs, especially in rural areas, are usually very high.

2.3 Cost - Benefit Considerations

Having assessed the possible water supply and/or sanitation options in a given environment, the final choice of technology is usually based on an analysis of the costs and benefits associated with each option. These costs and benefits will not only be based on financial considerations, but also on a number of other factors related to the physical environment, the community and the agency or government department providing support. The costs and benefits relevant to a particular project may include some or all of the following aspects.

2.3.1 Costs

Financial costs

The financial costs associated with a community water supply or sanitation improvement project will usually consist of a number of components, e.g.:

- capital costs
- other construction costs (labour, equipment)
- education and training
- recurrent costs (operation and maintenance)
- agency costs (manpower, travel, etc.)
- consultancy costs (if required)

- *Capital costs*

Capital costs depend largely on the technology under consideration and on local conditions. Table 2.2 lists the capital costs of certain technologies commonly used in rural water supply and sanitation projects. The capital costs of each component of a system must be carefully controlled so as to meet only the needs for which it is required. Expected population increases of 3-4% p.a. over a 15 year period often do not materialise in rural areas. The design should rather be based on an "as is" situation, or limited to projected growth over a 5 year period, unless specific growth plans have been made for the area. Rural towns where some industry or commercial prospects exist would require a more careful projected water need assessment.

Table 2.2 Capital or material costs of various components of a water supply or sanitation systems for rural areas (1990 prices)

COMPONENT	COST RANGE
Spring Protection	R 300 - R 600
Ferro-cement Storage Tank (5 000l)	R 350 - R 700
Ferro-cement Storage Tank (10 000l)	R 525 - R 1 000
Ferro-cement Storage Tank (40 000l)	R 2 500 - R 4 500
Commercial Storage Tanks (5 000l)	R 600 - R 1 500
Borehole Drilling per m (hard material)	R 35 - R 60
Borehole Drilling per m (soft material)	R 180 - R 300
Rainwater Collection (gutters/downpipe)	R 250 - R 750
Hand Pump	R 500 - R 3 500
Diesel Pump	R 2 500 - R20 000
Electric Pump	R 1 500 - R 8 000
Windmill type Pump	R 8 500 - R14 000
Package Treatment Plant (< 100kl/day)	R 1 500 - R40 000
Distribution Pipelines per 100m (32mm)	R 90 - R 250
Distribution Pipelines per 100m (40mm)	R 125 - R 350
Distribution Pipelines per 100m (63mm)	R 310 - R 850
Distribution Pipelines per 100m (75mm)	R 475 - R 1 050
Stand Pipes	R 50 - R 150
SanPlat Latrine	R 75 - R 250
VIP Latrine	R 350 - R 850
Aquaprivy/tipping tray/anaerobic digester latrines	R 1 500 - R 2 000
High Flush Latrines (incl. erf pipes)	R 2 000 - R 2 800
Sewer pipelines + treatment plant	R 2 000 - R 3 500 /erf

Note : 1 US$ = R2.60

■ Labour and other construction costs

Ideally the community will supply all labour on a voluntary basis. However this may not be practical in many situations, or it may not be the most fair system in a community where many people are migrant labourers and others are job seekers. In such cases income earners who are not available to provide labour should be prepared to contribute financially so that those who do contribute labour earn a subsistence income during the construction. If no payment is given to labourers, job seekers may be willing to contribute labour, but could be forced to seek alternative employment elsewhere to earn some subsistence income. Thus where possible some payment for labour should be considered.

The agency supporting the project may place a supervisor on site during construction, whose salary must also be taken into account.

The labour requirements to implement a specific task can be estimated from the activity figures given in table 2.3. These are typical figures for labour, but should be adjusted to take into consideration the level of labour available within the community (e.g. if the labour is to be undertaken primarily by older men and women).

Table 2.3 Productivities for labour

CONSTRUCTION ACTIVITY	RATE (man days)
Excavation < 2m deep (per m^3):	
Ordinary soil	0.45 - 0.7
Gravelly soil	0.80
Boulder mix	1.10
Medium rock cutting	1.60
Hard rock cutting	2.50
Concreting (per m^3 placed):	
Mason labour	1.1
Unskilled labour	4.0
Brick masonry (per m^3 placed):	
Mason labour	1.2 - 1.5
Unskilled labour	2.8
Plastering (per m^2 placed):	
Mason labour	0.15
Unskilled labour	0.25

The value of the labour depends on the skills level required, the location, and on the perceived value of time by the community. Rates in rural areas of R 8 to R 12 per day (1990 prices) are usual for piece work, and this could form the basis for determining the value of the labour contributed.

In addition to labour, certain equipment will be required during the construction and implementation phase. Besides that provided by contractors for the sinking of boreholes etc., tools will be required by the labour force for masonry, earth works, plumbing, etc. These tools could either be purchased for each job and then given to the community members who have contributed their labour, or kept for other such projects should the implementing agency be involved in further projects.

- *Education and training*

Elected persons from the community should be sent on training courses to learn how to construct the various elements of the water supply improvement project, how to operate and maintain the system once it has been completed, and how to plan and manage such a project through a village water committee. In addition the agency or government department concerned should initiate a general educational thrust within the community itself, focusing on health education (domestic hygiene), the need for community involvement and support, and discussing details of the project. All of these aspects will have associated costs.

- *Recurrent costs (operation and maintenance)*

Regardless of how the capital purchases of the water supply project are financed, recurrent costs should be borne by the community. Community management, including financial management, of system operation and upkeep is the only way to achieve acceptable reliability at an affordable cost. Incomes in the rural areas are typically in the range R50 to R500 per month. Recurrent costs should constitute only a small proportion of that income. To ensure commitment by the community to this, it is necessary to ensure that representatives of the community are involved in the planning and decision making aspects of the project from an early stage, and that the community is kept well informed at all stages of the project. Recurrent costs of water supply schemes depend on how much, if any, the elected caretaker will be paid by the community. In the case of supply systems comprising pumps, the time and skills required from the caretaker will usually be more than what can be expected from the caretaker without some monthly payment. Operation and maintenance figures for various schemes are given in table 2.4 below.

Table 2.4 Operation and maintenance costs of various water supply or sanitation systems for rural areas (estimates, excluding caretaker allowance/salary) [1990 prices]

SYSTEM	COST RANGE (R/annum)
Spring Protection Works	R 30 - R 250 /system
Rainwater Collection Systems	R 10 - R 50 /household
Hand Pump on a Borehole	R 50 - R 150 /pump
Diesel Pump on a Borehole	R 500 - R5 000 /pump
Electric Pump in a Borehole	R 400 - R4 000 /pump
Wind Pump on a Borehole	R 150 - R 800 /pump
Package Treatment Plant (< 100kl/day)	R 200 - R4 000 /system
Distribution Network	R 50 - R1 000 /system
On-site dry sanitation system	R 15 - R 50 /household
On-site wet sanitation system	R 30 - R 150 /household
Off-site treatment (solids trap on-site	R 80 - R 250 /household
Off-site treatment and disposal	R 120 - R 350 /household

Note that costs associated with diesel and electric pumps, as well as treatment plants, are primarily governed by running costs for the purchase of diesel, electricity, and chemicals respectively. Other costs are related to maintenance costs for such items as pipe leaks, concrete repairs, tap replacements, and external repair services. Costs will vary depending on location, number of users, level of operational personnel, etc.

■ *Agency costs (manpower, travelling, etc.)*

The costs associated with the manpower and running costs of the agency or government department involved with the project may make up a significant portion of the total project costs. However this can normally be justified because of the lengthy time required to ensure community acceptance and involvement, and the wide variety of skills required. Small, non-government organizations are often able to provide this service at a lower cost than larger organizations or government departments. However, the organization or government departments with the overall responsibility for water supplies and health in the region should always be kept informed, and preferably included in the project planning and execution. These departments are responsible for the long term maintenance of services and the development in the region. Government departments, at the same time, would do well to make use of and collaborate with the smaller development organisations for the implementation of rural schemes. The smaller development organisations usually are able to carry lower overhead costs, and can developed a good level of cooperation with community members.

Where community management is not effective, or the level of skills required is not available in the community, the effective operation and maintenance of the water supply and/or sanitation systems may require the ongoing, long term involvement of the agency in the project.

■ *Consultancy costs*

It may often be advantageous to employ additional consultants who have relevant knowledge and experience. This is particularly important where the agency either has little technical expertise, or has had little experience in working with the community. This input would normally be defined in relevant contract documentation, and would replace some of the agency costs as defined above.

Non financial costs

Non financial costs or costs which cannot be easily quantified may include the following:

- time required to collect water (important for comparing alternatives);
- health problems associated with poor water supply and/or sanitation, and resulting costs (disability, health care, time);
- aesthetic impact of blocked latrines, poorly drained standpipes, etc.;
- poor image of a technology if inappropriately used or poorly constructed (this is especially relevant with sanitation systems e.g. VIP latrines, and may make its further use in an area unacceptable);
- environmental degradation (usually associated with large projects only, e.g. the flooding of habitats due to the construction of a dam);
- loss of community support and unity (for projects implemented without community involvement).

To quantify these costs so that alternative projects can be compared is somewhat difficult. Often a monetary value can be associated with time savings as a way of comparing costs, but the importance of the other factors must be compared on a perceived effect basis. The analytical hierarchy process (AHP) described in appendix A is a way of dealing with these costs should the comparison become complex. It is important though, that all non-financial costs be considered when selecting the technology for a water supply or sanitation upgrading project.

2.3.2 Benefits

The benefits associated with an improved community water supply or household sanitation are seldom direct financial benefits. Rather the benefits are associated with factors which are difficult to measure in financial terms. Some of the benefits which may be associated with an improved village water supply are:

- time savings in the collection of water;
- reduced drudgery in the collection of water;
- better living conditions due to more easily available water for washing and bathing, and hence greater cleanliness;
- health benefits in terms of reduced sickness in the community from water related diseases resulting in reduced medical care and health service costs, as well as improved productivity (especially where the water project forms part of an overall development package);
- development of organizational, management, and technical skills within the community, and hence the opportunity for undertaking further development projects;
- improvement of unity in the community, and their capacity for self improvement;
- environmental protection of water source areas and reduction of the need to use steep paths down to springs and streams with subsequent soil erosion possibilities; and
- creation of short- and longer-term job opportunities.

Sanitation improvement benefits are primarily health related, although job creation, environmental protection, better living conditions, and status may be important perceived benefits.

Often the benefits of the supply options which are under consideration in a given situation will only become evident through close contact with the community themselves. Ongoing support from the development agency is often required to realise these benefits in the longer term.

Benefits are influenced by two key technical factors :-

the service level	(i.e. the distance water must be hauled from the water point to the dwelling, queuing plus filling time for each water hauler, and the ease of drawing water, or likelihood of smells from the latrine); and
system reliability	(assurance of water being available at the water point when needed, or likelihood of latrine blocking).

Based only on service level and associated financial costs, projects may be compared fairly rigidly in terms of costs, time savings, and (human) energy needs. Note that the value of time savings will not in general be the same in monetary terms as that used for estimating labour costs. In the rural areas a value of time in the range R0.20 to R1.00 per hour may be applicable. An example of such an analysis is presented in the following section.

2.4 Technology choice

The choice of the most suitable technology for a given situation should undergoe the selection process represented in figure 2.1 below:

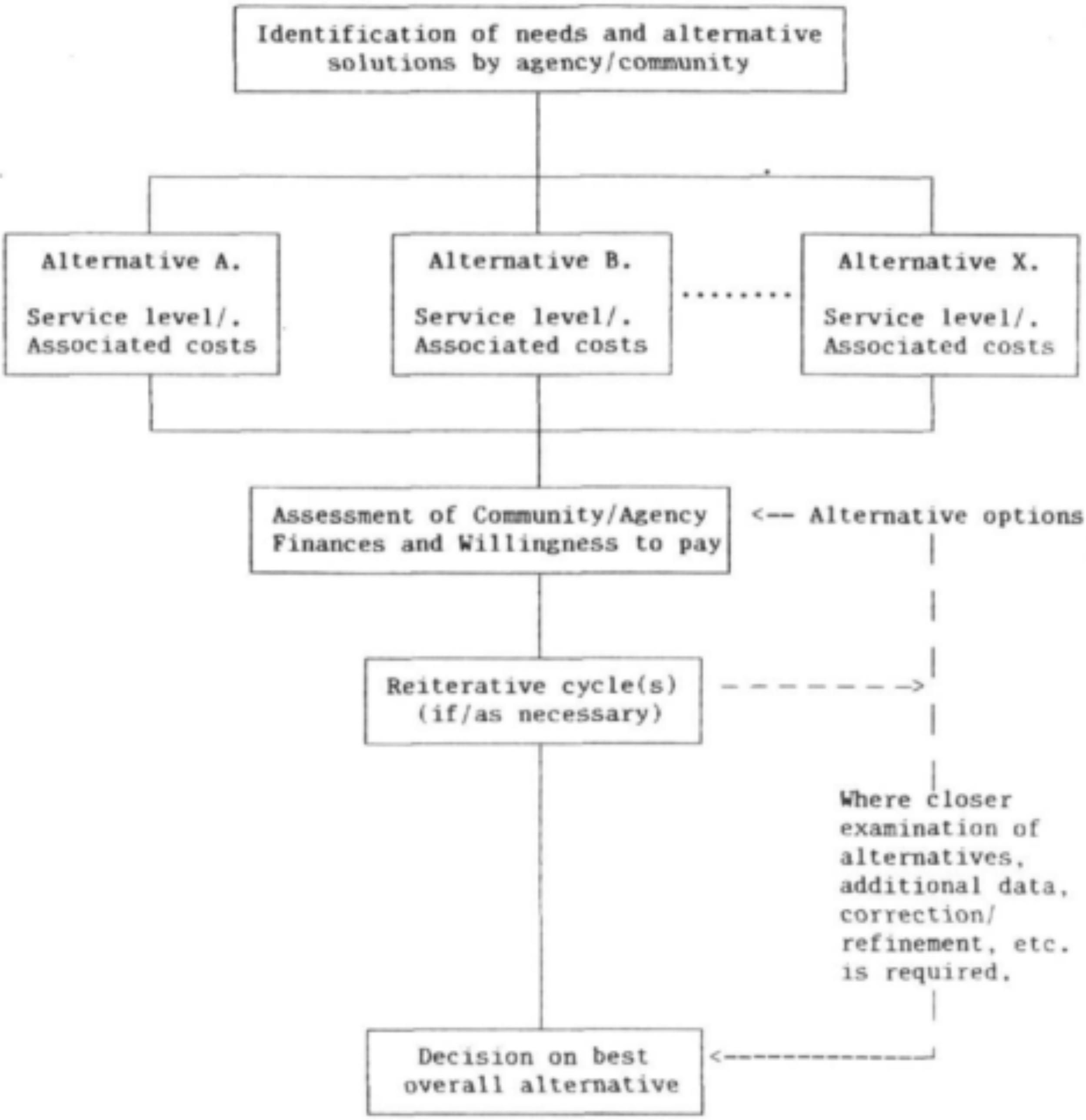

Figure 2.1 Community based decision-making model

Important steps in this process are as follows:

- discussion with community on their needs and aspirations;
- assessment of technical alternatives;
- assessment of skills available in the community;
- assessment of affordability and willingness to pay;
- assessment of additional finances which can be obtained to support or subsidise a project;
- selection of best alternative in consultation with the community.

As already mentioned, it is important that community members (especially women) are actively involved in the decision making process, and that they themselves discuss options, associated service levels and costs, sources of cash and in-kind inputs, etc., and then select, in negotiation with the agency, the technology best suited to their own situation. This type of decision making process can lead to more realistic projects, and lower investment and operational phase costs. It is important to realise, allow for and accept that this community consultation process takes time and skilled guidance by trained community workers.

2.4.1 Comparison of costs and time savings for water supply projects

In order to carry out an analysis of the costs and time savings associated with a particular alternative, the parameters in table 2.5 will be required.

Table 2.5 Parameters for community water supply scheme

PARAMETER	TYPICAL VALUES		
Demographic characteristics			
Total population	100	to	20 000
Persons per household(Av)	5	to	10
Total households	20	to	2 500
Density - distance between homes	50	to	500 m
- households per hectare	0.2	to	20
Economic conditions			
Av. value of time	20c	to	R 1.00/h
Av. household income (R/month)	50	to	600
Capital discount rate	0.10	to	0.20
Diesel cost	R1.00	to	R1.80/l
Electricity cost	10c	to	18c/kWh
Chlorine cost	1.50	to	R7.50/kg
Village water use and collection	HP/SPR	SP	YT
Distance to water source	500m	250m	10m
Amount of water carried per trip	20l	20l	-
Walking rate (km/h)	4	4	-
Av. queuing and filling time/trip	5min	5min	-
Daily water consumption (l/cap)	20	30	60
Water supply system	HP/SPR	SP	YT
Number of water points per 100 homes	2	5	90
Capital cost per water point (R)	3000	2500	200
Storage volume/daily flow	0/0.6	0.3	0.3
Useful life (years)	10	10	10
Annual O&M costs	750	3500	5000

Note - in table 2.5, the following abbreviations and assumptions have been used:

HP/SPR = hand pump or protected spring
SP = stand pipes supplied by diesel or electric pump and a distribution system
YT = yard taps supplied by diesel or electric pump with storage and a distribution system
chlorine costs based on on-site generation (R1.50/kg) to granular calcium hypochlorite (R7.50/kg as chlorine).

The comparison of costs and benefits is best shown by means of an example.

EXAMPLE

Demographics
A rural community of 1 500 people have an average of 6 people per household giving a total of 250 homes. The average distance between households is 75m and there are approximately 1.25 households per hectare, and hence a total area of some 200 ha (2 km²).

Economics
The estimated value of time in the community is 40c/h based on the agricultural output of the women in a communal garden project. The average household income is R400 per month, but ranges from R80/mnth to R1200/mnth. There is no electricity available in the area, and diesel costs R1.50 per litre. Chlorine in the form of dry granular chlorine can be obtained at a delivered cost of R7.00/kg as chlorine. The capital discount rate is taken as 15% or 0.15.

Water use/supply
Presently people obtain water from two different sources:
i) a spring where the collection point is, on average, 1.5km from all households; and
ii) a stream which is mainly used for washing of clothes which is, on average, 1.0km from all homes.

During the dry season the spring flow is low and it takes approximately 10 minutes to queue and fill a 20l container at the spring. Water consumption is approximately 12l/cap/day excluding the washing of clothes.

Present cost of water supply
The time consumed in collecting water for domestic purposes is as follows:

walking = 3km (return trip) @ 4km/h	=	45 minutes
queuing and filling	=	10 minutes
total per trip	=	55 minutes

water consumed per household/day	= 12l/cap x 6	= 72l
number of trips per day	= 72l/20lper trip	= 3.6
at 55 minutes per trip this means	3.6 trips x 55	= 3.3h per home
at 40c/h the value of this time is	3.3h x 40c	= R1.32/day
for the community as a whole this is	R1.32 x 250	= R330/day

Supply options

The following supply options are considered feasible:

i) Protect spring and construct a 10 000l storage tank

capital cost (incl. labour) = R 3 000
discount over 10yrs @ 15% = R 600/a
annual O&M costs (incl. caretaker) = R 500/a
time savings = 7 minutes/trip due to reduced queuing and filling time
= 25 mins/household/day
@ 40c/h this means 17c per household per day, or
R42/day for the community as a whole
annual savings amount to R15 200

ii) Drill 5 boreholes and fit 3 with handpumps (60% success rate)

capital cost = R35 000
discount over 10 years = R 7 000/a
annual O&M costs (incl. caretaker) = R 900/a
(assume average distance to handpump is 300m)
time per round trip = 2 x0.3/4 =0.15h = 9mins
time savings (walking) = 45 - 9 = 36 minutes/trip
time savings (queue/fill) = 10 - 7 = 3 minutes/trip
total time saved per day = (36+3)x3.6 = 2.34h/day
@ 40c/h this means 93c/household per day, or
R234 for the community as a whole
annual savings amount to R85 400

iii) Each household build a 10 000l rainwater collection tank

capital cost (250 x R1 000) = R250 000
discount over 10 years = R 50 000/a
annual O&M costs = 6 250/a
(assume provides domestic water for 75% of year)
time savings (75% of 3.3h/day) = 2.5 h/day
@ 40c/h this means R1.00/household per day, or
R250/day for the community as a whole
annual savings amount to R90 000

iv) Protect spring and distribute to standpipes closer to homes

capital cost (spring, storage, 3 km pipe, 10 taps)
(R3 000 + R6 750 + R 850 + supervision) = R15 000
discount over 10 years = R 3 000/a
annual O&M costs (incl. caretaker) = R 1 750/a
(assume av. distance to standpipe is 250m)
time per round trip = 0.5/4x60 = 7.5 mins
time savings (walking) = 37.5 mins/trip
time savings (queue/fill) = 5 mins/trip
total time saved per day = 42.5x3.6 = 2.55h
@ 40c/h this means R1.00/ household per day, or
R250/day for the community as a whole
annual savings amount to R90 000

v) Drill 2 boreholes, equip one with a diesel pump and storage

capital costs	= R30 000
discount over 10 years	= R 6 000/a
annual O&M costs	= R23 000/a

(assume av. distance to reservoir is 500m)
time per round trip = 1.0/4x60 = 15 mins
time savings (walking) = 30 mins/trip
time savings (queue/fill) = 5 mins/trip
total time saved per day = 35x3.6 = 2.1h
@ 40c/h this means 84c/household per day, or
R210/day for the community as a whole
annual savings amount to R76 600

vi) As for (v), but distribute to yard taps

capital costs	= R50 000
discount over 10 years	= R10 000/a
annual O&M costs	= R28 000/a

time savings (walking) = 45 mins/trip
time savings (queue/fill) = 10 mins/trip
total time saved per day = 55x3.6 = 3.3h
@ 40c/h this means R330/day for the community as a whole
annual savings amount to R120 000

vii) As for (v), but with a windmill as pump

capital costs	= R30 000
discount over 10 years	= R 6 000/a
annual O&M costs	= R 2 500/a

value of time savings as for (v)

viii) Construct small dam on stream, purify with slow sand filters and chlorination, pump to storage, distribute to stand pipes

capital costs	= R55 000
discount over 10 years	= R11 000/a
annual O&M costs	= R17 500/a

value of time savings as for (iv)

Table 2.6 below summarises these results, and indicates the net benefit for the different options for different values of time.
Net benefit = benefit (value of time saving) - cost

The ratio (net benefit)/cost gives an indication of the return resulting from the investment in the particular water supply scheme. Note that in this case only the financial costs and time saving benefits have been taken into consideration. Table 2.6 indicates the variation in benefits with changes in the value of time. In a similar way the consequences of variations in the distance to the original source, pumping lift, capital and O&M costs, size of village population, cost of pumping, etc. Simple computer programmes can be compiled to ease the repetitive calculations, and to plot the consequences of changing various variables. However it should always be remembered that this is just a tool to helping the decision making progress, and should not be used to influence choices on its own.

TABLE 2.6 : Summary of cost/benefit for various options wrt example

value of time (per h)	R/a x 1000	OPTION							
		i	ii	iii	iv	v	vi	vii	viii
	cap. cost	0.6	7.0	50	3	6	10	6	11
	O&M cost	0.5	0.9	6.3	1.8	23	28	2.5	17.5
10c	benefit	3.8	21.4	22.5	22.5	19.1	30	19.1	22.5
	net benefit	2.7	13.7	-33.8	17.7	-9.9	-8	10.6	-6
	net ben/cost	2.5	1.7	-0.6	3.7	-0.3	-0.2	1.2	-0.2
20c	benefit	7.6	42.8	45	45	38.2	60	38.2	45
	net benefit	6.5	34.9	-11.3	40.2	9.2	22	29.7	16.5
	net ben/cost	5.9	4.4	-0.2	8.4	0.3	0.6	3.5	0.6
40c	benefit	15.2	85.4	90	90	76.6	120	76.6	90
	net benefit	14.1	77.5	33.7	85.2	47.6	82	68.1	61.5
	net ben/cost	12.8	9.8	0.6	17.8	1.6	2.2	8.0	2.2

General statements that can be made from an analysis of this nature are the following:

- As time cost values increase, so yardtaps become more competitive. Above a certain value, yardtaps will always show the greatest net benefit.

- Except for the protection of the traditional source (e.g. spring protection and storage with or without further reticulation), technology selection is virtually independent of the distance to the traditional source. Distance to the traditional source will only influence the magnitude of the benefit, not the relative advantage.

- For ground water supply systems, at pumping lifts up to 25m, hand pumps will normally prove to be optimum. For lifts from 25 to 45m, electric pumps may be more advantageous if electricity is available, otherwise handpumps. At depths of 45m or more, either electric (if available) or diesel pumps will usually be more advantageous. At higher time cost values, power driven pumps with reticulation will always prove to be more advantageous.

- In the above analysis, rainwater collection did not come out favourably. However, for small populations the advantage of such systems is greatly enhanced. For very large populations, standpipes or yardtaps tend to be optimum solutions on this basis.

- Technology choice will always be strongly affected by local supply considerations.

The benefits arising from the availability and use of more water have not been specifically addressed here, but could be included in the analysis if desired. Health benefits could also be taken into consideration, although these are difficult to quantify.

2.5 Summary of steps for project selection

In summary, the following steps should be considered, and if appropriate included in the development of a community water supply and/or sanitation project in rural areas:

- make contact with the community to assess their needs and to assess their willingness to participate in such a project;
- make contact with other relevant authorities who are working in the area, including tribal authorities, to obtain their support and to avoid a conflict of interests and priorities;
- with the support of the community, undertake a project feasibility study in which the technical and institutional options are assessed;
- carry out a cost/benefit analysis on the various options, which will include a preliminary concept design of the technical options;
- discuss the options with the community representatives, and if possible with the community themselves;
- plan financing and other inputs required to implement the chosen option;
- carry out a detailed design of the chosen option;
- organise for the necessary training programmes to get underway;
- supervise and implement;
- commission and monitor the performance of the installed system.

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A P P E N D I X   A

THE COST-EFFECTIVENESS OF RURAL WATER SUPPLY AND SANITATION PROJECTS BY THE ANALYTICAL HIERARCHY PROCESS

1. INTRODUCTION

When evaluating the cost-effectiveness of alternative projects, the most appropriate method of comparative costing has to be selected. A cost minimization rule can be applied in the case of mutually exclusive alternatives with identical benefits. In such cases the alternative with the lowest present value should be selected. However, this tends to be an oversimplification in most cases.

The least cost project will often not be the most cost-effective one where there are differences in the output or service. Alternative Sanitation and Water Supply systems provide a wide range of benefit levels. Although most properly selected systems can be designed to provide the potential for full health benefits, many benefits exist in the mind of the user, and varying qualities of service result in varying benefit levels. A least cost comparison will therefore not provide sufficient information to select among project alternatives. An example might be a case where pit-latrines are installed without taking the fears of small children into account. This might lead to them refusing to use the system for fear of falling into the pit. A safer, though more expensive latrine, would have been a better alternative in this case.

Benefit-cost ratio

A commonly used measurement in project evaluation is the benefit-cost ratio. the advantage of this method is that it provides a single, summary figure representing the net economic effect of a given project. This figure can then be readily compared with that of alternative projects. The disadvantages of benefit-cost calculations are:

1. They do not easily accommodate non-economic benefits and costs. (Particularly if these are unquantifiable).
2. They may give misleading results if applied to mutually exclusive projects.
3. They may not reflect macro-economic goals such as the creation of employment or the generation of savings and investments.

There are, however, difficulties in measuring benefits for sanitation projects. In the case of water supply projects, it has been concluded that it is inappropriate to attempt to measure the benefit as part of project appraisals - ('Measurement of the Health Benefits of Investments in Water Supply, The World Bank, 1976'). This is mainly due to theoretical and empirical problems involved in quantifying incremental health benefits. There are also unquantifiable costs associated with alternative sanitation technologies. An example of such costs may be the environmental consequences of installing a particular system.

There is therefore a need for a somewhat more involved method of cost-comparison for projects with unquantifiable costs and benefits.

Analytic Hierarchy Process (AHP)

Thomas L Saaty developed an approach to multiple criteria decision making based in part on pairwise comparisons of preference for elements in a hierarchy. The process requires the decision maker to provide judgements about the relative importance of each of the criteria and then to specify a preference for each decision alternative relative to each criterion. The output of the AHP is a prioritized ranking indicating the overall preference for each decision alternative.

The first step in the AHP is to develop a graphical representation of the problem in terms of the global goal, the criteria, and decision alternatives. Such a graph depicts the hierarchy of the problem. Fig A1 shows an example of a hierarchy for a sanitation-selection problem.

```
Overall      .        Selection of the most cost-effective
goal:        .                  Sanitation system
             .                          |
             .                          |
             .      ____________________|____________________
             .      |              |              |              |
Criteria     . Financial      Community      Health       Environmental
             .   cost         acceptance     benefit         impact
             .      |              |              |              |
Decision     .  System A       System A       System A       System A
alternative. .  System B       System B       System B       System B
             .  System C       System C       System C       System C
```

Figure A1 : Heirarchy for sanitation selection

It is important to note that each decision alternative can contribute to each criterion in an unique way.

Establishing priorities using the AHP

Pairwise comparisons are used to establish priority measures for both the criteria and the decision alternatives. Two main types of priorities need to be established:

1. The priorities of all the criteria in terms of the contribution of each towards the overall goal.
2. The priorities of the decision alternatives in terms of each criterion.

   For each criterion the decision alternatives are considered two at a time (pairwise). A scale with values from 1 to 9 score the relative preferences expressed for two items. Table A1 provides the numerical scores recommended for the verbal preferences expressed by the decision maker.

TABLE A1

VERBAL JUDGEMENT OF PREFERENCES	NUMERICAL RATING
Extremely preferred	9
Very strongly to extremely	8
Very strongly preferred	7
Strongly to very strongly	6
Strongly preferred	5
Moderately to strongly	4
Moderately preferred	3
Equally to moderately	2
Equally preferred	1

It is necessary to develop a matrix of the pairwise comparison scores in order to develop the priorities further e.g. for Financial Cost criteria (fictitious figures):

FIN. COST	SYSTEM A	SYSTEM B	SYSTEM C
System A	1	2	8
System B	1/2	1	6
System C	1/8	1/6	1

Note: In the pairwise comparison matrix, the value in row i and column j is the measure of preference of the system in row i when compared to the system column j.

The matrix is then normalized by dividing each element in the matrix by its column total.

i.e. In our example:

1	2	8	----->	8/13	12/19	8/15
1/2	1	6		4/13	6/19	6/15
1/8	1/6	1		1/13	1/19	1/15

Tot. 13/8 19/6 15

Normalized pairwise comparison matrix.

Note that all the columns in the normalized pairwise matrix now have a sum of 1.

The elements of each row in the matrix is now averaged.

FIN. COST	SYSTEM A	SYSTEM B	SYSTEM C	ROW AVE.
System A	3/13	12/19	8/15	0.593
System B	4/13	6/19	6/15	0.341
System C	1/13	1/19	1/15	0.066
			TOTAL	1.000

The priority vector showing the relative priorities of System A, B and C with respect to Financial Cost is written as follows:

0.593
0.341
0.066

A similar priority vector can be set up for each criterion e.g.

	FINANCIAL COST	COMMUNITY ACCEPTANCE	HEALTH BENEFIT	ENVIRONMENTAL IMPACT
System A	0.593	0.087	0.123	0.265
System B	0.341	0.274	0.320	0.655
System C	0.066	0.639	0.557	0.080

A pairwise comparison matrix must now be set up for the criteria, using the same principals as before. In this case, however, they are compared with the overall goal in mind. From this a criteria priority vector can then be set up using the same principals as before:

e.g.

Priorities for the overall goal:

Financial Cost	0.218
Community Acceptance	0.085
Health benefit	0.398
Environmental impact	0.299

Developing an overall priority ranking

A matrix has to be set up that summarises the priorities for each decision alternative. This matrix is referred to as the priority matrix.

	FINANCIAL COST	COMMUNITY ACCEPTANCE	HEALTH BENEFIT	ENVIRONMENTAL IMPACT
System A	0.593	0.087	0.123	0.265
System B	0.341	0.274	0.320	0.655
System C	0.066	0.639	0.557	0.080

The overall priority for each decision alternative is obtained by summing the product of the criterion priority times the priority of the decision alternative with respect to that criterion. This process can best be understood if the priority for each criterion is seen as a weight that reflects its importance.

System A = 0.593 (0.218) + 0.087 (0.085) + 0.123 (0.398) + 0.265 (0.299)
Priority = 0.265

System B = 0.341 (0.218) + 0.274 (0.085) + 0.320 (0.398) + 0.655 (0.299)
Priority = 0.421

System C = 0.066 (0.218) + 0.639 (0.088) + 0.557 (0.398) + 0.080 (0.299)
Priority = 0.314

Ranking these priority values we have the following AHP ranking of the decision alternatives:

ALTERNATIVE	PRIORITY
System A	0.265
System B	0.421
System C	0.314
TOTAL	1.000

Whether one decides to implement the choice that has been derived in this way may not be as important as the additional understanding of the problem that one has obtained as a result of performing the analysis required by the AHP.

Consistency and the AHP method

The AHP provides a measure of the consistency of pairwise comparison judgements by computing a consistency ratio. This ratio is designed in such a way that values of the ratio exceeding 0.10 are indicative of inconsistent judgements. In such cases, the decision maker would probably want to reconsider and revise the original values in the pairwise comparison matrix. Values of the consistency ratio of 0.10 or less are considered to indicate a reasonable level of consistency in the pairwise comparisons.

The example which was used to derive the ranking of the decision variables will be expanded to show how to calculate the consistency ratio.

The first step is to separate the columns of the pairwise comparison matrix. Each column is then multiplied with the relative priority of the corresponding system. The values obtained for each column are then summed across the rows. This then forms the 'weighted sum' vector.

$$
\begin{array}{r}
0.593 * \text{col1} = \\
+\ 0.431 * \text{col2} = \\
+\ 0.066 * \text{col3} =
\end{array}
\begin{bmatrix} 0.593 \\ 0.297 \\ 0.074 \end{bmatrix} + \begin{bmatrix} 0.682 \\ 0.341 \\ 0.057 \end{bmatrix} + \begin{bmatrix} 0.528 \\ 0.396 \\ 0.066 \end{bmatrix} = \overset{\textbf{WEIGHTED SUM VECTOR}}{\begin{bmatrix} 1.803 \\ 1.034 \\ 0.197 \end{bmatrix}}
$$

Divide the elements of the vector of weighted sums by the corresponding priority value.

1.803/0.59 = 3.040
1.034/0.341 = 3.032
0.197/0.066 = 2.985

The average of the values computed above is denoted by λ_{max} = 3.019

The consistency index is defined as follows:

$$CI = \frac{\lambda_{max} - n}{n - 1} \quad \text{where } n = \text{number of systems}$$

$= 0.010$ in this case.

The random index, RI, is the consistency index of a randomly generated pairwise comparison matrix. It takes on the following values:

n	RI
2	0.00
3	0.58
4	0.90
5	1.12
6	1.24
7	1.32
8	1.41

The consistency ratio, CR, can now be computed as follows:

$$CR = \frac{CI}{RI} = \frac{0.010}{0.580} = 0.017$$

As mentioned previously, a consistency ratio of 0.10 or less is considered acceptable.

It should be pointed out that the AHP is only one approach to decision analysis with multiple criteria. An important feature of these types of approaches to multiple-criteria decision making is that they enable the decision maker to apply the methodology in problem situations that depend largely on subjective matter.

PART II:

GUIDELINES ON THE TECHNOLOGY FOR AND MANAGEMENT OF RURAL WATER SUPPLY AND SANITATION PROJECTS

PART II: GUIDELINES ON THE TECHNOLOGY FOR AND MANAGEMENT OF RURAL WATER SUPPLY AND SANITATION PROJECTS

CONTENTS

TECHNOLOGICAL ASPECTS OF RURAL PROJECTS

1. INTRODUCTION

Water is essential to man, animals and plants. Water is required by man for consumption and hygiene purposes, as well as for aesthetics and an improvement in the quality of life. Man uses water for the production of all his food and drinks, and in the production of most building materials, commercial and industrial products. Although the availability of safe, clean water is often taken for granted in towns and cities, its procurement in the rural areas has important social and economic implications for the community or individuals concerned. In many of the rural areas the collection of water for domestic use is a major task in the daily chores of a typical family. Due to population increases, climatic conditions, and the often negative impact of man's development, the quantity and quality of the water available to rural families has come under stress in more recent times. This has resulted in people having to travel long distances to fetch their water supplies, and then these supplies are often contaminated. Outbreaks of diseases like typhoid and cholera are serious, but can usually be controlled and contained by appropriate medical intervention. These measures, however, have a significant cost associated with them. Despite medical advances, a large number of children, particularly in the rural areas, die each year from gastro-intestinal diseases which could have been prevented with proper water supply, sanitation and education.

The participants of the United Nations Water Conference at Mar del Plata in 1977 reached a number of recommendations for rational development, management and utilisation of water resources. All participating countries supported the resolution that:

> **"All people have the right to have access to drinking water in quantity and of a quality equal to their basic needs"**

These conclusions lead to the decision by the United Nations to proclaim the period from 1981 to 1990 as the "International Drinking Water Supply and Sanitation Decade". The aims of the Decade i.e. to supply every one on earth with a minimum level of water supply and sanitation by the year 1990, were ambitious and praiseworthy, but unrealistic. Nevertheless much has been learned through many hard lessons by the international community during the decade, and it is envisioned that we are now much better equipped to carry out successful water supply and sanitation projects in rural and developing areas than we were at the beginning of the decade.

In South and Southern Africa a number of government and non-government agencies have taken up the challenge realistically and seriously. During the drought years of 1983 to 1986 a number of initiatives were taken to improve and develop new water sources in rural and metropolitan areas. Many of these initiatives have continued since the drought years, and has resulted in a large number of rural water supply improvement schemes having been implemented. The lessons learnt internationally on the need for community participation, for education and training, and for the use of appropriate technologies, have also been part of the Southern African experience. Besides government actions to supply water to rural communities in Southern Africa, a number of other organizations have made significant advances in this sphere, some of which have been recognised internationally. Included in these are the Valley Trust in Natal (community participation, training and appropriate technology), the Blair Research Laboratory in Zimbabwe (appropriate technology), the Amatikulu Training Centre in Natal (training and education), Drought Relief in Bophuthatswana (community participation), the Rural Industries Innovation Centre in Botswana (appropriate technology), the Lesotho urban and rural sanitation programmes (appropriate sanitation technology and community involvement), and the Southern African branch of the IWSA (technology transfer). In addition a large number of organisations are now involved with developing and implementing improved water supplies and sanitation in the rural and developing areas of Southern Africa.

It has been realised that community management of water supply projects is vital to their long term success when government support is limited. This has been taken up by those organisations focusing on community participation in particular. The long term success of such initiatives have borne fruits in terms of sustainable systems without extensive external support.

These guidelines attempt to bring together the lessons learnt and the technologies which are applicable to water supply and sanitation in rural areas, with particular reference to the Southern African situation.

2. WATER SUPPLIES AND SANITATION IN RURAL SOUTH AFRICA

In the rural areas of South Africa, water has traditionally been obtained from rivers, wells and springs. As technology for improving water supplies was introduced into the region, and as rural villages and towns became established and growing, new methods for supplying water in bulk to many consumers were gradually implemented. Today most of the larger villages and towns have some form of improved water supply. These supplies are not always reliable or sufficient, and residents may often have to revert to traditional unprotected sources until the supply is restored.

In the same way sanitation has traditionally been through people using shallow holes in the areas away from the homes, or through roughly constructed pit latrines. Improved latrines have not been extensively adopted in the rural areas of South Africa, while in the neighbouring countries of Lesotho and Zimbabwe they have been. Presently it is estimated that less than 20% of rural residents in South Africa have access to adequate sanitation facilities.

A brief survey was undertaken to assess the actual level of coverage in the rural areas in terms of water supply and sanitation facilities. It was found that in most areas little data is available on water supply and sanitation coverage, and on the needs for improved facilities in the various areas. Hence the data given below reflects only estimates of what the situation is in most cases.

2.1 Lebowa

2.1.1 Background

Lebowa is one of the national states that is situated in the Northern Transvaal. The rural population is approximately 2 500 000. Until the end of 1988, the Department of Agriculture and Environmental Conservation was responsible for the provision of water in the rural areas. Currently, a Water Affairs section within the Department of Agriculture has been established to deal with water supplies in rural areas. This section will probably develop into an autonomous body in the near future.

2.1.2 Existing water supply and sanitation situation

In 1986, Lebowa had a rural population of approximately 2½ million people with an annual growth rate of 3 %. The rural people depend mainly on water from boreholes, rivers, wells and springs. In some cases, treated water is brought into the community from the nearest town, for example, Tafelkop village near Groblersdal. Other water supply developments include borehole water reticulation to villages. However, costs for construction, operation and maintenance of the schemes have usually been fully borne by government. The Department of Water Affairs is concerned that a large portion of their budget for water supply is being used for maintenance of the existing rural schemes. Hence, plans to change the top-down approach to more community-based water schemes are being implemented.

Like in most national and independent states, Lebowa has applied for funds to implement regional water schemes from the Development Bank of Southern Africa. These are long term projects which might take about five to ten years before they are implemented. As far as sanitation is concerned, there is no department which is specifically responsible for rural sanitation. However, the Department of Health assists in building latrines at public places like schools and clinics and at the same time educates people on the need to build toilets. Thus far, there are few significant improvements as far as sanitation in rural homes is concerned.

It is estimated that over 50 % of the rural population in Lebowa is still without easy access to an assured safe water supply and over 80% without adequate sanitation (1990).

2.2 Gazankulu

2.2.1 Background

This national state is situated in the North-Eastern Transvaal. Relatively speaking, Gazankulu is a small homeland with a rural population of approximately 750 000. Surveys to establish the prevailing water supply and sanitation situation were undertaken in 1986 and 1990.

In 1986, two Departments, i.e. Works and Agriculture, were responsible for rural water supplies. However, more recently, the Department of Works through its Water Affairs section has taken over the responsibility for the provision of water in the whole area.

2.2.2 Existing water supply and sanitation situation

In Gazankulu, there are approximately 750 000 people in rural and 36 000 in urban areas with a growth rate of 3 %. Approximately 90 % of the rural population in Gazankulu has access to piped water supplies. The remaining 10 % use boreholes (in drier areas) and springs.

The principal health problem in the area due to waterborne diseases is typhoid. This problem is predominant in the Elim and Rhitavhi areas. At the moment, rural people including those who have access to piped water do not pay for their water supply. However, businesses like butcheries, shops and local authorities who have water reticulated into their yards do pay a nominal fee for water. The maintenance of all the schemes is solely the Department's responsibility.

In practice, although the regional water schemes are in place in most areas, water is often not available at taps. The proper functioning and maintenance of the regional projects is a major task, even when under control of a government department.

It is estimated that some 20% of the rural people have access to improved sanitation facilities.

2.3 Venda

2.3.1 Background

Venda is an independent state situated in the far Northern Transvaal. Investigations into the prevailing water supply and sanitation situation were also carried out in Venda in 1986 and 1990. In Venda, the Department of Water Affairs (established in 1986) has taken over the responsibility of water supply for the whole area. This department receives funds in the form of loans from the Development Bank of Southern Africa.

2.3.2 Existing water supply and sanitation situation

There are approximately 400 000 rural people in the Republic of Venda. Approximately 80 % of these people have easy access to potable water. The water supply situation in Venda is 60 % boreholes, 30 % communal taps from piped supplies, and 10 % wells, rivers, streams and fountains. For superstitious reasons and due to past failures, springs in Venda remain unprotected. Apparently, some of the springs which were protected in the past dried up, and therefore no protection of springs is now permitted by residents.

The principal health problems due to waterborne diseases are diarrhoea, typhoid, malaria and bilharzia, especially prevalent amongst children. No rural people, except those who own businesses like butcheries and shops, pay for their water. Arrangements are that as soon as water is reticulated into individual households, they will start to pay a consumption fee for water. As far as sanitation is concerned, no department is specifically responsible for this task in rural areas. However, the Department of Health is responsible for health education. Through the Health Care Groups, the Department motivates communities to care for their water as well as build sanitary facilities. The provision of public sanitary facilities in rural areas is the responsibility of the Department of Works. It is estimated that only some 10% of rural households have access to improved sanitation facilities.

2.4 Bophuthatswana

2.4.1 Background

The Republic of Bophuthatswana's capital is situated in the South-Western Transvaal. This homeland has populations scattered in three provinces viz Transvaal, Northern Cape and the Orange Free State. Investigations into the water supply and sanitation situation were carried out in 1986 and 1990. The Department of Water Affairs is responsible for the provision of water supplies country wide, and the Bophuthatswana Water Authority for the maintenance of the schemes.

2.4.2 Existing water and sanitation situation

Bophuthatswana has a land area of 50 000 square km and a population of 1 740 600. Approximately 76 % of the population (or some 1 300 000) live in the rural areas and almost all of these depend on basic groundwater supply schemes such as boreholes equipped with windmills, diesel pumps and handpumps. In many cases this water is relatively saline. The rural villages are not yet individually provided with piped water supplies, but all have access to either natural surface water, boreholes or installed standpipes. Some regional piped schemes are in the process of being implemented at the present time.

Most rural communities do not pay for their water. However, in some areas people contribute labour in digging trenches for water reticulation and a nominal fee to purchase diesel for the pump. However, house connections in the areas served by the new regional schemes will be charged both a connection fee and a consumption fee. In the more remote rural areas, people often travel long distances to fetch water. Due to lack of transport, spare parts and sometimes funds, maintenance of water schemes in the rural areas often takes a long time, and communities may have to travel even longer distances for water until repairs have been affected. However, since its inception in 1984 and until 1988, Drought Relief (later Thusano Foundation) had been assisting the Department in maintaining borehole equipment throughout Bophuthatswana. The Borehole Maintenance Team of the Thusano Foundation has recently been taken over by the Bophuthatswana Water Authority. Like all other homelands, Bophuthatswana has established regional water schemes to ensure the availability of potable water to the urban areas.

Rural sanitation is solely the responsibility of individual households. Therefore, no department is particularly concerned with construction of pit latrines, except in public places like schools, clinics and churches. The Thusano Foundation has helped to erect Ventilated Improved Pit (VIP) latrines at some of these institutions.

In summary it may be said that approximately 60% of the rural population have access to reasonable water supplies, and 20% to improved sanitation.

2.5 Qwaqwa

2.5.1 Background

Qwaqwa is a national state that is situated in the South-Eastern Orange Free State. There are some 300 000 rural people in Qwaqwa. The Department of Works is responsible for the provision of water supply and sanitation in the whole area. However, sanitation projects are mainly in urban and public places like schools and hospitals, and in rural areas at clinics.

2.5.2 Existing water supply and sanitation situation

Comparatively speaking, Qwaqwa is the smallest homeland in South Africa. The land area is estimated at 1500-2000 km². The total population was 366 949 in 1986 with a growth rate of 3 %. As in the Transkei and partly in KwaZulu, Qwaqwa is blessed with perennial springs, streams and rivers. Despite this source of water, almost all rural villages now have access to regional water schemes. However, there are four protected springs that cater for 1045 people. There are also five borehole schemes which were implemented between 1971 and 1975. Two of these schemes have piped reticulation and all are partly efficient. Approximately 4000 people are being catered for by these schemes. Besides springs and boreholes, there are six water tankers that began operating in 1972 and cater for 6500 people. Problems associated with tanker operation include breakdown of vehicles and lack of funds for repairs.

As far as boreholes are concerned, lack of community participation and vandalism are reasons for their partly efficient operation. Qwaqwa communities expect the government to supply water without cost following past precedents. Cost-recovery is a problem because there are no guidelines as to how to recover community water supply costs. Apparently, only those people with in-house reticulation or businesses pay for their water. Except for people who fetch water from boreholes, springs and tankers, all people travel less than 250 m to fetch water.

In Qwaqwa, water supply is not a problem and at least 90% of the people have access to a safe water supply. There are expensive regional water schemes that operate fairly well. However, cost-recovery is still a major problem. Some 20% of rural households have access to improved sanitation facilities.

2.6 Transkei

2.6.1 Background

The Republic of Transkei is situated along the North -Eastern Cape coast. Transkei was South Africa's first homeland to opt for independence in the late seventies. There are some 2 500 000 rural population in the Transkei. While the provision of water in the whole area is the responsibility of the Department of Agriculture and Forestry, small water supplies are also implemented by the Transkei Appropriate Technology Unit (TATU) and TRACOR.

2.6.2 Existing water supply and sanitation situation

The rural population of Transkei is estimated to be living in 3893 wards or villages that are scattered over an area of 43458 km^2. The general situation regarding the supply and availability of potable water to rural areas is still unsatisfactory. Only 15 % of the rural population have more than 20 litres per person per day available, 43 % have between 10 and 20 litres per capita available and the remaining 42 % have been classified as "critical".

Of the total rural population, 24 % lives within the supply area of the six regional water schemes. The rest depend on springs, wells, boreholes and river water. Since 1946 to date about 1600 boreholes fitted with windmills and handpumps have been installed. However, about 40 % of the windmills are inoperative at any given time due to lack of maintenance.

The rural populations are distributed in the wards at an average population density of 654 people per ward.

If we assume that 1600 of these wards are supplied with one fitted borehole (i.e. the number of fitted boreholes installed at 1/ward), these would have been categorized as having adequate supplies. However, with

40 % of these being inoperative at any time, this implies that approximately 418 500 of the people with "adequate" supplies actually experience shortages at any time.

Furthermore, some 902 000 people have inadequate or non-improved supplies, and an additional 873 000 are in critical need of water supplies (presently <10 l/person per day).

Hence, the situation in the rural areas is as follows:

Supplied by regional schemes		611 000
Supplied by fitted boreholes:	operating adequately	628 000
	inoperative	418 500
Non-improved but obtain > 10 l/person per day		902 000
Obtain <10 l/person per day		873 000

Thus the total number of people with inadequate or unimproved supplies is approximately 2 193 500. This represents 83 % of the rural population.

This alarming figure of 2 193 560 people led to the establishment of the Rural Initiative Water Programme (RIWP). Through RIWP, TATU has managed to implement approximately 90 protected springs all over the Transkei between 1985 and 1987. However, this is but just the tip of the iceberg.

As far as sanitation is concerned, there is no specific department that is responsible for this aspect in rural areas. Therefore, the provision of household toilets is the responsibility of individual householders. However, the Department of Education provides toilets for schools.

The Transkei government has changed their approach to rural water supplies, acknowledging that there is a need for a concept approach that involves communities at all stages of project development and the maximum utilization of all water sources. There is also a need for development of cost-recovery methods for water supply schemes in the developing areas.

2.6 Ciskei

2.6.1 Background

Ciskei is the fourth national state to opt for independence in the early eighties. This state lies south of the Transkei. In Ciskei, the Department of Public Works is responsible for water supply in the whole area.

2.6.2 Existing water supply and sanitation situation

By the mid-eighties, the Republic of Ciskei had made considerable progress towards providing the entire population with water. Further progress has been made in this field since. The Republic of Ciskei has a land area of 7760 square km and a population of 860 000 people with a growth rate of approximately 3 %. Approximately 62 % of the population live in rural villages most of which are well planned residential units for the farming communities of various agricultural areas. These rural villages are not individually provided with services such as water, sanitation, electricity or roads and storm water drainage, but all villages have access to either natural surface water, boreholes or installed standpipes. Ciskei depends heavily on groundwater resources for most of the water supplies to the rural villages. Until the beginning of 1985, 1270 boreholes had been sunk in all the regions, viz. Victoria East, Peddie, Middledrift, Hewu, Zwelitsha, Mdantsane and Keiskammahoek. Between 53 % and 74 % of all boreholes sunk are successful, the success rate being above 60 % except in the Keiskamma and Alice areas.

Ciskei has three types of water supply schemes, namely:

- Basic schemes to rural areas - these schemes involve boreholes equipped with handpumps. However, some schemes consist of a pump station extracting raw water from a river into a reservoir, a normal borehole with a pump (electricity, hand, wind or diesel driven) or a dam to ensure that water is available for the surrounding population;

- Intermediate schemes for institutions (e.g. hospitals and colleges) outside of the urban areas. These schemes consist of sedimentation tanks, slow sand or pressure filters. Aluminium sulphate is used as a coagulant, lime for pH adjustment and HTH for chlorination. Pumps are either diesel or electrically operated;

- Regional schemes that supply all of the urban areas in Ciskei. These schemes are also combined with the local schemes to supply water to the rural areas where feasible.

As in other homelands, the rural villages in Ciskei are not individually provided with sanitation. The individual households provide their own sanitation, usually by way of pit latrines. However, the provision of sanitation facilities in the urban areas and institutions in rural areas is the responsibility of the Department of Public Works.

Boreholes are the basic water supply schemes for rural areas and approximately 60 % of these are in good operation at any time. Up to 20% of rural households have access to improved sanitation.

2.7 KwaZulu

2.7.1 Background

KwaZulu is the only national state that is situated in Natal. While it is the responsibility of the Department of Agriculture and Forestry to provide water in the rural KwaZulu, there are also several parastatals and non-governmental organizations that are involved in the provision of potable water and sanitation in rural and peri-urban areas.

2.7.2 Existing water supply and sanitation situation

KwaZulu has approximately 3½ to 4 million rural population with an annual growth rate of 3 %. Most of these people have access to basic water supplies such as springs, wells, rivers/streams, boreholes, rainwater and in some cases standpipe systems. More than in any other national state, KwaZulu has drawn a wide range of interest from parastatal and non-governmental organisations in terms of water supply and sanitation projects. Therefore, there are many schemes that have been completed and some are underway to improve the quality, quantity and accessibility of water supply in the rural areas.

As of March 1988, the following water supply schemes were completed by various development organizations in KwaZulu (see Table 1 below).

TABLE 1: Number of improved rural water supply schemes completed (up to March 1988)

PROTECTED SPRINGS	TUBE AND RINGWELLS	BOREHOLES WITH HAND PUMPS	STANDPIPE SYSTEMS	RAIN WATER
2216 1106 *	125	2500	12	No data available

* Fitted with reservoirs

These improved schemes are said to cater for only 25 % of the rural population. Therefore, approximately 75 % of the rural KwaZulu population do not have access to an assured safe water supply.

The Department of Health is responsible for health education. Therefore, the village health workers advise people at grassroot level to build toilets. The health inspectors also urge schools and creches to build toilets failing which they (inspectors) are empowered to close these institutions. Generally, the provision of sanitary facilities is a responsibility of households concerned. However, various organizations have supplied about 436 prototype VIP latrines all over KwaZulu. Some community members do approach these organizations for assistance in building the VIP toilets. The provision of sanitary facilities to government institutions like magistrate offices, agricultural and other offices in rural areas is the responsibility of the Department of Works. Although the government provides subsidies for building classrooms at schools, sanitary facilities must be provided for by the parents through the school committees. It is estimated that only 10% of the rural population have access to improved sanitation facilities.

2.8 KaNgwane

KaNgwane is a national state that is situated in the Eastern Transvaal. This homeland is divided into three districts, i.e.

. Nsikazi
. Nkomazi
. Mlondozi

As in most homelands, KaNgwane's rural population has access to basic water supply such as springs, streams and boreholes. However, in some areas the communities have access to regional water schemes. In their 1990 report on groundwater quality in KaNgwane, the Water Care Services Programme of CSIR reported that there are approximately 172 boreholes either fitted with hand pumps, windmills or engine pumps. Non-government organisations have also been involved extensively in the protection of springs in the rural areas. It is estimated that to date some 75% of the rural population have access to an improved water supply. Little has been done to improve household sanitation in the region, except at schools and villages which have been earmarked for industrial development. It is therefore estimated that some 20% of the rural population have access to improved sanitation facilities.

2.9 KwaNdebele

KwaNdebele is a national state that is situated about 130 km North-East of Pretoria. The rural population depends mostly on water from boreholes and regional schemes. It is estimated that a high proportion of people (some 90%) have access to improved supplies through these schemes. Little cost recovery is practised. Improved sanitation in the rural villages is estimated as 30%.

2.10 Commercial farms in South Africa

The water supply and sanitation situation on commercial farms is being monitored by the Rural Foundation which operates in 9 regions, viz North-Western Transvaal, Eastern Transvaal, Natal, Eastern OFS, Northern OFS, Western Cape, Eastern Cape, Southern Cape and Northern Cape. The geographical distribution of membership of the Foundation is 61 % in the Cape Province, 21 % in Transvaal, 15 % in the Orange Free State and 3 % in Natal with a total population of 249 107 served in 1988/89. It is estimated that up to 70% of these families have access to an adequate water supply, but sanitation is often still rudimentary. Only some 20% of these households have improved sanitation facilities. It could be assumed that commercial farms which are not members of the rural foundation have a similar water supply and sanitation situation until more accurate figures can be obtained.

2.11 Conclusion

Notwithstanding the fact that the information reflected in this report is slightly outdated or not complete or reliable for some regions, and given the 3 % annual population growth of South Africa, it is evident enough that more than 50 % of the rural population in this country is still without access to safe, adequate and convenient water supply and more than 80% to adequate sanitation facilities. In those areas where improved water supplies have been provided, there is little cost recovery, and operation and maintenance is primarily undertaken by the relevant government departments. Therefore, more coordinated efforts should be employed to improve the situation in all developing areas of South Africa. There is also a need to establish national guidelines for cost-recovery of water supply schemes in order to achieve our goals within a reasonable budget.

3. WATER SOURCES

3.1 Hydrological cycle

Water on earth can occur as water vapour or droplets in the atmosphere, surface water in rivers, streams, ponds, lakes, seas and oceans, or ground water in the subsurface ground strata. This water is for the most part not at rest but in a state of continuous cyclic movement. This movement of water is called the hydrological cycle.

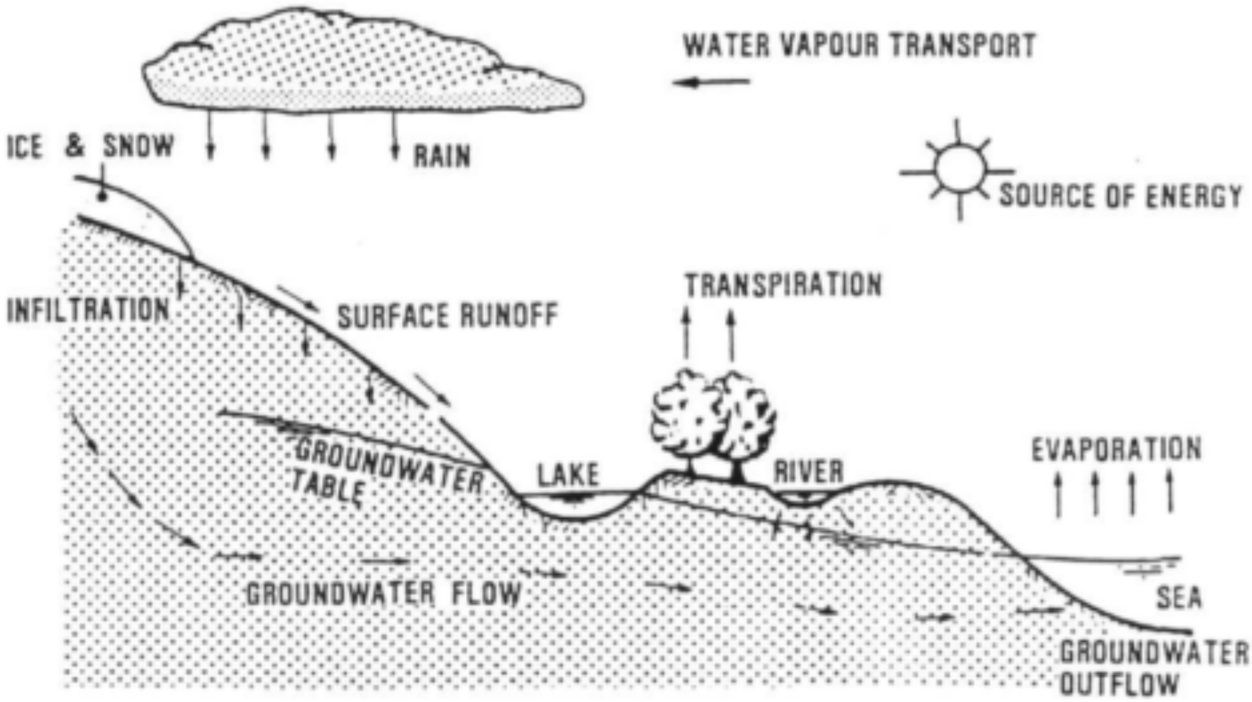


Figure 3.1: Schematic of the hydrological cycle

The driving forces behind this recycling movement are the earth's gravity and the thermal energy of the sun. Water from the atmosphere precipitates on the earth as snow, hail, rain, or condensation depending on meteorological conditions. Not all of this water adds to the surface or ground water resources, as part of it evaporates and returns directly to the atmosphere. Another portion is intercepted by the vegetation or is retained within the top soil, and is not available for exploitation for domestic use.

Part of the accumulated water flows as surface runoff towards streams, rivers, and lakes. Another portion infiltrates the ground. This water may flow either at shallow depth underneath the ground to open water courses, or it percolates further downward to reach deeper ground water strata. Neither the shallow nor the deep ground water is stagnant. It flows underground in the direction of the downward slope of the water table. This water may at some stage emerge again at the surface, either in the form of a spring, or as a ground water outflow in a river or a lake.

Further water losses due to evaporation occur from any water surface exposed to the atmosphere. This includes water in the top soil, in pools and marshes, and in streams, rivers, lakes and oceans. A portion of the ground water is also taken up by the roots of plants and re-enters the atmosphere through transpiration from the leaves.

Through technology, man has been able to obtain water from virtually all of these sources for domestic and other uses. Even in the most arid regions water can be obtained from precipitation interception (rainwater harvesting) or from groundwater sources, even though these may be very deep and very old (some groundwater reserves are estimated to be many thousands of years old).

Man has a marked influence on all aspects of the hydrological cycle. In particular man's activities often result in a deterioration of the quality of the water at any point, and in a change in the rate of flow of water at any point of the cycle (e.g. infiltration of rain into the soil). This has resulted in the pollution of and change in the flow characteristics of our surface waters, in some cases our ground waters, and even of the rain in certain parts of the country. Water collected from these sources may then be unfit for human consumption without some form of treatment to purify it. Poor farming practices in some areas have resulted in loss of vegetation and topsoil. This in turn has resulted in a reduction in infiltration and the drying up of natural springs or a lowering of the water table. In the worst cases this has resulted in the desertification of the area whereby not only is water more difficult and costly to obtain, but the soil is also unable to sustain the normal vegetation or agriculture it was able to in the past. Hence it is of vital importance to assess any proposed development in an area from a holistic, long term viewpoint, taking both the water quality and water flow characteristics into consideration.

3.2 Location of water sources

With a knowledge of the hydrological cycle, it is possible to locate all possible water sources in an area so that the optimum choice (part 1) can be made. Water for use by man can be obtained from the following sources:

rain water (as rain or snow)
surface water from streams, rivers, lakes or dams
groundwater from wells, boreholes or springs

It is important to not only locate these sources, but also to estimate how much water each source will provide on a continuous basis, especially during the dry periods. Common water sources utilised in rural areas are springs, wells and boreholes, streams, and on an individual basis rain water. The following methods can be used to locate and estimate the yield of the various sources:

rain water : rainfall records can be obtained from various weather stations by contacting the Weather Bureau, Department of Environment Affairs, Department of Agricultural Development (Soil and Irrigation Research Institute), or Department of Water Affairs; or by contacting the nearest measuring station directly. Note that not only should average rainfall data be used, but also rainfall records of the driest years to estimate the worst situation likely. Choose two or three stations closest to the region where the water supply project is to be implemented.

spring and stream water: usually the residents of the area will be able to point out all springs and streams, and to indicate whether they dry up during the dry season. The actual streamflow in the spring or stream can be measured with a portable weir or other suitable method, but this will not give a picture of the yield of the spring/stream during the year. The Department of Water Affairs can be approached to provide streamflow records of a river gauging station closest to the area of interest. By assuming similar catchment characteristics for the spring/stream as the measured river, the flow records can be adjusted to give an indication of the expected long term flow pattern which can be expected. A further adjustment to the flow records can be made by comparing the measured flow in the spring/stream with the flow at the Dept. of Water Affairs gauging station on the same day. Alternatively an adjustment may be made for differences in catchment characteristics based on experience, and using the catchment characteristics figures supplied by the Dept. of Water Affairs.

wells and boreholes : an experienced geohydrologist should be consulted to locate the most promising sites to drill for water. However, information from previous drilling in the area will usually provide valuable information on the likely location and potential of wells or boreholes.

3.3 Springs

A spring is the place where the free surface of the underground water system, known as the phreatic surface or water table, intersects the natural surface of the ground. Springs can be of a vast range of appearance, size and water yields. Some springs occurring in limestone topography are in fact the outlets of vast underground water compartments and can be quite dramatic in appearance. In South Africa the Kuruman "Eye" is such an example which discharges 650 000 to 750 000 *l* per hour of crystal clear water. The Kuruman eye and a nearby similar spring have made possible much of the development in that area.

In other situations the spring may emerge under pressure as a fountain of water. These are termed Artesian springs and may discharge hot water, or water rich in dissolved salts or sulphur. It is common to find Health Spas or resorts developed on such springs, many of which in Europe and Asia date back to the period of Roman imperialism or beyond.

In Africa springs are commonly found in rolling topography which is incised with drainage courses (streams). Springs generally occur at the heads of these drainage courses, largely irrespective of the underlying geology. Some springs occur as mere seepages from marshy areas, sponges or vleis on hill slopes. Springs from which water flows continuously throughout the year are termed perennial springs and may be as low as 1 *l*/minute and are the ones which can be used for domestic water supplies. Seasonal springs are not reliable and therefore do not warrant development or improvement, except when the spring is conveniently situated close to households in which case the community will use it when it does provide water, or when it can be used for agricultural purposes.

3.3.1 *Spring water resource management*

It must be appreciated that a spring is the visible outlet of a natural underground water system. Management and conservation of the entire system is essential if the spring is to continue functioning adequately in the future. Unfortunately this has not been appreciated throughout much of Africa. Cultivation and the planting of exotic tree species, particularly *Eucalyptus*, in the seepage area above springs has lead to the drying up of the underground water systems, and hence also of the springs. It is therefore essential to identify the seepage area and to protect this against exploitation or cultivation. The quality of the spring water can also be affected if animal-lots, pit latrines or other sources of contamination are established upstream of the eye of the spring. Latrines and other potential sources of contamination should not be permitted within 50m of the spring eye, or 100m when upstream of the spring. All surface drainage emanating from such facilities must be directed away from the spring area. In practice this is not so difficult as the seepage area can often be identified by visual inspection of the topography, and identification of various plant species associated with saturated ground conditions. The area can be fenced off, surrounded with a hedge or just left under natural bush or marsh vegetation. Gardens and trees can be safely planted downstream of the spring outlet.

The conservation of wetlands is therefore an extremely important and integral part of spring water development and management. Unfortunately in most rural areas there appears to be virtually no evidence of this being practised. When a spring does dry up, a process of desertification may begin. The community and animals that depended upon the spring for their water supply, have to seek another source, invariably putting more pressure on the new source and the community that uses it. The process is cumulative and in most cases virtually irreversible. It is therefore of vital importance that conservation of wetlands receives the priority and attention that it deserves. The community benefiting from a spring as well as the government extension workers should be taught the importance of this practice.

3.3.2 *Protection of springs*

The area immediately above and around the spring outlet (eye of the spring), or spring protection works (if it is a protected spring), should be fenced to prevent faecal matter from humans and animals entering the area. Furthermore, furrows and berms should be dug to prevent the direct ingress of contaminated surface runoff into the spring after rains.

The outlet from the spring must be accessible to the community, and hence the spring eye is usually physically enclosed and an outlet pipe installed to transmit the water to a point away from the spring. This is what is often meant by the term "spring protection", in its common but narrow meaning.

[**Note** that the practice of sinking a drum into the stream bed at the spring to act as storage for the spring water is not a form of spring protection. Usually the water in such drums becomes more contaminated than the spring or stream water as no flushing of the container takes place. Such systems should only be used when constructed as a proper well with a lid on to prevent contamination - see section 3.4]

3.3.3 *Types of spring protection*

Spring protection consists of enclosing the eye of the spring in some form of structure or drainage system, and piping the discharged water to a convenient spot, where containers can be filled. Excess water is allowed to continue flowing in the natural stream course or directed to vegetable gardens. It is usual to provide some point where cattle and other animals can have access to the water separate from the place where people collect their water.

It is often said by spring protection personnel that no two springs and hence no two spring protections are the same. This is true and is the reason why prefabricated or s andardized solutions are not generally applicable. However springs fall into three broad categories. These are:

. Open springs
. Closed springs
. Seepage fields.

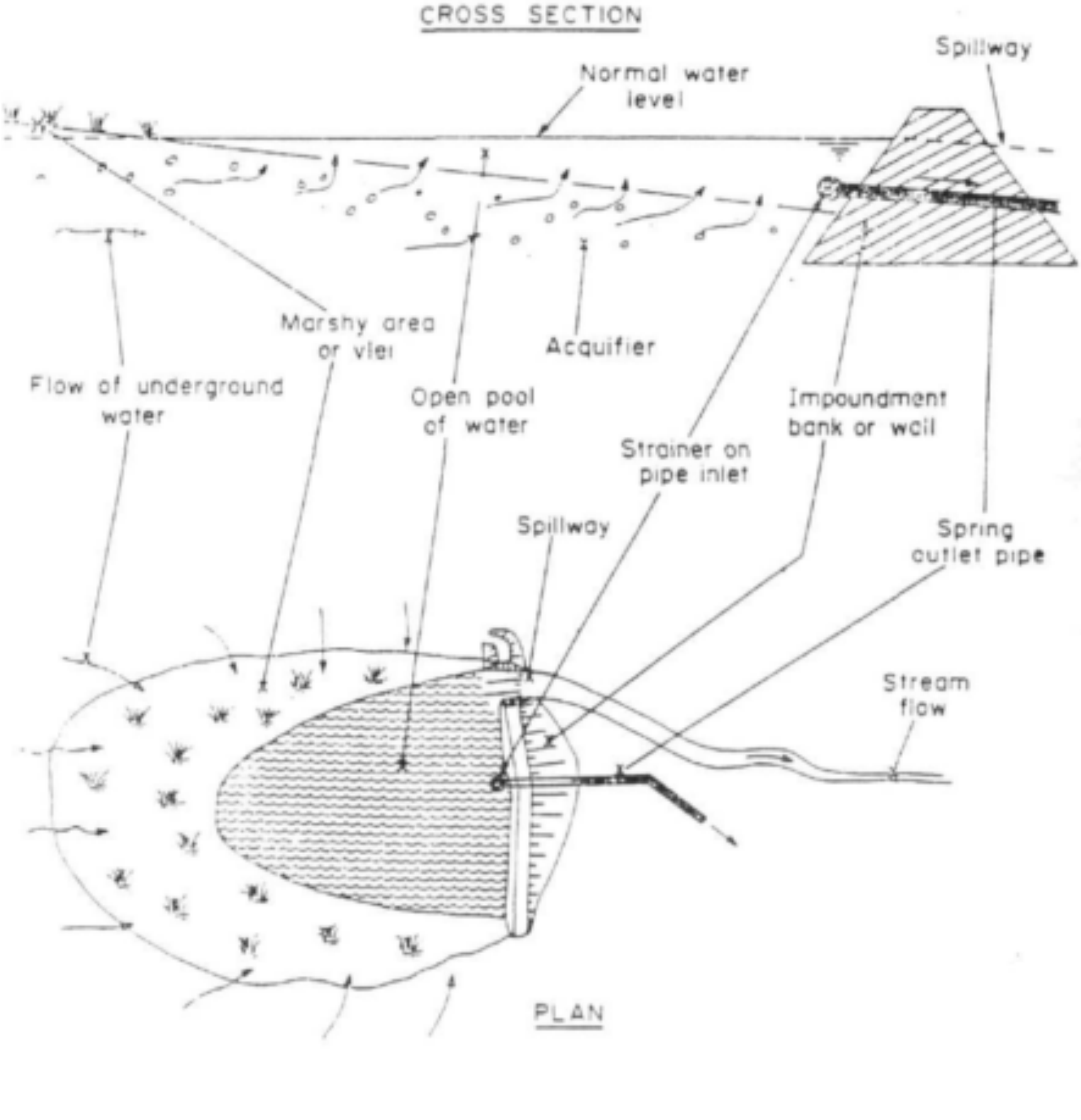


Figure 3.2: Details of a typical open spring type protection works

Open springs

In some springs such as the Kuruman Eye described previously, or where springs occur as pools in open country, it is often not feasible or necessary to close-in completely the eye of the spring. Some form of sump or central collection point from which the outlet pipe can be lead, and a suitable fence to prevent access to the pool is all that is required. When algae are found growing in the spring pool it can lead to the need to strain or filter the water. Although most algae are not harmful for human consumption *per se*, they may be removed for aesthetic reasons. A typical protected open spring is shown in Figure 3.2.

Closed spring

The more common form of spring to be found in rolling topography is where the underground water seeps out at a specific point and flows on as a small stream. In such cases a spring chamber is constructed around the spring eye so that the eye is completely enclosed to the sky (figure 3.3). The spring chamber may be provided with an access or inspection manhole, or merely closed by filling the chamber with stones, covering this with some form of plastic cover, and finally earthed over. It is preferable however, to provide some form of access manhole, which can be covered with earth but marked, in order that desilting of the spring eye, and routine maintenance and inspection of the pipe intake can be undertaken.

It should not be the function of the spring chamber, cut off wall, (sometimes referred to as the spring box or V-box) to store water, although some degree of water impoundment is desirable to promote a steady flow of water from the spring outlet pipe. Damming the spring can cause the water level to rise above the natural eye of the spring, which will result in a rise in level of the water table and the possibility of the water finding alternative outlets or eyes.

The spring chamber should form the intake to the spring outlet pipe, and should be designed in accordance with principles of underground filters. A graded filter or filter membrane should be provided between the *in situ* material of the water bearing layer (aquifer) and the outlet pipe. The outlet pipe itself may be slotted or provided with a strainer at the intake end. The chamber should be constructed with masonry, blocks, bricks or concrete and be large enough to allow adequate access for desilting and maintenance. The top of the chamber should be covered with a lid (a large flat stone or precast slab), and covered with earth to prevent children from opening it. Its position should be marked either by building up earth on top, or with a permanent marker. Being completely cut-off from the light, the growth of algae is prevented.

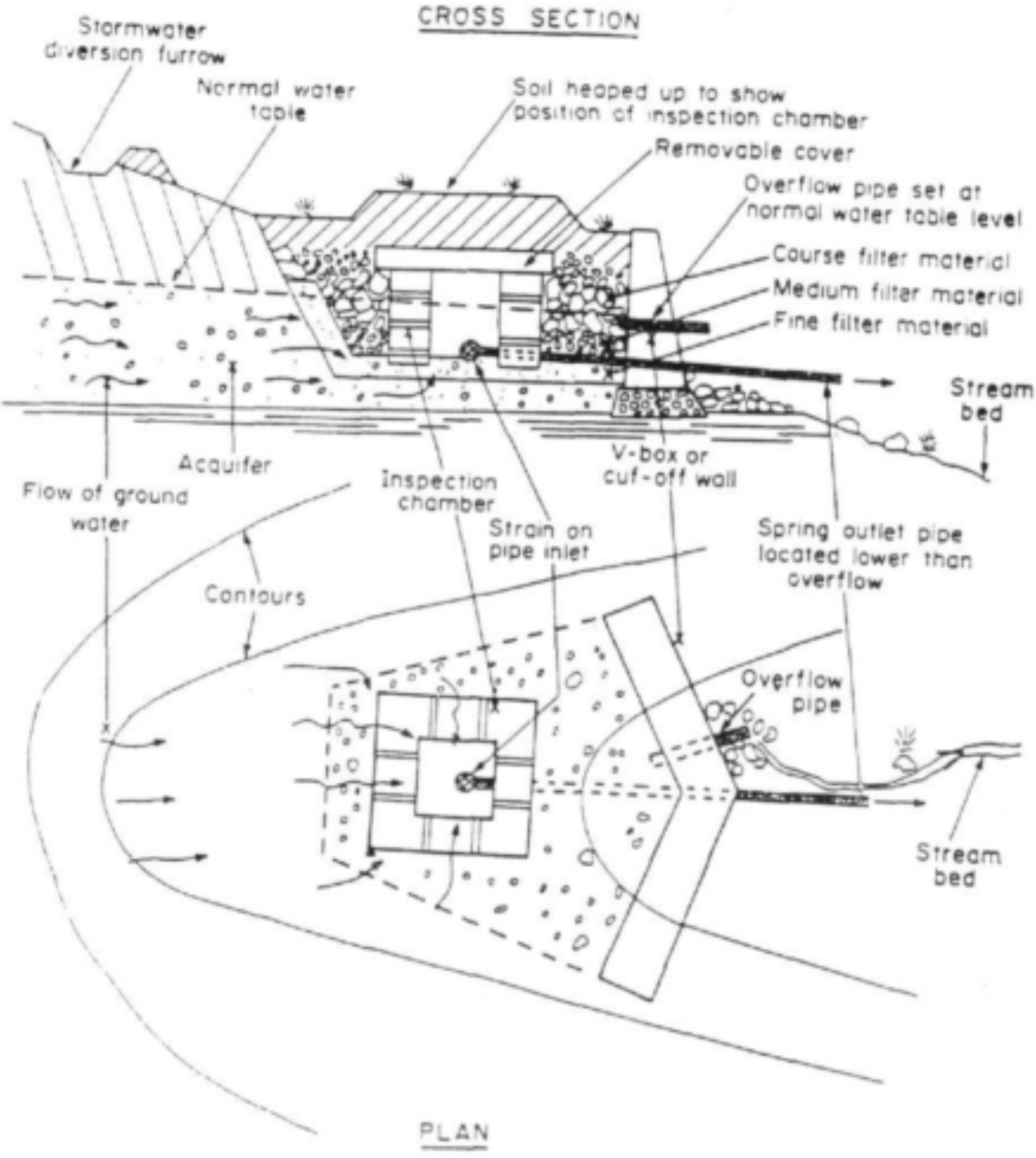


Figure 3.3: Details of a typical closed spring type protection works

Seepage field

If the spring contains several eyes or is seeping out over a large area, the simplest solution is to dig infiltration trenches to collect the underground flow of water and to concentrate it in sub-soil drains to a central collector pipe. These sub-soil drains can be made of stone, gravel, brushwood, tiles or slotted pipes (e.g. "Core drain"). Alternatively if the water is flowing at shallow depth over an impervious surface such as solid rock, it is possible to build a series of water containment walls on the rock surface with sub-soil drains located on the upstream sides to concentrate the water into a collector pipe. In both cases the sub-soil drains are backfilled with the earth dug out of the trench initially. The principles of sub-soil drainage as practised in civil and agricultural engineering apply. Figure 3.4 shows a typical seepage field spring protection.

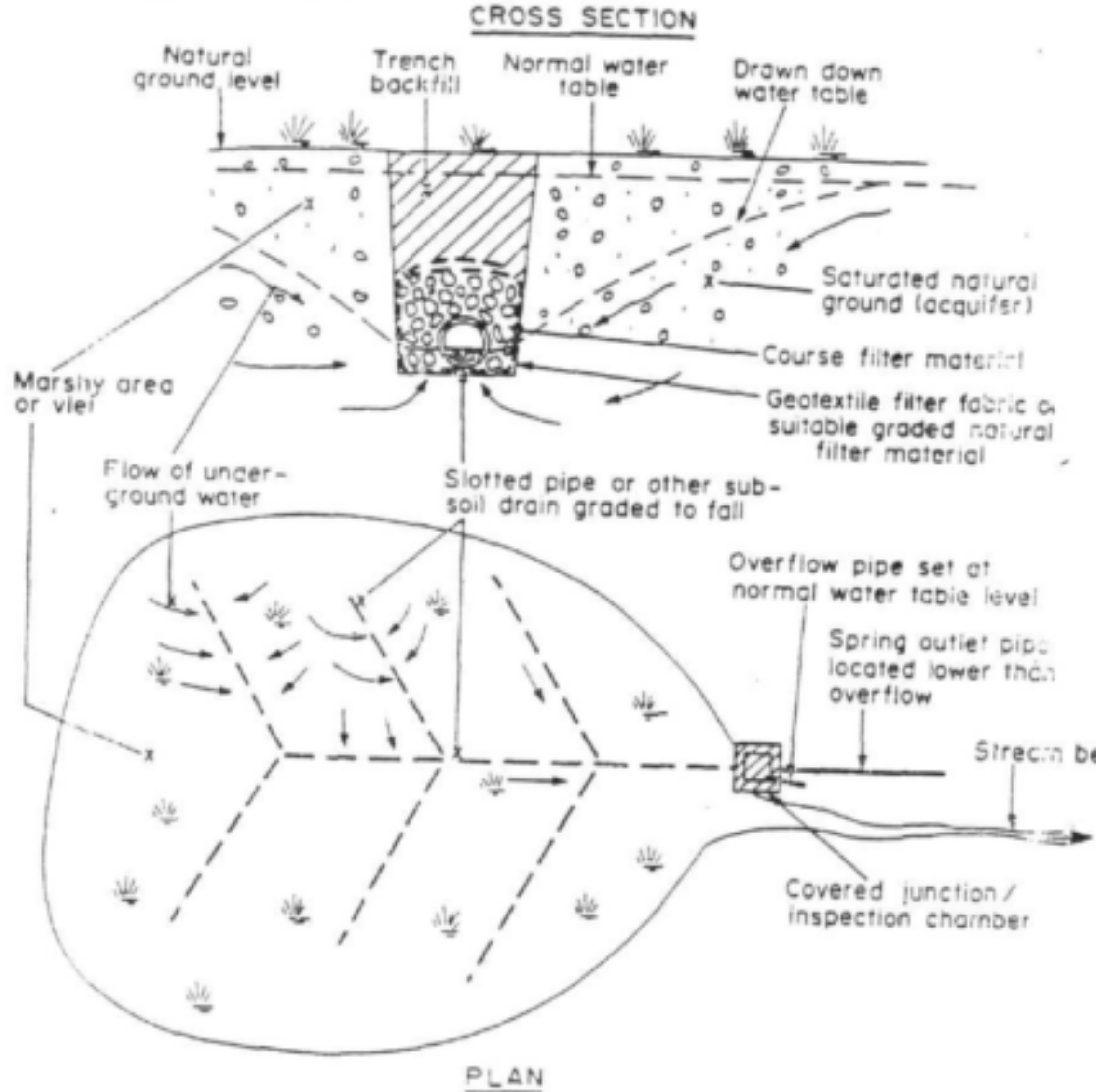


Figure 3.4: Details of typical seepage field type spring protection works

3.3.4 *Estimation of balancing storage requirements for springs*

In the case of a strong flowing spring where there is a small daily water demand, there is virtually no need to provide storage. Some queuing to fill containers may occur at peak hours in the morning or late afternoon, but this may create no problems. In the other extreme, if virtually all the water from a spring is required to satisfy the daily water demand, there is a need to catch and store the water flowing in off-peak times (particularly at night) to satisfy the demand at peak times.

To determine the required storage capacity accurately a daily water use pattern of the community is required. In practice the water consumption pattern over a daily cycle is similar to that shown in Figure 3.5.

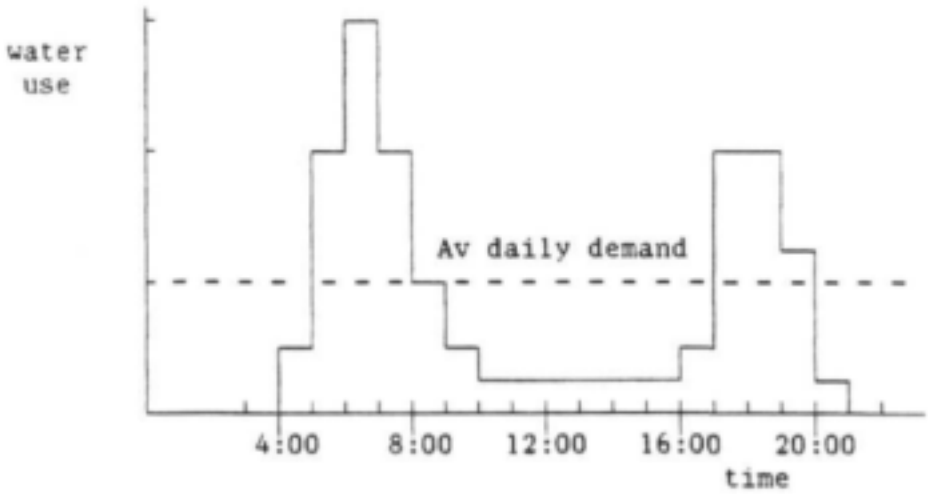


Figure 3.5: Typical daily pattern of water use in a small community

The two daily peaks occur typically between 5:00 and 7:00am and between 17:00 and 19:00. The morning peak is more pronounced and typically has a peak factor of around 3.0 (i.e. peak demand to average daily demand). As a first approximation the capacity of the storage tank should be approximately 40% of the average daily consumption.

More scientifically, the storage capacity required to meet the peak daily demand is obtained by plotting the accumulated daily water supply from the spring expected during the driest period of the year (sometimes known as the water production) and the cumulative daily water demand (water consumption) as shown in Figure 3.6.

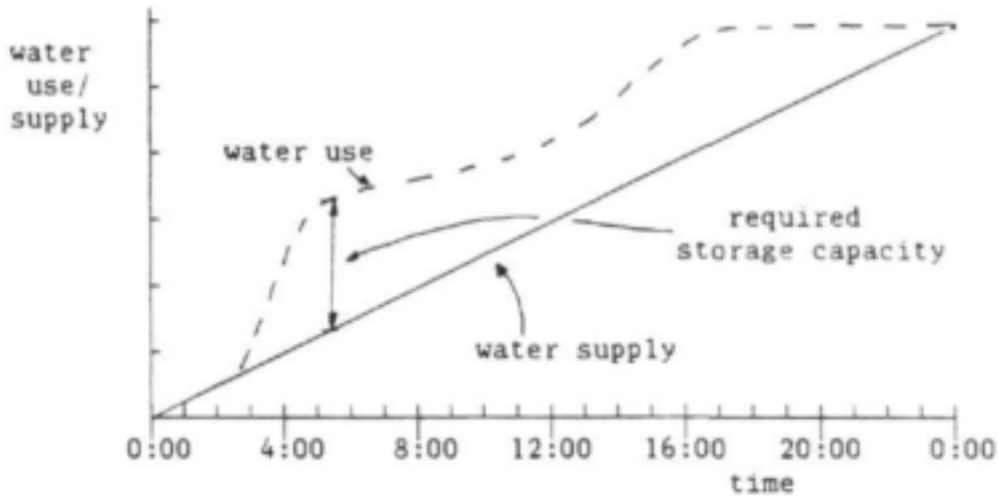


Figure 3.6 : Accumulated water production and water use

The required storage capacity is the greatest difference between the water consumption curve (dashed line) and the water production line (solid straight line). A greater storage capacity should be provided to allow for the variations likely to be experienced in practice; and to accommodate modular sizes of available storage tanks or storage tank designs. See also chapter 7 on storage and distribution.

3.3.5 *Measurement of spring flow*

In order to determine storage requirements (section 3.3.4) and the safe yield, it is necessary to know the average water flow from the spring. As there is usually a significant difference between wet season and dry season flows from springs, it is advisable that records of both wet and dry season flows are kept. For design purposes the minimum or dry season flow should be used in assessing the safe water supply of a particular spring. Wet season flows are also important for designing the outlet pipe sizes, and estimating the amount of excess water which may be available for other uses. There are several simple methods available to measure the flow from a spring.

<u>Container and watch method</u>

As it is necessary to design the water system prior to construction, some form of stream flow measurement is required. The stream can be dammed at a point lower than the spring (ensuring that the water does not back up over the spring eye), and a suitably large pipe inserted through the embankment and directed to a point lower down where it is convenient to measure the flow using a container and watch method.

A container of known volume and a stop watch or wrist watch is all that is required. The time taken to fill the container is measured at least three times consecutively and the mean value calculated. This can be expressed as a flow either in litres per minute or litres per hour.

A very simple device known as the hole-in-the-bucket flow meter, which is just that, can be used for measuring larger flows with a small container (9 litre bucket). The method is described by Sternberg et al (1982), and is reprinted in appendix B for reference.

In practice these methods may not be strait forward. Care must be taken not to lose excessive flow by seepage past the temporary embankment; and not to create a damming effect by constricting the flow with too small a pipe, or by raising the outlet end of the pipe for measuring. The pipe must not be submerged otherwise siphoning effects may give false flow rates. The flow must only be measured when a steady state has been achieved, i.e. the dam level must be constant. A weir plate with a rectangular, trapezoidal or V-notch cut away section has been found to be easier to use and more accurate for measuring small stream flows. The stop watch and bucket techniques are better suited for use to measure the flow from the outlet of an already protected spring.

<u>V-notch weirs</u>

This is a more accurate flow measuring method for small flows. A weir plate of mild steel can be hammered into a soft stream bed and bedded with mud into the banks of the stream to force the water to flow through the weir opening. The depth of water above the bottom of the weir opening is measured at a suitable distance upstream to avoid draw-down effects over the crest of the weir. For rectangular and V-notch weirs this distance should be at least twice the depth of the head on the crest of the weir (H). The bottom of the V-notch should also be at least 2H above the stream bed.

In the case of a 90° V-notch (i.e. when both sides of the notch are sloped at an angle of 45° from the vertical), the stream flow Q in litres per second is given by:

$$Q = 1368\ H^{2.5}\ (l/s)$$

where H = head above the crest of the weir (point of V) in metres.

Other flow measuring devices include Cipolletti Trapezoidal Weirs, Compound Weirs (horizontal and V-notch used for measuring very low flows and also high flows); and Parshall Flumes. These devices are described in greater detail in most text books on hydraulics and open channel flow.

3.3.6 *Pipelines survey for spring outlets*

In order to site the storage tank, standpipes, washing areas, cattle troughs, overflow pipes, etc. at the correct levels in relation to the spring outlet to ensure sufficient water flow in the pipeline and to avoid syphons and air locks developing, it is necessary to undertake a topographical survey. The most convenient way to go about this, is to stake (knock pegs into the ground) along the proposed route of the outlet pipe from the spring to its end point (reservoir storage site, and/or to standpipe positions). Once staked the pipe route can be measured using a long (30 or 100m) tape measure and a surveyor's level or theodolite. The levels (height above or below a fixed reference point) along the pipe route together with levels of the eye of the spring, intake pipe, spring overflow pipe, and the level at the site of the reservoir and/or stand pipes should be determined in the field and then plotted on a drawing. From the drawing any adjustments which may be required to ensure that the system operates efficiently can be assessed and the water supply designed. It is important that the following conditions be adhered to:

- the spring outlet chamber must be continuously drained;
- potential air locks in the pipeline must be avoided;
- syphons are avoided;
- troughs in the pipeline are avoided if possible;
- if a reservoir is planned, outlet taps or stand pipes are positioned such that the reservoir can be drained if necessary;
- the pipeline size and slope must be such that water can be drawn from all water points simultaneously.

If the topography is sufficiently steep so that the pipe route and levels can be reasonably judged in the field, and if there is only one outlet on any particular pipeline such that the above conditions will be met, then merely marking the pipe route and measuring with a tape is all that is required for the pipeline survey. Where more than one tap outlet is provided on a pipeline, it will be necessary to obtain a more accurate survey to ensure that all taps operate under a positive pressure at all times (i.e. provide water even when taps lower down are opened). See also section 7.6 on storage and distribution.

3.3.7 *Location of storage reservoir and inlet pipe*

The end of the inlet to the storage reservoir should be located slightly above the top water level (TWL) in the tank, which must be below the eye of the spring. The TWL will be controlled by the position of the overflow pipe in the tank. The reason for having the inlet above the outlet is so that it is possible to hear whether water is flowing into the tank even when it is full. Bottom inlets are sometimes preferred to minimise the trough in the pipeline feeding the tank which may block with accumulated sediments. It is important to ensure that the water in the pipeline feeding the reservoir cannot back up to the spring and drown the eye. The spring must always be freely drained.

The greater the difference in levels (height) between the highest water level in the storage tank and the level of the eye of the spring, the greater the flow of water for any given size pipe. This means that a smaller diameter and hence less costly pipe may be used in the case of a large height difference. Sizing the pipe is dealt with in section 7.6.

It is important to ensure that the storage tank is located on firm ground away from the natural water course. The choice by the community as to the location of taps for water collection may also affect the location of the storage tank.

3.3.8 *Reservoir and spring overflows*

In most situations where the storage reservoir is to be located adjacent to the spring an overflow pipe will be provided in the storage tank. The overflow water should be directed back into the natural water course, or it may be channelled into vegetable gardens or a livestock watering trough first. Clothes washing areas can be provided with water from the overflow pipe, or these can be separately supplied from the storage reservoir by means of suitable valves, pipelines and taps. The outlet of the overflow should always be at least 5 m away from the storage reservoir foundations, with flow directed away from the tank.

Where the storage reservoir is to be located at some distance from the spring, say above a group of houses, it is desirable that a separate overflow be located at the site of the spring. The spring overflow will ensure that any pipe blockages do not cause a flooding of the spring eye, and ensure that excess water is put back into the natural water course to prevent undue ecological upset. The overflow pipe should be installed at a level just above the normal level of water in the spring chamber. This overflow pipe should be cast into the spring box or V-box wall, and is used to convey the overflow water back into the natural water course. In such cases the inlet pipe to the storage reservoir may be equipped with a ball valve which closes when the reservoir is full.

3.3.9 *Spring construction details*

As stated previously, there is no standard design for spring protection works. Each spring is unique. There are, however, a few common principles that should be followed in spring protection.

These principles relate to the objectives of spring protection, which are:

- To prevent pollution of the spring water by humans, animals, insects and surface water run-off. The spring should therefore be isolated from all possible human, animal and insect contact.

- To enhance the filtration capacity of the spring outlet by removing the mud or silt overburden, and by concentrating as much natural seepage as possible into the spring delivery pipe. This leads to one of the most important aspects of the spring protection works - the development and protection of the spring outlet itself.

- To make it more convenient and hygienic to fill water containers, without the possibility of one person's container being able to contaminate the whole water source. The water is therefore usually led to a closed storage tank with a tap outlet or a series of standpipes.

Many of the design aspects are related to the need to protect the spring outlet itself. Some design aspects related to the protection of closed springs and seepage fields are listed below.

■ It should never be the intention to use the spring box or spring chamber to impound or store water. A separate reservoir must be provided if water from the spring is to be stored. The upper part of the V-box or spring wall does not therefore need to be completely watertight.

■ Since the water bearing strata (aquifer) and the eye of the spring constitute a natural filter, the basic principles of filters should be employed to design and construct the spring chamber. In the case of a closed spring or a seepage field the following steps should be included in the design:

  - A graded filter consisting of layers of successively larger particle sizes should be provided between the eye of the spring and the pipe intake.

  - The sizing of the successive layers of sand, gravel or stone or synthetic geo-textiles must be designed to prevent migration of smaller particles into the spaces of the larger particle layers and/or openings in the pipe intake.

  - Fine silt or clay particles in the underground water should be removed by artificial filter layers as far as possible to ensure clean water. The more silt and clay that is removed by the artificial layers, however, the quicker the filter will clog. It is therefore important that access to the spring chamber be facilitated for cleaning of the filter layers as and when necessary.

- In practice the V-box, spring intake box and/or spring chamber is made from bricks, blocks, stone masonry, wooden planks, or cast in-situ concrete. Some experimentation has been carried out using pre-cast concrete fencing panels for constructing the spring chamber (Alcock, 1988).

- In the early days of spring protection in South Africa, it was common to completely bury and grass over the spring chamber. However, subsequent experience with clogged filters after periods of increased rainfall, has highlighted the need for providing access to the spring intake for purposes of desilting and cleaning the artificial (man-made) filters and pipe intake. The spring chamber therefore should be constructed suitably large to allow easy access (600 x 450 mm minimum). The height of this chamber should be raised above the top of the finished level of the spring by 300 mm to 400 mm. It should be covered with a suitable lid (precast concrete or large flat stone) and covered over with soil.

- The volume behind the V-box, spring wall or retaining wall through which the outlet pipe (and overflow pipe if necessary) pass, which is not utilised for the construction of a graded filter, is normally filled with large stones or boulders. If the spring is to be fully enclosed these stones must be covered to prevent the downward migration of the covering soil into the intake chamber. This can be done with successive layers of smaller particle sizes (stones, gravel, sand); or with geo-textile fabric, or with an impervious plastic sheet. This filter or impervious covering is then finished off with a layer of top soil and grassed. When the top soil is replaced, it can be formed into a mound around and over the cover slab to the access chamber, thereby marking the position of the access chamber and at the same time preventing inquisitive children from removing the cover slab.

- Finally a diversion ditch or furrow is dug above the whole spring protection works to divert storm run-off from entering the spring chamber directly, thereby short-circuiting the natural drainage path through the soil. The whole seepage area above the spring should be fenced or marked off to complete the spring protection (see Section 3.3.2).

3.3.10 *Location of standpipes*

Where storage facilities are provided with the spring protection works, the location of standpipes must be such that the tap is below the bottom of the storage tank. If this is not the case, the bottom of the tank becomes 'dead storage' and cannot be emptied completely for cleaning and maintenance purposes. Standpipes must be robust and of such a height as to permit easy filling of all containers in everyday use. The area underneath and immediately surrounding the tap should be stone pitched and adequately drained to prevent muddy conditions from developing.

Where more than one standpipe is connected to the outlet pipe from a spring or spring water storage tank, care must be taken in the design to ensure that taps located on higher ground are not starved of water when lower taps are opened.

3.3.11 *Treatment of spring water*

Water from protected springs is normally of a good quality, with minimum contamination by bacteria or fine silt. As a result it may not need any form of treatment. However, before and after protecting a spring it is advisable to have a sample of water tested for bacteriological quality. When contamination is found, it may be necessary to provide some form of disinfection system - section 5.2.

Provided the water tastes, smells, looks good and is acceptable to the users, it is probably not necessary to have chemical, turbidity and colour tests performed. If there is any doubt as to the quality or chemical constituents of the water, samples (usually 2 litres) should be taken for laboratory analysis.

If any trace of faecal pollution, as indicated by the presence of *Escherichia coli* bacteria in the sample, is found in the water supply, it will be necessary to chlorinate the water source (the intake chamber or the storage tank) on a regular basis, until bacteriological tests on successive samples prove negative. The cause of the faecal pollution should be investigated and steps taken to prevent further contamination.

Generally spring water is crystal clear. However, after heavy rains, the water might become turbid (cloudy) due to the increased flow of fine silt or clay particles into the aquifer. Some settlement of silt may take place in the storage tank. If this occurs, it might be necessary after some time to empty the tank and scour out the bottom prior to refilling. The flow rate from the spring delivery pipe should be checked. If it has diminished, it could indicate clogging in the spring chamber. This can be remedied by opening up the spring chamber and unclogging the filter.

Slightly turbid water is not harmful of its own. If the water becomes more turbid than about 20 NTU's even under normal conditions, the filter within the spring chamber is not working satisfactorily. Where the soil conditions are such that the graded filter within the spring chamber is insufficient to cope with the fine silt load, it may be necessary to make provision for additional filtering of the water.

3.3.12 *Spring management and maintenance*

As stated previously a successful spring fed water supply system comprises the physical spring protection works, the natural water seepage system of the earth, the community and their animals, and in some instances a government department or development agency.

In order for the system to function properly on a long term basis, a management structure must be instituted within the community and a management/maintenance programme implemented.

It has been found by the experience of such organizations as the Valley Trust, World Vision and ACAT for example, that the formation of spring committees, spring associations or spring savings clubs are an essential pre-requisite for successful management of the water supply. Such public bodies should be formed prior to commencing work on the spring. The functions of the spring committee or association are to firstly motivate people to take an active interest in an important facility that is for their benefit. A second function is to organise contributions from the community whether in cash, materials or labour. In order to meet maintenance requirements as they arise, or preferably to implement a programme of preventative maintenance, it will be necessary to appoint a maintenance person or caretaker. It is furthermore advisable to start a fund in the form of an account at a local bank or post office, in order to have sufficient funds to cover maintenance costs (including payments to the caretaker) as they arise. The exact amount and nature of the fund should be decided upon by the local spring committee or association in consultation with its members.

In Natal/KwaZulu amounts of between R1 and R10 per household (per kitchen) per year are levied as contributions for protected springs (1988). All households that normally use a spring must be members of the spring association or club. Contributions should not be sought or collected from people who do not use the spring, unless there is an umbrella association or authority responsible for ensuring every household within a specified area has access to a protected water source. In such cases, the management of the springs and maintenance will be done by the relevant committee or authority at a district level in conjunction with the local committees.

The importance of managing the natural water resources cannot be over stressed. The destruction of natural seepages and excessive removal of indigenous bush will seriously effect the ability of the springs to recharge and produce a reliable, safe source of water. Conservation and sound integrated agricultural and firewood management will be essential if successful management of the natural water resources is to be achieved.

3.4 Hand dug wells

Where the underground water does not emerge above the natural surface of the ground, but is nevertheless within a reasonable depth of the surface (say within 15 metres), this water can still be accessed by sinking a well. A well is simply a shaft which is excavated vertically to a suitable depth below the free standing surface of the underground water (water table). As the well is normally dug by people using hand tools, it is often referred to as a hand dug well. It is necessary to provide some form of lining to prevent of the walls of the shaft from collapsing, both during and after construction. Shoring of the hole during excavation is required if the sides tend to collapse as with sandy soils, and in all cases when the hole is more than 1.8m deep.

When excavating in soft or collapsible soil, it is usually impractical to batter the sides of the excavation sufficiently to prevent the risk of collapse. In such situations the use of concrete rings for lining the well is recommended as the rings can be used to support the excavation while still digging the hole. The approach is to lower the first concrete ring into the excavation and to excavate inside this ring, which will then sink under its own weight. Subsequent rings are placed on top of the first ring forming a tube of rings which is allowed to sink as the soil is excavated from within it. The bottom of the first ring can be fitted with a cutting edge, in which case it is called a cutting ring, to facilitate its sinking.

There are many types of well linings. These include:

- reinforced concrete rings (caissons)
- curved concrete blocks
- masonry (bricks, blocks or stone)
- cast *in situ* ferro-cement
- curved galvanised iron sections
- wicker work (saplings, reeds, bamboo etc.)

Once the water table is reached it will become necessary to bail or pump water out of the well, in order that the excavation can continue. Note that if a petrol or diesel pump is used the engine must not be placed inside the hole, as this will consume the oxygen and emit toxic fumes which are heavier than air. The build up of fumes could cause asphyxiation of the workers in the hole. The well must be sunk sufficiently deep below the free standing surface of the groundwater to provide adequate water storage, to increase the infiltration capacity into the well, and to accommodate seasonal fluctuations in the depth of the water table. A minimum depth of 1.5 m below the water table is recommended.

It is advisable to cover the bottom of the well with a gravel layer to prevent silt from being stirred up as the water percolates upwards into the well, or when the water is disturbed by the bucket or pump used for abstraction. The well lining is then built up to the ground surface and extended to at least 1 metre above the ground. This is to prevent dirty surface water from running down into the well. The excavated gap between the casing and the soil is then backfilled and thoroughly compacted around the outside of the casing to prevent subsidence.

Joints between rings are sealed with mortar or bitumen above the water table, but below the water table they are normally left open to facilitate infiltration of water into the well. Sometimes the bottom rings are constructed using no-fines concrete to facilitate infiltration of ground water. To finish the well properly the ground around the well should be mounded-up and paved with a concrete apron so that water drains away from the well. The well should be covered with a suitable slab or covering and equipped with a suitable pump or bucket and raising mechanism.

3.4.1 *Concrete rings*

If precast concrete rings are to be used it is essential to have a good mould. The best material for a concrete mould is mild steel, preformed to the desired inside and outside diameters of the rings. The mould is normally bolted together in two or more sections to allow it to be stripped once the concrete has been cast. A steel mould would have the following typical dimensions:

Inside diameter	- 1 metre
Wall thickness	- 50 mm
Height of ring	- 600 mm

The rings must be reinforced with steel wire, pig mesh, welded mesh or reinforcing steel. A good concrete mix for reinforced concrete rings would be a 1:2:4 ratio (1 cement, 2 sand, 4 stone). If stone is not available a 1:4 or 1:3 mortar mix (1 cement, 3 sand) would be preferred depending upon sand suitability. The concrete must be thoroughly tamped into the mould and damp cured for at least five days.

Rings can be fitted with lifting rings or have keys cast into their upper and lower edges to facilitate interlocking. Rings must be accurately made and the thickness should not vary by more than about 10 % around the circumference. Alternatively pre-cast concrete manhole sections can be purchased from suppliers.

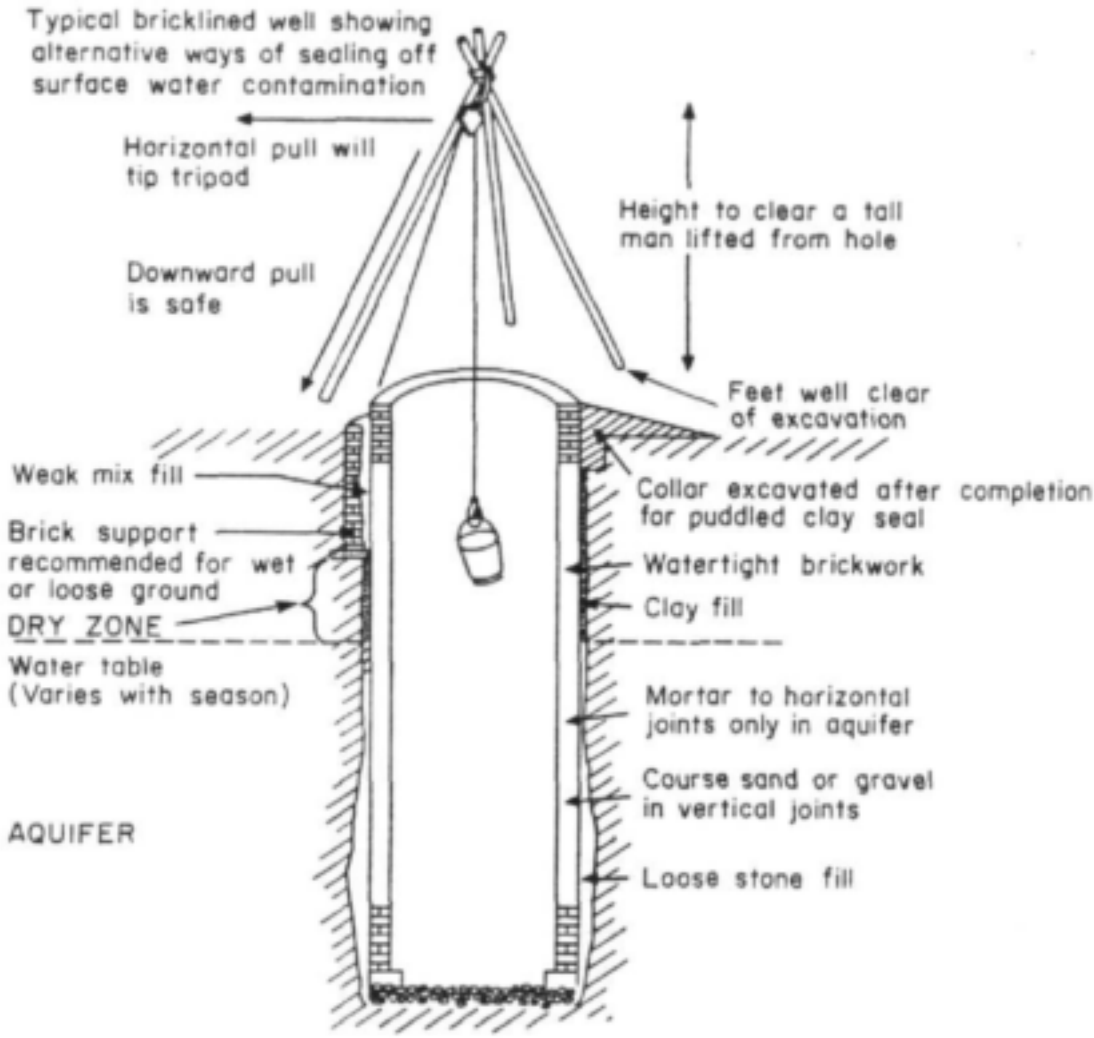


Figure 3.7 : Cross section of a typical hand dug well

3.4.2 *Curved concrete blocks*

Curved concrete interlocking blocks or panels can be used in conjunction with a cutting ring, or to line the upper part of a concrete ring well. There are not many manufacturers of interlocking curved block moulds at present (Parry Associates in U.K. manufacture and market a vibrator and mould accessory pack suitable for 2 m and 3 m diameter tanks). Depending upon available aggregates and sands, similar concrete mixes to those specified above for concrete rings should be used. The same comments made above with regard to curing and tamping of concrete rings apply equally to concrete blocks. When using concrete blocks it would be advisable to key successive courses of blocks to prevent horizontal shearing of the well shaft. Hoop reinforcing in the form of galvanized mild steel wire should be laid every third or fourth course of blocks to give added strength to the well lining.

For more information on hand-dug wells the reader is referred to:

Technical Brief No. 4, Waterlines, vol. 3, no. 4, April 1985

FEIST, I (1987) Training in well construction, Waterlines, vol. 6, no. 2, pp 28-30, IT Publications: London

WATT, S B and WOOD, W E (1977). Hand dug wells and their construction. Intermediate Technology Publications Ltd, London. 234 p

TRIETSCH, R (1984) Shallow wells for low cost water supply in Tanzania, Waterlines, vol. 3, no. 1, pp 10-14, IT Publications: London

Variations on the hand dug well include increasing the infiltration capacity of the well by boring horizontal small diameter shafts into the water bearing strata and installing slotted steel pipes into these shafts. This technique was developed by Dr. H Fehlmann of Switzerland and is known as a "Fehlmann Well". The technique is a variation on the ancient techniques developed about 3000 years ago in the middle East and Iran known as the Falaj or Qanat systems respectively, where horizontal galleries are manually driven from vertical shafts into water-bearing strata. These infiltration galleries can extend over several kilometres and are distinguished by vertical shafts surrounded by rings of an excavated material at intervals on the land surface. Refer also to Sutton (1984), Birks (1984) and National Academy of Sciences (1974).

3.5 Tube wells

Hand digging of wells is time consuming and expensive; and since non-cohesive sands tend to collapse, large volumes of sand are sometimes excavated unnecessarily, when the hole is unlined, in order to go down a short distance. For this reason cheaper methods of hand drilling, jetting or by use of an auger small diameter holes have been developed. These small diameter holes (50 to 500 mm), which are too small for a person to enter, are lined using either uPVC or mild steel casings to prevent collapse of the hole. The bottom section of the hole which is below the water table, is fitted with some form of a well screen to allow the infiltration of water but to prevent the ingress of silt and sand into the well. Because of the small diameter compared to conventional wells, these relatively shallow wells (usually less than 25 metres) are termed tube wells. They represent a technology intermediate between hand dug wells and boreholes, the later requiring sophisticated drilling machinery.

Whereas hand dug wells require simple tools (modified spades, picks and buckets), tube wells require more specialised equipment. The manually-operated 'Vonder Rig' developed by the Blair Research Laboratory in Zimbabwe and employed successfully in Maputoland (Northern Natal) by the University of Zululand is an example. (See Figure 3.8).

Figure 3.8 : The "Vonder Rig" hand operated auger drilling rig

3.5.1 *Hand drilling or boring*

Hand drilling or boring is achieved by excavating soil from the bottom of the hole by the rotation of a cylindrical tool with one or more cutting edges. Several types of augers have been designed for different soil types. A variety of these are shown in Figures 3.9 and 3.10. Excavated soil is normally contained in the body of the auger or bit and must be removed by withdrawing the auger or bit from the hole and emptying it. This is a time-consuming operation which must be continuously repeated, and consists of pulling the auger and the extension rods, to which it is attached, to the surface after each drilling cycle.

Winches, hoists and pulleys are used to raise and lower the drilling assembly. When the bit is in place at the bottom of the hole, a cross-piece, handle or tiller is clamped onto the extension rods. The cross-piece can then be turned and the bit or auger forced into the soil manually. See figure 3.10. In sandy soil a casing of semi-rigid plastic or mild steel must be driven down the hole during the drilling operation to prevent collapse of the hole. Care must be taken to prevent damage to the casing by the bit or snagging of the bit on the underside of the casing. Special equipment may be required to drive the casing down if it does not sink easily.

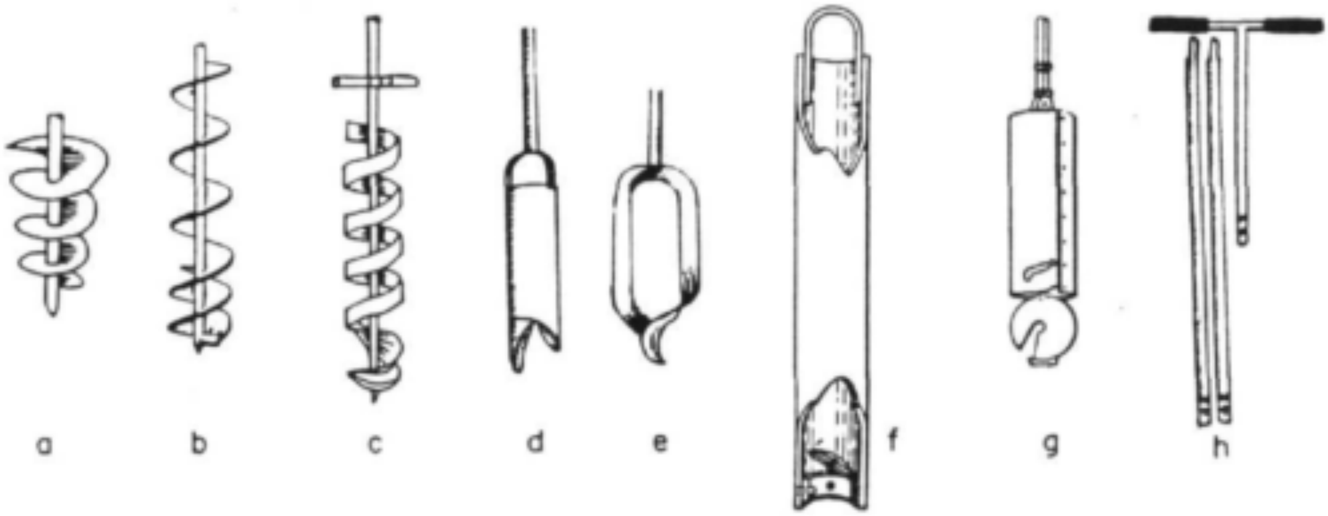


Figure 3.9 : Bits for drilling tubewells
(a) spiral auger, (b) helical auger, (c) continuous flight auger, (d) riverside bit, (e) combination bit, (f) locally fabricated bailer, (g) cylindrical bucket auger, (h) handle and extension rods

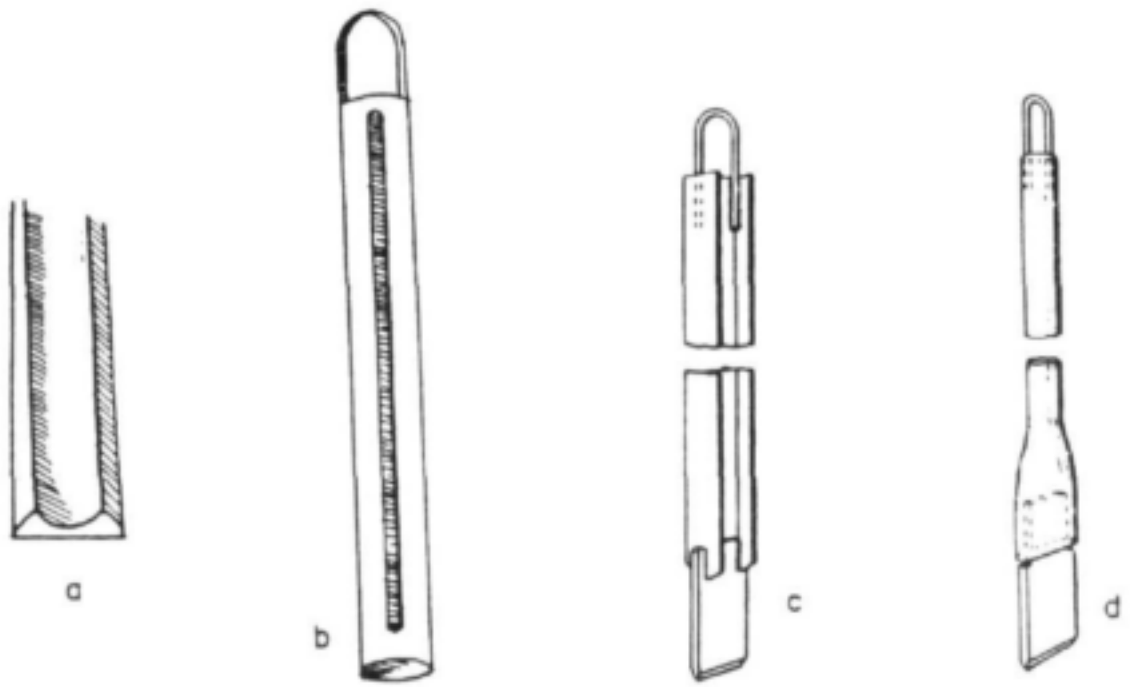


Figure 3.10 : Bits for constructing wells by percussion
(a) club-type chisel bit, (b) hollow rod bit with slot, (c)&(d) home-made percussion bits

3.5.2 *Percussion driving*

In this method of sinking a tube well, specially designed tools are dropped onto the bottom of the hole (usually from a height of about 0,5 metres) to penetrate or break the formation (soil or rock). Percussion tools can either be attached by extension rods to a tiller and employed in a similar manner to that described in section above; or in softer soils they can be attached by cable to a winch (motorized or manually operated). Under saturated ground conditions a cylindrical bailer designed with a simple flap valve at the bottom end, is used to remove sand, slurry or loose cuttings. In addition a variety of fishing tools are used (See Figure 3.11) to remove bits, cables and other equipment which may fall down the hole. In practice a variety of bits will be found suitable for each particular geological formation and a combination of drilling techniques used based on experience.

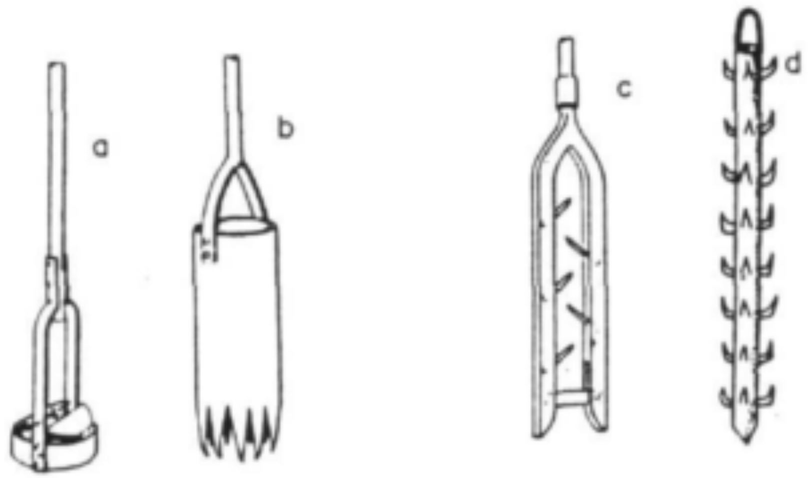


Figure 3.11 Fishing tools for removing equipment down the hole

3.5.3 *Jetting*

This method employs the use of a high pressure pump pumping water through a specially designed nozzle and cutting bit. A simple method is described by Metianu (1982).

The method described is best employed to sink a 60 mm diameter slotted liner to a maximum depth of about 12 metres in a soil free from rocks and hard layers or thick clay.

Jetting, rotary and percussion drilling are all methods commonly used in the sinking of conventional boreholes, and is covered in more detail under that heading. The simpler manual techniques of tubewell construction will normally be a combination of drilling, boring, percussion driving and bailing.

3.5.4 *Well screens*

In order to facilitate infiltration of water into the lower part of the tube well, a well screen should be installed. The casing is sometimes withdrawn slightly to increase the length of screen which is in contact with the aquifer. The size of the screen openings should be designed to be compatible with the particle sizes of the water bearing stratum (aquifer). The screen should ideally allow the free passage of underground water, while retaining the material of the aquifer without clogging. In practice gravel packing is sometimes placed on the outside of the well screen to increase its effective surface area and hence infiltration capacity. Surging and jetting techniques are employed to 'develop' a hole by removing fine particles from the soil around the screen which would tend to clog the well screen.

A simple well screen could consist of a perforated or slotted pipe wrapped in a layer of geo-textile, or alternatively a slotted brass or stainless steel screen.

Another type of well screen consists of winding a wire with a trapezoidal cross-section around a set of longitudinal rods and welding the intersections. This type of screen has the advantages of a high percentage of open area and less likelihood of clogging due to the shape of the slots formed (narrow outside and wider inside). A well screen developed from coin string and bamboo strips is described in Waterlines Technical Brief No. 5 (1985), Vol 4 (1). There are numerous types of commercially available well screens suitable for installation on boreholes. These are manufactured from stainless steel (see Figure 3.12), brass, nylon or uPVC. They are usually relatively expensive when compared to the total cost of tube wells and are not therefore commonly used with tube wells.

Well screens should be inserted inside the tube well casing, as driving them at the bottom of the casing would usually damage them. Where they are driven integrally with the casing they are termed 'well points' and are manufactured with a pointed penetrating tip and are of a robust construction to withstand being driven by a percussion tool.

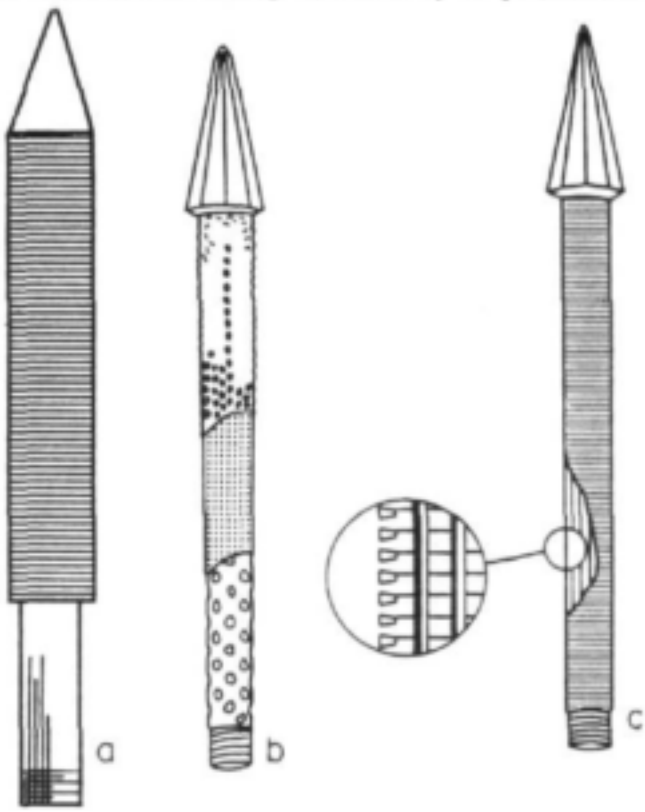


Figure 3.12 : Well points with driving penetrating tips
(a) continuous slot drive point, (b) perforated brass sheet screen, (c) screen made with wire of trapezoidal cross section

3.6 Boreholes

The study of underground water and the relationship between that water and the strata of the Earth is known as hydro-geology. It is a vast and fascinating subject and beyond the scope of these guidelines. Underground water supplies have been exploited for thousands of years, particularly in arid areas. The ancient city of Nineveh when constructed in *circa* 800 BC was supplied through an underground conduit known as the tunnel of Negoub. These ancient systems were constructed by hand and gave rise to a specialist class of people within those societies, whose duty it was to construct and maintain these underground water systems, many of them at depths of 30 metres or deeper. In more recent times the use of mechanical equipment has tended to displace these older technologies in the quest to tap deep (say greater than 15 metres) underground water sources. Because a relatively small diameter hole is bored as opposed to hand-dug, these holes or shafts are termed boreholes.

In many areas of Africa water can be found within 30 to 100 metres of the surface. A distinction needs to be drawn between the shallow water table, which is often associated with springs, seepages, wells and tube wells, and the deeper water bearing strata or aquifers. The former is sometimes referred to as a perched water table and normally subject to seasonal fluctuations, while the latter is often termed the static water table, which can be a confined aquifer on unconfined aquifer depending upon the overlying geological formations. In most instances it will be from the latter type of aquifer that deep boreholes will abstract water. There may be very little connection or movement of water between different aquifers, located at different depths within the same borehole. Hence water from the different aquifers can have very different qualities (temperature, dissolved salt, organic and bacteriological). Surface indications of the presence or absence of a shallow water table, will not give information of the presence of deeper underground water sources. However, surface geology (the direction of dipping of rock outcrops, and the presence of dykes and faults) can give invaluable information to the experienced hydrogeologist, water engineer or driller. By far the most useful information, however, comes from the records and drilling logs of previous drilling attempts in the same area, and for this reason it is normal practice for a central or regional register of all boreholes to be kept with the relevant government department in which the borehole number, location, depth, diameter, record of geological strata, safe pumping yield and other relevant data are kept. In South Africa logs for selected water control areas are registered with the Geohydrology Division of the Department of Water Affairs, Private Bag X313, Pretoria, 0001.

3.6.1 *Geophysical surveys*

The presence, amount and depth of underground water cannot normally be accurately predicted beforehand, although some of the geophysical techniques are able to give a reasonable estimation of these parameters. If a borehole drilling programme is being planned for a specific area, and no borehole data is available, it is advisable to conduct a geophysical survey beforehand. Geophysical methods can be classified into two groups: downhole methods, in which a preparatory borehole must be drilled in order to gain information of the underground hydrogeology; and surface methods. Downhole methods which are sometimes used include the following:

use of resistivity	-	lower resistivity indicates water bearing strata
self-potential	-	measurement of the natural electric field induced by the earth
inductance	-	induce an electric field and measure the decay with time
magnetic	-	measurement of the variations in the earth's magnetic field
visual (TV)	-	inspection of the hole for clues to the pattern of water and water flow

All of these techniques are highly specialised and are not used by engineers or even many hydrogeologists in general. However, when used in combination these techniques can provide a great deal of information on the condition of the underground geology and water.

Surface methods are more commonly used and include the use of:

resistivity	-	measurement of the resistance to electrical flow
seismic	-	measurement of the reflection and refraction of shock waves
electromagnetic	-	measurement of the distortion of an electric field that is established and then switched off
frequency	-	measurement of the change in frequency of induced electromagnetic wave
gravity	-	measurement of local variations in gravity
magnetic	-	measurement of local variations in the earth's magnetic field

Surface methods, in combination with geological mapping and the study of aerial photographs, are perhaps the most commonly used methods for assessing the presence of suitable underground water when no other information exists.

3.6.2 *Mathematical models*

In order to exploit the underground water supplies in a rational fashion, without adversely affecting the natural equilibrium of the aquifer either in respect of quantity or quality, it is necessary to have a model to represent the important and controlling parameters of underground water movement. Several mathematical models have been found to be successful, and a number have been evaluated and modified for aquifers in South Africa. Much work has been carried out by the Institute for Ground Water Studies (e.g. Hodgson et al 1981), the Department of Water Affairs (e.g. Bredenkamp 1982), the CSIR (e.g. Tredoux 1982), and various cosultants specialising in this field (e.g. Smith et al 1982). The mathematics involves the solution of second order differential equations or numerical methods like finite elements and is beyond the scope of these guidelines. Graphical methods involving flow-nets have been developed by Casagrande (1937).

In most cases where water is being abstracted for domestic purposes only, and particularly when equipped with handpumps, the amount of water abstracted will not affect the water supply of the aquifer to any marked extent, and provided extraction is well below the yield pump test results, no such analyses will be required.

Broadly speaking problems associated with abstraction of underground water will need to be addressed when one of the following two situations exist:

- The microscopic or local problem. This would be the case of a single borehole located in an aquifer of "unlimited" extent, where the problem would be to determine the flow rate (and hence safe pumping yield) under known conditions and given level of the water table. Or the inverse problem which would be to determine the position of the water surface (referred to as the cone of depression) for a given pumping rate.

- The macroscopic or overall resource problem. This would be the case of an underground water supply of finite extent in which numerous wells are operating. Here the problem would be to assess the safe yield capacity of the underground supply. Or the inverse problem which would be to determine the effect of abstracting a given volume of water from the underground supply within a certain time period.

In either case a knowledge of the mathematical theory is essential. In situations where the borehole is required to provide a large volume of water, or where the groundwater supply is very limited, it may be advisable to contact specialists in this field. It should be said however, that there are very few aquifers in Southern Africa which can easily be described by a mathematical model. Ground water resources are usually located in secondary source regions like faults and fracture zones.

3.6.3 *Yield pumping tests*

It is important to determine the safe pumping yield of a borehole, in order that the correct pump can be specified for the particular borehole, and to know the limitations of the borehole. For this reason a pumping test is performed on the borehole once it has been completed. A variable discharge pump should be used for this purpose with a sensor that can be lowered down the borehole to record accurately the level of the water in the borehole. The pumping rate at which the water level remains static in the borehole over an extensive period of time (usually 48 hours or more) is known as the safe yield of the borehole. In practice this is rarely, if ever done. The Borehole Association of South Africa has proposed the following yield testing method in their "Minimum code of practice for borehole construction and pump installation" (Mony 1989):

- Initially a step drawdown test, and then a maximum drawdown or constant-rate endurance test, should be carried out to determine the maximum yield.

- The period of the test shall be determined by the period that the borehole will be subjected to demand in normal use, as per the following table:

Production-demand period (hours per 24-hour period)	Minimum test period (hours)
up to 2 hours	4 hrs
2 - 4 hours	6 hrs
5 - 11 hours	24 hrs
12 - 17 hours	48 hrs
17 - 24 hours	168 hrs

3.6.4 *Borehole drilling planning*

A drilling rig is a specialised item of equipment requiring a skilled crew, support vehicles, equipment, maintenance and fuel. Good planning of a drilling programme is therefore essential if costly mistakes are to be avoided, and operational costs kept to a minimum. The reconnaissance of a drilling expert should be arranged prior to sending the drilling rig on site. For drilling programmes in rural areas, not only should a geophysical survey be undertaken, but also a sociological survey. Ideally the surveys should be carried out in conjunction with each other. This is especially relevant for a programme where handpumps are to be installed on each borehole. In such cases it is important for the community to have a say in the siting of the borehole, within the limits of the geophysical potential for a groundwater supply.

Alternative contracting approaches could be utilised for the drilling programme. For example Alan Hayes (1988) reporting on drilling operations in Sudan and Uganda found that the efficiency of the drilling programme was greatly improved by the introduction of a piece work payment scheme, based on progress rather than time spent in the field.

3.6.5 *Drilling equipment*

The most common machine is the truck-mounted rotary rig, which should be capable of drilling holes of up to 300 mm diameter to depths of over 100 metres. Another type of rig sometimes used is a cable-tool rig. This is similar to rigs used for site investigation. It can be towed by a tractor or 4-wheel drive vehicle. It is ideal for alluvial soils free from boulders. For shallow boreholes (in alluvial soils) a mechanical auger can be used (see Section 3.5 on tube wells).

It should be stressed that for best results in the longer term, a reputable driller should be employed. In South Africa, drillers recognised by the Borehole Association of Southern Africa are committed to following a code of practice.

3.6.6 *Borehole casing, screens and gravel pack*

It is essential to select the correct drilling diameter suitable for the size of the casing to be installed, plus any temporary casing required to keep the hole open during drilling and gravel packing.

The casings and temporary casings are manufactured from steel or semi-rigid plastic. For most hand pump installations a casing diameter of 100 to 110 mm is adequate, while submersible pumps normally require a minimum diameter of 120 mm.

The diameter of the hole should be a minimum of 100 mm over and above the external diameter of the casing, and temporary or permanent casing of this diameter may be required to keep the hole open during drilling and gravel packing operations.

The well screen or borehole screen is normally screwed or welded to the lower end of the internal casing. Screens are manufactured in sizes ranging from 120 mm to 300 mm diameter from plastics, nylon, brass or stainless steel. The screen is placed below the static water level in the borehole and is installed to allow the free passage of water while retaining the in situ material of the aquifer or gravel packing.

Gravel packing is normally placed in the annular ring between the outside of the well screen and casing and the inside of the drilled hole. The size of the gravel must be small enough to retain the *in situ* material but large enough not to pass through the slots in the well screen; a size of between 2 mm and 5 mm is usually ideal. The gravel acts as a filter medium to retain the fine silt and sand. Without a good filter, the life of the pump can be drastically reduced, and the water may be aesthetically unacceptable. The subject of borehole casing, screens and gravel packs is extensive, and again stresses the desirability of utilising recognised drilling contractors.

3.6.7 *Borehole development*

Development of the borehole is an important part of well construction. It is the process of removing fine particles from the aquifer surrounding the borehole screen in order to improve the yield of the borehole and to prevent clogging of the well screen and gravel filter.

Development can be performed by <u>surge plunging</u>, that is using a plunger or bailer dropped from a height to 'pump' the borehole; it is not suitable with plastic casings.

Compressors and pumps can be used for <u>air jetting</u>, <u>water jetting</u> or <u>air lifting</u> of the fines in the aquifer adjacent to the well screen. Development should usually continue for a period of between 12 and 24 hours, until the water from the borehole contains no appreciable quantities of sand, silt or clay.

3.6.8 *Use of drilling muds and foams*

A variety of foams and muds (commonly Bentonite) are employed in rotary drilling to minimize friction and hence enhance the drilling process. Where these are used they must be removed from the borehole after the drilling process is complete. If not removed by methods as described under 3.6.7 above, they may dry and coat or clog the inside of the hole.

3.6.9 *Borehole pump selection*

The various types of pumps suitable for installation with boreholes is described in detail in Chapter 6. It is important to have some idea in advance of drilling as to what type of pump is to be installed, so that the correct borehole casing can be selected. Pumps must be chosen to pump the desired quantity of water at the desired rate. This must be less than the safe yield of the borehole in the case of mechanical or electrical pumps to prevent the risk of the pump seizing if it runs dry.

For some pumps the rate of pumping can be controlled by adjusting the speed of the drive motor. In other cases this is done by selecting the appropriate combination of pulley sizes connecting the pump to the motor. Submersible electric pumps cannot be adjusted in the same way and it is important to specify the correct pump for the specified borehole yield and delivery head. If possible a borehole should be pumped lightly at first to prevent fine material from being sucked into the pump, thereby causing damage to it.

3.6.10 *Treatment of underground water*

Generally the treatment of underground water, to achieve the required bacteriological and physical (turbidity and colour) quality, is not necessary. Samples should be taken in sterilized bottles to laboratories in order to check for bacterial indications of pollution should this be remotely suspected (see section 4).

If coliform bacteria are identified in the borehole sample water, shock disinfection of the borehole by addition of chlorine is recommended. If persistent pollution is indicated by the presence of coliform bacteria in successive samples taken from the same borehole over a period of time, the source of the pollution should be identified and removed; failing which it might become necessary to abandon the borehole for domestic water supply purposes or alternatively to chlorinate the water on a continuous or semi-continuous basis.

Because chlorine is a strong oxidizing agent it is used, in addition to disinfecting the water, to remove tastes and odours, hydrogen sulphide, iron, manganese, slime and algae from borehole water.

Deep groundwater often contains salts and dissolved compounds (collectively measured as total dissolved solids - TDS), in higher concentrations than from shallow groundwater or surface water sources. It is not uncommon for people to complain about the salty or bad taste of borehole water, or its excessive hardness.

The cheapest method of remedying this problem is to blend borehole water with other potable water sources such as rainwater, or to use rainwater only for cooking and drinking purposes while using borehole water for other domestic uses. The removal of dissolved salts can only be achieved by distillation, ion exchange, reverse osmosis, or other sophisticated technologies that are described in section 5.7).

In many parts of Southern Africa, especially the Northern Transvaal, Lebowa and Bophuthatswana, borehole water contains excessive fluoride or nitrate concentrations. Levels of fluoride above 3 mg/l cause mottling of teeth, and at higher levels skeletal fluorosis may be observed. Nitrate levels of above 15 mg/l can be harmful to young children, especially to babies under one year old. Methods described in section 5.7 may need to be employed to reduce the fluoride or nitrate content of the water, especially for the water to be consumed by children.

3.6.11 *Management and maintenance of boreholes*

The topic of community involvement in the planning, operation and maintenance of water supplies has already been stressed in these guidelines. These principles are all the more applicable in terms of the management and maintenance of boreholes. Each borehole has a mechanical pump, whether hand or engine driven, which is likely to be in daily use. In many cases the borehole may be the prime source of water for the residents, and breakdowns will result in extreme hardships or inconveniences. Much emphasis has been placed in the International Drinking Water Supply and Sanitation Decade on the development of VLOM (Village Level Operation and Maintenance) hand pumps, and there are several pumps on the market that claim to approach this ideal (see Chapter 6). Many pumps already installed, however, cannot be repaired without specialised equipment. It is therefore of vital importance to the reliable operation of a borehole that each pump has an appointed caretaker who oversees its correct operation and maintenance on a daily basis, and contacts the relevant authorities when major breakdowns occur. The caretaker should be remunerated from the monthly contributions of the community, via a responsible water committee or equivalent.

It is important in designing hand pump installations that a concrete apron be constructed around the pump stand. This apron must be well sealed around the borehole casing to prevent ingress of surface water into the borehole itself. The apron must be constructed to drain water away from the pump. The height from the apron to the pump outlet must be designed to accommodate the water containers commonly used.

3.7 Rainwater

In households not served with a piped water system, the collection and storage of rainwater from roof run-off can play an important role in supplying potable domestic water supplies. In most areas of Southern Africa, the quantity of water which can be collected from even a modest roof area can be substantial. The limiting factor is, in nearly all instances, the provision of sufficient storage to allow a sustained supply of domestic water during periods of no rainfall. A further limitation of rainwater domestic supplies is that storage for long periods of time (typically 3 months or longer) can lead to the growth of organisms (algae, mosquito and other insect larvae) in the rainwater tank. Covering rainwater tanks used for rainwater collection is therefore essential to minimize these problems as well as to inhibit evaporation. Even when tanks are covered the presence of unwanted organisms in the water can present problems. Water that washes over roofs and gutters invariably carries small quantities of organic matter such as leaves, insects and bird droppings, plus small amounts of inorganic dirt and dust. These form the essential nutrients for the growth of algae, bacteria and various insect larvae. Stored rainwater may therefore require disinfection before use for human consumption. The removal of turbidity and colour is normally not necessary.

There is a danger of toxic substances associated with the use of various paints and coatings applied to roofs entering the rainwater as it flows across the roof. In this regard it is important to check on the manufacturer's specifications of various roof paints with regard to their suitability for use with potable rainwater collection systems. For example some of the recently developed acrylic based roof paints have built-in metal primers and fungicides, which are unsuitable for use with potable rainwater collection. Most of the older type paints are either oil-based or bitumen-based and as such are inert and present no dangers to human consumption.

3.7.1 *Estimating rainwater collection and storage requirements*

The quantity of water that is available for collection from an impervious roof is the total quantity of the rain falling on that roof area during the rainy period less losses due to evaporation, splashing, overflowing of gutters, etc.. Provided gutters are installed properly, designed to collect the roof run off from the average storm, and are regularly maintained and unblocked, a run off coefficient of 0,80 would appear reasonable.

Note that the area used for calculating the roof area is the horizontal projected plan area of the roof and not the slope roof area.

Example: A house with a roof plan area of 50 m^2 located in an area with a mean annual rainfall of 900 mm has the potential to collect 50 x 900/1000 x 0.80 = 38 kilolitres of water per year, using a run off coefficient of 0,80.

NOTE: A roof area of 1 m^2 will yield 0.8 litres of rainwater for each mm of rainfall.

Usually it is not economically feasible to provide sufficient storage to collect all the run off from a roofed area over a one year cycle. Instead the required storage capacity of a rainwater tank is calculated with reference to the demand, and this is compared with the estimated supply to see whether the latter is adequate. The capacity of the tank should be calculated as follows:

Storage tank capacity = The daily water demand multiplied by the longest expected period without adequate rain.

Alternatively a cumulative rainfall/consumption diagram as indicated in figure 3.13 could be used to calculate the required capacity.

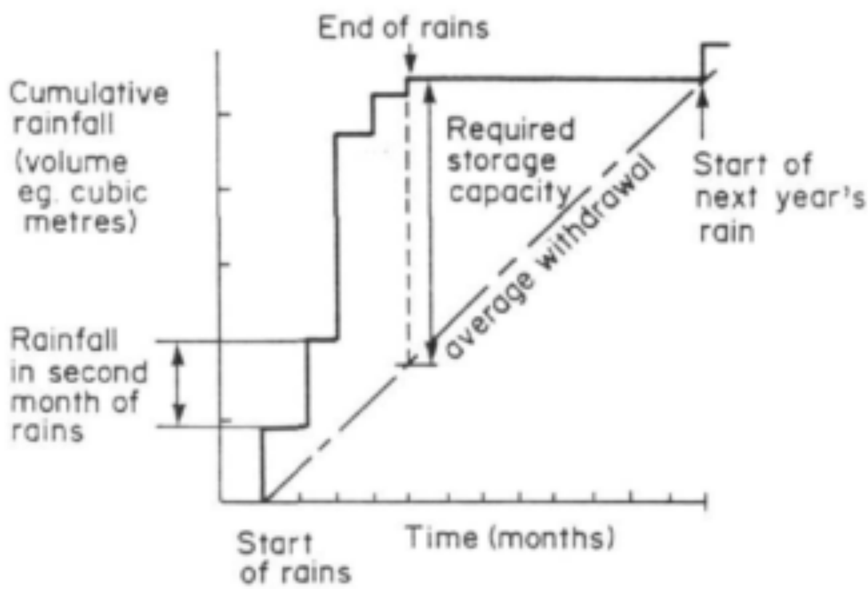


Figure 3.13 : Cumulative rainfall diagram used to calculate the required storage capacity

As rainfall is an unpredictable phenomenon, and because rainwater tanks are manufactured in standard sizes, it is not normal to go to great lengths in calculating required storage capacity. What is required is that the capacity is sufficient to meet normal demand over reasonably expected dry periods, and that sufficient roofed area and guttering is available to supply the tank with this volume of water, even under conditions of less than average rainfall.

Example: A family of 6 whose domestic potable water requirements is 100 litres per day live in an area where the longest expected dry period is 4 months (120 days). They will require 100/1000 x 120 = 12 kilolitres rainwater storage capacity to ensure a sustained supply of domestic water over this period. This could be provided by installing 3 standard 4,5 kilolitre rainwater tanks (combined storage 13,5 kilolitres). If the potential run-off was 38 kilolitres, as calculated previously, this would represent a balanced rainwater supply system (total consumption for the year will be approximately 36 kl, with storage for 13,5 kl). In a dry year (say 80% of average rainfall), the family would have to ration their water use, especially during the dry period.

3.7.2 *Types of rainwater storage tanks*

Materials commonly used in the manufacture of rainwater storage tanks include:

corrugated iron fibre glass asbestos cement high density polyethylene	) prefabricated
ferro-cement concrete interlocking blocks masonry reinforced concrete pre-cast concrete rings	) built <u>in situ</u>

The use of oil drums and various disposable plastic containers for rainwater collection is common in the rural areas. However, the storage capacity of these small containers (usually 200 l) is insufficient to meet the requirements as indicated in section 3.7.1. Instead, they provide just enough water for drinking (say 10 l per day). It is realised that rainwater collection systems are in most cases a family based system, and hence the cost of larger storage tanks may be out of the reach of many rural households. An important aspect then is to promote the use of lower cost tanks which can be constructed by the homeowner, or by a local contractor at a lower price. It has been found for example that the use of ferro-cement technology can substantially reduce the cost of rainwater storage tanks (at least 35 % cheaper than asbestos cement).

There are various technologies for constructing water storage tanks. The use of woven baskets coated with mortar having capacities of up to 3,5 kilolitres is reported by UNICEF in Appropriate Technology (1982). A method of constructing household water storage jars up to 1 kilolitre capacity using a sack filled with sand as a temporary mould covered with unreinforced mortar is described in Waterlines Technical Brief No. 1 (1984). These water storage jars are part of a successful integrated rural water supply programme in Thailand. In Mali bins built of adobe for grain storage have been converted to water tanks by lining with a thin ferro-cement layer.

3.7.3 *Guttering, downpipes and fittings*

Rainwater collection from rectangular roofs constructed from corrugated iron, asbestos sheets or tiles is a relatively straight forward affair. Most roofs constructed from these materials are supported by wooden rafters or trusses. Guttering can be attached directly using brackets to the ends of rafters or trusses or to facia boards which have been fitted to the latter. All well constructed houses should be equipped with guttering and downpipes to protect walls, doors, windows, and foundations from excessive damage caused by roof run off. Guttering, fittings and downpipes are commercially available in the following materials:

Asbestos cement
Galvanized iron
uPVC plastic
Aluminium sheet

These are available from most hardware stores and builder's suppliers. Methods of joining and attaching to the rafters depend upon the guttering material used and the necessary fittings will be available from the suppliers.

Where the woodwork to attach the guttering is poor or unsuitable, brackets are available to attach the guttering directly to the iron roof sheets. The Msinga Gutter Sling Bracket is one such low cost version which will accommodate any gutter shape. The slope of the gutter can be adjusted on the sling. It is manufactured by Gugulethu Enterprises, P O Box 30, Pomeroy, 3020, Natal.

Thatched and circular roofs are much more difficult to fit with guttering and related fixtures. In addition the amount of debris from weathered thatch increases the amount of contamination the rainwater will collect as it flows over thatched roofs. However, since thatched circular roofs are predominant in many rural areas of Southern Africa, methods for the collection of rainwater from them have been investigated. However most of these solutions are costly or impractical. A simpler solution to the problem of collecting water from circular or thatched roofs, is to allow the water to run off the eaves onto an impermeable surface or a gravel surface on the ground, and to direct the run off from these to underground water storage cisterns or tanks. More will be said on these techniques in Section 3.7.6 on Rainwater Harvesting.

3.7.4 *First flush separating mechanisms*

Because the first water to run off the roof will carry most of the debris and dirt which has accumulated on the roof and in the gutters, some mechanism to separate this "first flush" from the subsequent run off is desirable. Two mechanisms to achieve this aim are illustrated in Figure 3.14. The one is automatic the other manually operated. Another idea is to have a flexible elbow connecting the downpipe to the gutter outlet which can be manually controlled to divert the first flush to waste. A number of other such mechanisms are described in the literature, some beig automatic and others requiring manual supervision. In any rain water collection programme being implemented with the backing of an agency, the need for a first foul flush disposal mechanism should be stressed, and a mechanism which is acceptible to the community should be promoted.

3.7.5 *Rainwater quality*

A comparison of water quality from different sources in the Vulindlela district outside Pietermaritzburg by Alcock and Verster (1987) revealed that samples from rainwater tanks and experimental rainwater harvesters consistently showed the best water quality of all sources as measured by *E. coli*, coliform and total viable organism counts.

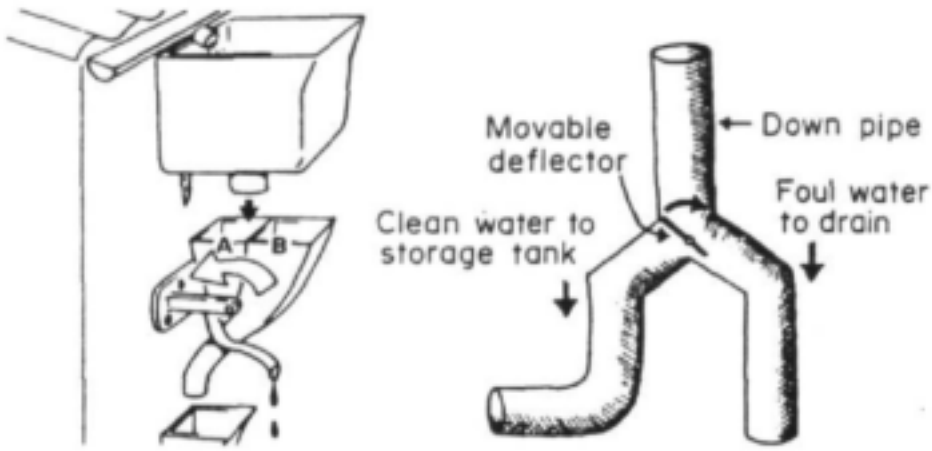


Figure 3.14 : First flush separating mechanisms

The total coliform bacterial count of 4,9 organisms per 100 ml, and an *E. coli* count of 1,9 organisms per 100 ml were significantly better than those obtained from protected springs (10,7 and 5,2 respectively), which in turn were significantly better than those for unprotected springs (19,1 and 10,2 respectively). Water obtained from surface water sources contained several thousand organisms per 100 ml.

3.7.6 *Rainwater harvesting at ground level*

The term "rainwater harvesting" has been used to describe systems as diverse as the collection of run-off from barren slopes in the Negev desert, developed 4000 years ago to irrigate lower lying fields; to the use of small area fibre glass, plastic or rubber collectors to catch and channel rainfall to underground storage tanks.

The former is more correctly classified as run-off agriculture, which is beyond the scope of these guidelines and is described in "More Water for Arid Lands" (National Academy of Sciences, 1974). The latter technologies were researched and reported on by Alcock (1985). Rainwater harvesting at ground level refers to the collection of run-off from impermeable surfaces at ground level.

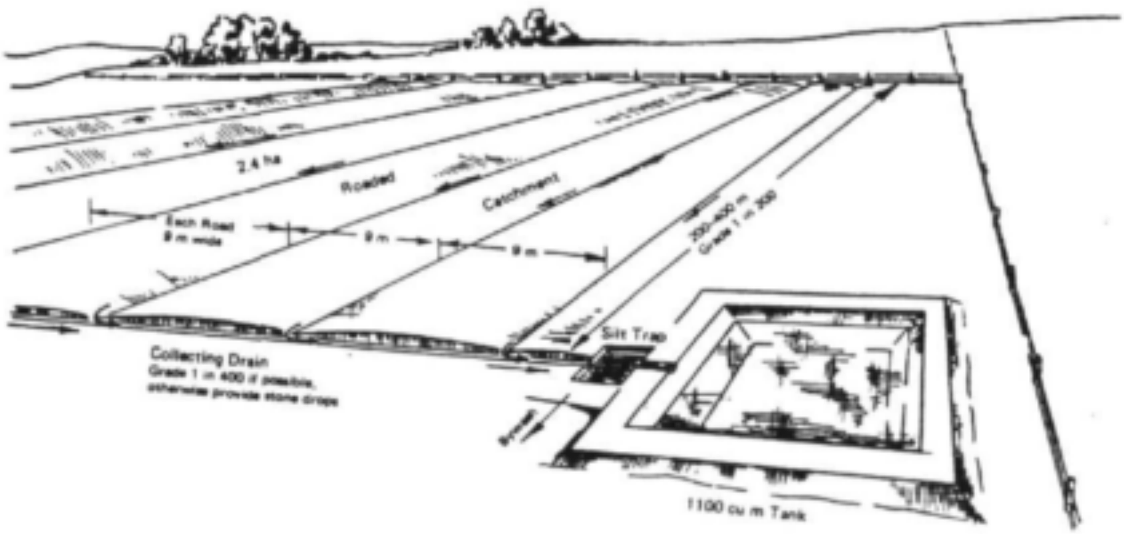


Figure 3.15 : Ground level rainwater catchment and storage

The water is collected from a prepared catchment area and directed to a suitable storage tank, which of necessity is normally located underground. The ground surface of the catchment area can be treated in a number of ways to increase the run-off. Some methods which have been used with varying degrees of success and cost are the following:

- concrete lining;
- corrugated iron sheets;
- plastic sheeting;
- plastic sheeting covered with a layer of gravel;
- various chemical soil treatments (e.g. paraffin wax);
- bitumen or tar lined;
- clay lined and compacted.

However, due to greater potential for pollution in ground level water catchments, water quality could be unsuitable for human consumption unless the catchment area is isolated from any possible human or animal interference. Storage tanks for ground level catchments are usually constructed underground. Again ferro-cement storage tanks have been found to be suitable for such applications (e.g. Waterlines 1986).

3.7.7 *Covering of rainwater storage tanks*

It has been shown previously that rainwater collection from impervious roofs is usually of a high quality, while harvesting water from ground level could be of a considerably inferior quality. In both instances the quality of the water will be further affected by the storage arrangements employed. For instance, settlement of dirt will occur in most storage tanks, therefore the outlet tap should not be located right at the bottom of the tank. However, by far the most significant improvement in stored water quality will be achieved by covering the storage tanks. Open storage tanks in a hot climate will develop algae within a few days of filling. In addition they will become the breeding ground for mosquitoes and other insects, will lose water through evaporation, and could be subject to pollution by humans, particularly children. Covering water storage tanks has several advantages, including minimizing algae growth, preventing the breeding of mosquito larvae, preventing the build-up of organic material, shading the water from the heating effect of the sun, and preventing animals and humans from contaminating the water. As a rule all rainwater storage tanks of whatever size should be completely and effectively covered. Sunlight should be eliminated entirely to prevent algae growth.

3.7.8 *Rainwater management*

In order to maintain a rainwater system in good working order and to maintain the optimum water quality from the system, certain tasks should be undertaken on a routine basis. As most rainwater collection systems are single home based, the responsibility for this should be taken on by some family member. These tasks include:

- Gutters should be periodically (every 3 or 4 months) cleaned to prevent build up of leaves and dirt;

- Inspection of attachments for deviation of "first flush" run-off after long dry periods and simple strainers or sieves on outlet pipes.

- Storage tanks should be checked for leaks and repaired internally if necessary. It will be necessary to clean out settled solids and slime built up on the inside of tanks when tanks are nearly empty, and for this reason an access manhole in the tank roof is essential.

- Inspection of the quality of the water in the tank. If water from rainwater storage tanks contains insect larvae, algae or other matter, it will be necessary to filter the water before use. If there is danger of faecal contamination of the water, as in the case of ground level catchments, the water should be checked for the presence of *E. coli* bacteria both at the end of the rainy season, and at the beginning of a new rainy season. If these are present, the water should be boiled before drinking or disinfected in the tank. This can be achieved by the addition of sufficient calcium hypo-chlorite granules or liquid chlorine bleach (see section 5).

- The presence of mosquito larvae can be avoided by the addition of small quantities of a light oil such as paraffin to the surface of the water in the tank. Overflow pipes must be covered with a suitable fly/insect proof screen material to prevent insects, frogs and snakes from entering the tank.

- The settled solids should be allowed to settle in the bottom of the tank and periodically flushed out. Tanks should be kept covered at all times to prevent the growth of algae. Potable rainwater should be conserved and used sparingly during dry periods. Other more likely contaminated sources of water can be used for washing and cleaning purposes.

The cost of rainwater supplies, including rainwater harvesting, is of the same order, if not more expensive than municipal supplies. However, rainwater supplies have in the past, and will in the future, afford an extremely important supplementary source of water supply during droughts and extended dry periods. In many other situations rainwater collection or rainwater harvesting offers an economically viable relatively safe and easily accessible supply of potable domestic water where no feasible alternatives exist.

3.8 Surface water

Surface water sources are used extensively throughout the world for the supply of water on a large scale both for irrigation purposes and for water supplies. The term "surface water" is used to describe run-off from rainfall which gravitates into the natural water courses such as streams and rivers. It is also used to describe open, semi-stationary bodies of water such as lakes, ponds, pans and dams. Specifically excluded from the definition are the shallow and deep groundwater resources, and rainwater, which have already been covered in the previous sections of these guidelines. Large rivers, dams and lakes, in addition to being potential sources of water, also can be used for a number of other purposes. These include transportation, fishing, recreation, receiving bodies for waste product disposal, treatment of certain organic wastes, and in some instances the generation of hydroelectric power. Where there is multiple usage of surface water resources, there is a need for rational management of these resources. In South Africa the responsibility for management of the water resources is vested in the Department of Water Affairs.

The quantity and quality of surface water resources varies considerably between geographic regions as well as between different sources within the same geographic area. As a general rule the quality of surface water with respect to bacterial content will be inferior to underground or rainwater within a specific populated geographic area. For this reason it is a general principle that surface water sources always require treatment to at least improve the bacteriological quality sufficiently for human consumption. There are of course exceptions to this rule such as mountain streams in areas remote from human habitation.

From a chemical and physical point of view the quality of surface water depends upon the nature of the ground through and over which the water flows. In Southern Africa where rainfall is markedly seasonal, high turbidity occurs in many surface water sources after heavy rains. This problem is becoming more severe in rural areas due to over-grazing and other poor farming practices, resulting in large quantities of top soil landing up in the rivers. During the dry season the water is usually of a low turbidity. The removal of dirt and suspended silt and clay from surface water sources is thus a feature of most water treatment facilities, even where the water may be relatively clear for most of the year. Another feature common to most surface water sources in areas of seasonal rainfall, is the need to impound or store surface water in order to ensure a continuous supply during periods of diminished stream flow. In South Africa most water supplies from surface sources requires the construction of dams to impound the water during the rainy season..

Because of the need to construct dams, build water treatment works and continuously operate and keep these works supplied with energy, skilled manpower and chemicals, surface water supply systems are often expensive and subject to economies of scale. They are, therefore normally restricted to supplying large complexes such as factories, institutions or towns. Small scale surface water treatment works, although usually more costly in terms of capital outlay than groundwater systems, can be designed to require a minimum of operation and maintenance inputs. An example would be a gravity fed and operated slow sand filtration treatment system. In many areas groundwater resources are not reliable or not sufficient, are very deep, or their quality makes them unsuitable for human consumption without costly treatment. So while a more desirable solution to rural water supplies is through the exploitation of ground water resources, experience and the needs of growing populations has resulted in surface water resources being developed and utilised on an increasing basis in rural areas.

The technology already exists to treat raw water of any quality to potable water standards. The problem is to correctly identify the most appropriate combination of technologies that will achieve criteria of quality and quantity, at the least affordable cost to the consumer, and within the technical, organisational and financial capabilities of the community to be supplied.

Because water quality constraints are far less stringent for agricultural applications, water treatment for irrigation is either not necessary or relatively less costly than for domestic and industrial supplies.

This section on surface water resources will briefly cover the aspects of impoundments and abstraction. Treatment is dealt with in section 5.

3.8.1 *Impoundment*

Where water is to be supplied on a year round basis from a stream or river with a seasonal flow, and where the dry weather flow in the stream is insufficient to meet the average daily requirements, some form of impoundment is necessary to store water during wet weather for use in the dry season. Such impoundments may be natural, such as pools in the course of the river or sub-surface dams created by one or more sheets of bedrock lying across a river bed. Where such natural impoundments cannot be found, or are inconveniently located with respect to the location of the community to be supplied, artificial dam walls, weirs or barrages can be constructed. The construction of dam walls with a water height in excess of 3 metres, must by law be designed by a civil or agricultural engineer. Walls, weirs and barrages of a lesser height can be constructed following fundamental guidelines and principles. A variety of materials including earth, masonry, concrete, boulders, brushwood, trees, timber, brickwork and plastic sheeting; or any combination of these, can be used.

Some low dam walls are constructed with wash-out sections, designed to be destroyed when the stream is in flood. Most, however, are provided with a spillway or weir crest which is designed to allow the passage of flood waters. The predicting of "design floods" takes into account a number of parameters, and is a complex procedure best performed by an engineer or hydrologist. For small dams or impoundments, provision for floods is best made by raising the artificial embankment some height above the natural topography alongside the watercourse. Flood waters will then pass down the natural embankment which is likely to be more stable instead of the artificially constructed embankment. It may be necessary to further stabilise the spillway section by planting of grass, or by lining the flow channel with rocks.

3.8.2 *Selection of site and site investigations*

When selecting a site for the construction of a small dam, attention should be paid to the following four basic requirements:

- the site selected should be the most economical;
- the storage should have low seepage losses, stable sides, and will not silt up;
- the foundations must be watertight and capable of supporting the dam;
- there must be sufficient material available for building the dam.

Usually the favoured site is one where the smallest embankment size provides the required storage. The optimum conditions occur when a wide valley occurs just upstream of a narrow gorge.

Having selected one or two possible sites, careful testing of the sites should be carried out. The main elements of these tests are :

. assessment of seepage losses from proposed site;
. investigation of the foundations for the proposed dam; and
. checking the availability of suitable building material close to the site.

<u>Seepage losses</u>: Because most storages are built well above the water table, pervious soils such as sands and gravels may result in excessive water losses due to seepage. Even some rock formations are permeable and care should be exercised when selecting a site underlain with such materials. A seepage test proposed by Nelson (1986) to assess the suitability of a site for a storage dam is as follows:

- sink 3 or 4 holes of about 100 mm diameter and 3 m deep in the storage area;
- pre-soak each hole to a 2 m depth for at least 1 hour before starting the test;
- maintain each hole at the 2 m level over a 24 hour period, and record the amount of water that has to be added to maintain the level at the 2 m mark.

If the water added is less than 3 l/hour, the site should be satisfactory. If the rate exceeds 30 l/hour, the site is too permeable for storage. For rates in-between these values, the site should be regarded as doubtful, and further tests will be needed.

<u>Foundations of dams</u>: Foundations must be capable of supporting the weight of the dam, and must be sufficiently watertight to prevent seepage under the dam. Generally foundations can be of clay, rock, or sand and gravel. Clay foundations are usually satisfactory for an earth dam provided they are not soft and saturated, in which case they should be removed. Most rock can support a dam, but care must be taken to ensure that :

. seepage does not occur between the rock foundation and the earth fill dam;
. weathering of the rock does not lead to weakening of the foundation; and
. permeable zones are not created by joints and faults.

Sand and gravel foundations should be avoided due to the high cost of making such dams watertight.

Availability of suitable dam building material: Soils placed in the embankment must be sufficiently impervious to keep seepage at a low, safe rate; and sufficiently stable to provide stability and strength. Good material contains approximately 25 per cent clay, with the balance made up of silt, sand and some gravel. Too much clay results in the embankment being weak, and prone to expand and contract with changes in moisture. Too little clay can cause excess seepage. Test holes should be dug to a depth of approximately 3 metres in promising borrow pits, and the material assessed for suitability.

The cost and time spent in investigating the proposed site for a small dam will save both time and money in the long run.

3.8.3 *Design considerations for dams and weirs*

Dams and barrages are used to impound water, whereas weirs are used to measure flow or divert part of the stream flow into a channel or sump. Permanent weirs and dam walls must be designed and built to withstand the total force of the water. This force under stagnant conditions is the hydrostatic pressure of the water and is a function of depth of water only. Under flood conditions dynamic forces caused by the velocity and momentum of the water can exert a considerably greater force on the retaining structure than the static water pressure. In addition floodwater can deposit a large quantity of silt and debris upstream of the dam wall and can cause erosion and undermining at the sides and downstream of the dam wall. It is very easy to under-estimate the force of raging floodwater, as witnessed by the large number of small dams that collapsed in Natal during the October 1987 floods.

Masonry and concrete dam walls or weirs should be founded, wherever possible, where continuous sheets of bedrock occur over the full width of the water course. Where the rock is not solid over the full width (it must be at least wider than the length of the spillway which is to be built into the weir), the wall can be keyed into earth retaining walls which in turn must be adequately keyed into the river banks.

An earthfill dam should be constructed on a layer of impervious material, which occurs over the full width of the water course. The dam wall itself should be constructed from relatively impervious material or else should have a central core of impervious material (such as clay) keyed into an impervious foundation.

The slopes of the earth retaining wall must be compatible with the material being used and the method of compaction. In general the slope should not exceed 2.5 to 1 (horizontal to vertical), and should preferably be at least 3 to 1. A sand filter should be placed on the downstream side of the clay core to drain any water which seeps through the core.

Spillways and the downstream toe of the dam must be adequately protected against scouring and erosion by floodwater. One method of doing this is to use gabion baskets or mattresses made from heavy diamond mesh wire filled with rocks and boulders. By employing a number of steps, the energy in the overflowing water can be dissipated as it flows over the spillway. The upstream face of the dam wall may require some protection against wave action in the dam. Rocks placed on the upstream face usually provide adequate protection.

An outlet pipe is usually necessary to both control normal flows passing downstream, and as an off-take for irrigation or water supply purposes. Importantly, the outlet can be used to lower the water level in the dam for inspection and maintenance. In areas where salinity problems occur, a bottom outlet pipe may be used to flush out saline water which accumulates at the bottom of the storage dam. Outlet pipes must be installed at the time of construction of the dam wall or weir. They should have one or more flanges welded on the outside of the pipe embedded within the wall or embankment to act as a cutoff for water seeping along the outside of the pipe. A suitably sized gate valve should be installed preferably on the upstream side of the weir or dam wall so that the pipe can be inspected for leaks, etc.. An energy dissipator (stilling box) may be required to break the force of the water at the outlet. The outlet pipe and valve should not be located in the path of flood water that overflows the spillway or weir crest. Because an earth embankment is likely to settle with time, the outlet pipe must be able to withstand differential movement. A steel pipe should therefore be used.

Masonry and mass concrete walls should have vertical upstream faces and downstream faces at a slope of 2:3 (horizontal to vertical). Where they are designed as reinforced or prestressed structures, these slope conditions do not apply.

Concrete can either be cast into removable steel or timber shuttering; or into permanent masonry or brickwork which is left in place, but not considered as part of the design width of the base of the wall. All concreting must be done with good concrete making materials, mixing and placing techniques. The rock layer onto which the masonry or concrete structure is to be built must be thoroughly cleaned, weathered material chipped away and the rock surface prepared to 'key' into the concrete wall (alternatively steel dowels can be drilled and grouted into the rock base). Large boulders (plums) of solid material not greater than one-third the thickness of the wall can be cleaned and tapped into the wet concrete to form a key for the next layer. Each layer of concrete must not be more than 500 mm thick and, according to Stephens (1986), must be thoroughly bonded onto previously constructed layers or the rock foundation using a 1 : 3 (cement : sand) slurry 10 mm thick. The concrete mix should be 1 : 2,5 : 3,5 (cement : sand : stone) or 1 : 3 : 5 if "plums" are incorporated. The concrete must be damp cured for a minimum of seven days but preferably 28 days.

The cross sections of typical dam walls/weirs are given in Figure 3.16, after Stephens (1986).

Rectangular or compound weir crests can be used to measure stream flow. The flow can be calculated from the following formula:

$Q = 1{,}84\ (L - 0{,}2H)H^{1.5}$ where

Q = flow in m^3 per second
L = length of weir crest in m
H = head of water over crest of weir in m

The head of water over the weir (H) can be measured by a gauging staff, which is permanently set at a convenient place in the stream flow upstream of the weir. It should be placed at a minimum distance of $4H_{max}$ upstream of the crest of the weir to avoid "draw down" effects, where H_{max} is the maximum head expected over the crest of the weir.

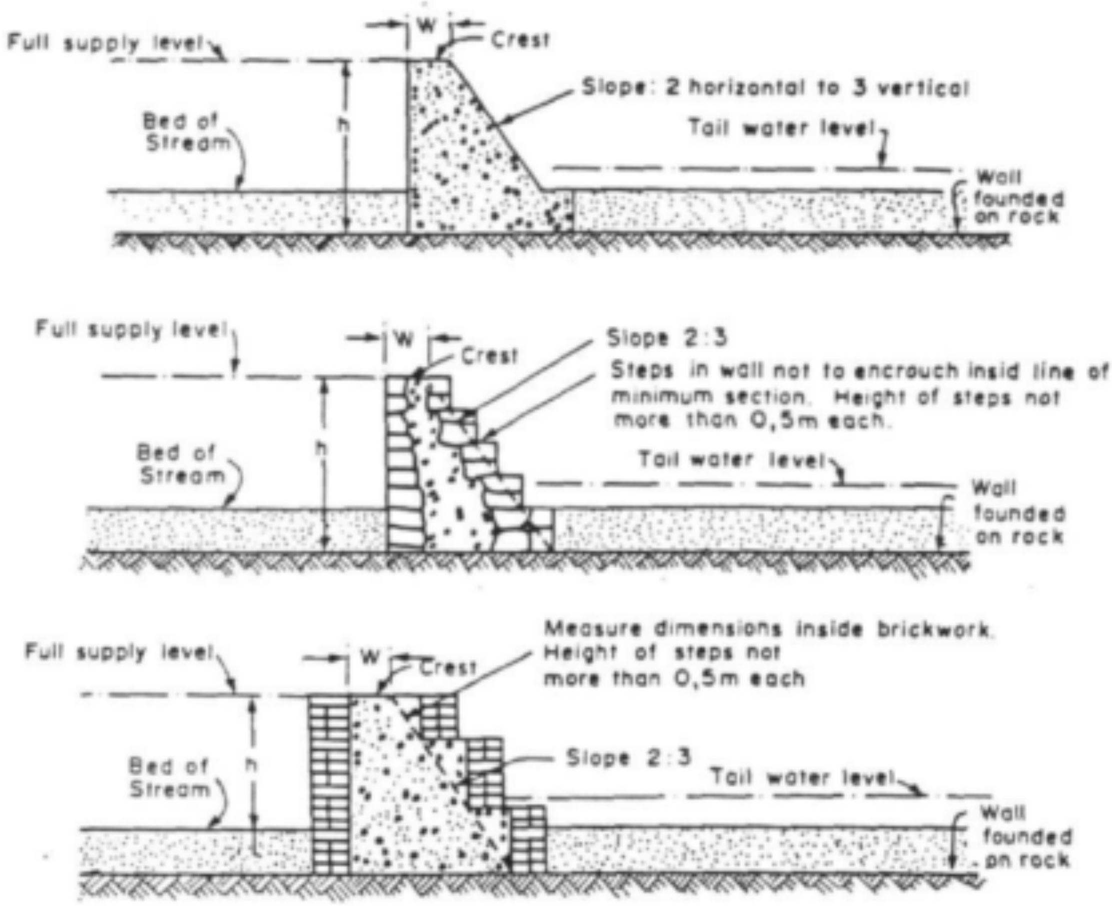


Figure 3.16 : Cross sections of weirs made of concrete (top), masonry or masonry-concrete (middle), and brick-concrete (bottom) - Stephens (1986)

3.8.4 *Sand-filled dams (sub-surface dams)*

In areas where there is a high rate of evaporation and a marked seasonal run off pattern, there may be advantages in storing water within a sand bed, as opposed to storing water in open surface reservoirs or dams. Where the water level is kept 30 cm below the surface of the sand, evaporation can be reduced by about 90 per cent in high evaporation areas. Where the water level is 1 metre below the sand surface, evaporation effectively ceases. Several sand-filled dams have been built in Namibia. The dam wall is constructed across the dry river bed during the dry season. When the river floods, sand and gravel are deposited behind the dam wall. Since most of the soil carried in flood water is fine sediments which would tend to silt up and clog the dam, the dam wall is heightened in stages of about 1 metre only. The floodwaters then tend to deposit the heavier gravels and sand while carrying the fine silt over the wall. The dams are typically built to a full height of between 6 and 10 metres. Water is drawn off by either an outlet pipe built through the dam wall or by a well dug into the sand upstream of the dam wall, into which a pump can be lowered.

Sand filled dams can be particularly effective when built over fissures that lead to underground aquifers, the dam impounds floodwater to recharge the aquifer. This technique has been described as "gulley plugging" in India.

The best sites for construction of groundwater or sand dams are on gently sloping land (typically 1:500 to 1:25), where the soil consists of sands and gravels, with rock at a depth of a few meters.

3.8.5 *Abstraction*

Water to be abstracted or withdrawn from surface sources can either be diverted under gravity into a channel or pipe at a weir or groin extending part way across a river, or be pumped from the river directly. In the former case some form of sluice gate and intake works will be required to control the rate of flow of the water. Where water is to be pumped, some arrangement of screens and strainers is normally provided to prevent debris from entering the pump suction and thus damaging the pump. The design of intake works must take into account the configuration of the river during both normal flows and during floods. Since surface water often carries high silt loads, some degree of "pre-treatment" at the point of abstraction is desirable. In areas where salinity problems in surface waters occur, care must be taken not to locate the abstraction pipe at the bottom of the dam or reservoir as this is where the more saline water will accumulate. A floating type intake (figure 3.19) would be preferable.

3.8.6 *River bed infiltration*

In river beds of sand or gravel, perforated pipes can be laid to abstract water from beneath the river bed. The sand and gravel in the river bed acts as a natural filter in removing finer particles from the water. The continuous action of the flowing water tends to wash the filter so that clogging is prevented. In some cases these infiltration galleries provide water of such good quality that no further treatment may be necessary, except perhaps chlorination. The slotted pipes should be laid in a granular bedding. The size of the gravel or sand must be sufficiently large not to enter the slots or holes in the pipe, but small enough to prevent the river bed material entering the slotted pipes. It might be necessary, depending upon the particle sizes of the river bed material, to form a graded filter around the pipe with successive layers of different particle sizes. The top of the gravel pack must be at least 0,5 metres below the surface of the river bed to prevent scouring. The infiltration gallery must be placed in a part of the river that is not likely to be subjected to excessive scouring during floods. A flow velocity of between 0,5 to 1,0 metres per second should be maintained within the infiltration pipe when water is being abstracted. Slower velocities lead to the accumulation of silt within the pipe, which could subsequently block; higher velocities cause excessive friction losses, which can lead to uneven withdrawal of water from the infiltration pipe.

A modification of this system is shown in Figure 3.17. Here a perforated pipe is laid upstream at the foot of a weir. The pipe is surrounded by a graded gravel filter and leads to a sump at one side of the river. The filtered water is then pumped from the sump. Another simpler modification is to divert part of the flow from the river into a small channel into which the perforated pipe has been laid and surrounded with a graded gravel pack. In the former modification the pipe can be cleaned, if necessary, from the open sump. In the latter case the gravel in the channel can be cleared by raking and scraping-off excess sediment.

When river bed infiltration systems become blocked and the flow is reduced to unacceptable rates, they can be backwashed with water, or preferably, first an air scour followed by water. The river or stream should be flowing when this is done so that the fines and debris removed by backwashing are carried away by the stream. In extreme cases it may be necessary to excavate around the pipe and replace the gravel filter.

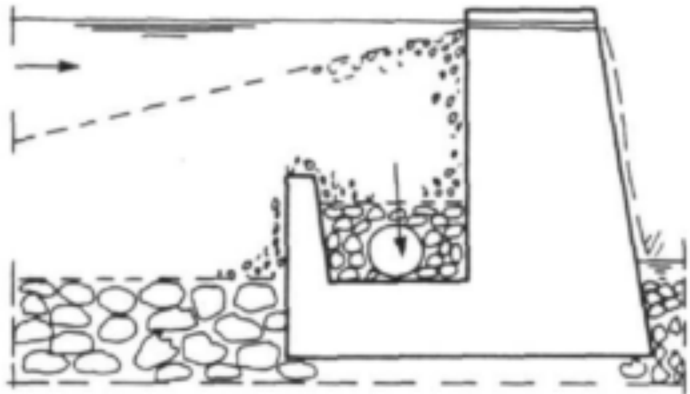


Figure 3.17 : Sub-surface abstraction system in a weir (Salazar 1980)

3.8.7 *Pump intake filter*

Most pumps abstracting water from a river or dam will be equipped with a strainer at the intake of the pump suction.

An efficient and useful adaptation has been developed and tested by G Cansdale of SWS Filtration Ltd, England (1982) and is shown in Figure 3.18.

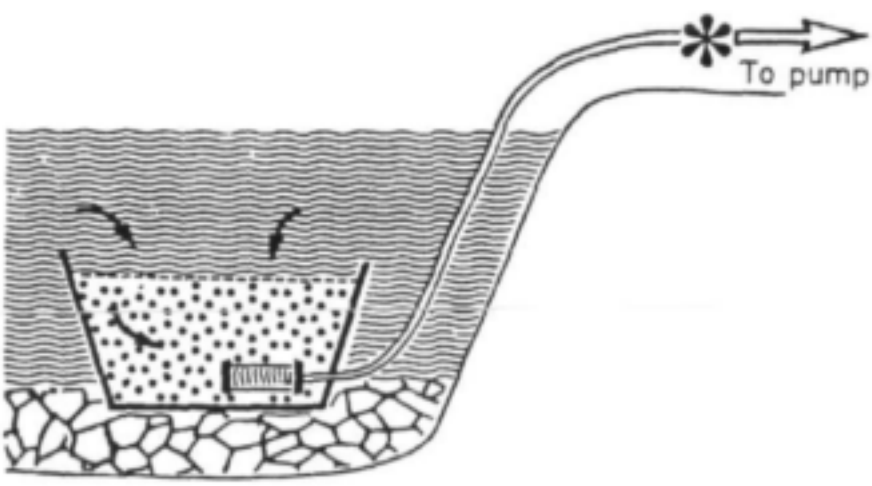


Figure 3.18 : Filter in gravel filled container on canal bed

The Intake Filter which is attached to the intake end of the pump suction is a fibre glass box or stainless steel box and has a cross-section of 0,3 m x 0,6 m. The box is fitted with a slotted plate and is buried in the river bed if the river bed material is sufficiently course (at least 50 per cent of the particle sizes should be between 1 mm and 5 mm). If the river bed material is too silty the filter can be placed inside a gravel-filled container, which is then placed on the river bed. The unit is capable of delivering between 12 and 22 m^3 of water per hour, and suspended solid reductions of 98 per cent and *E. coli* reduction of 80 to 90 per cent are claimed. The filter works on the principle of an induced gravity filter, whereby the opening of the box induces a zone of the river bed or in the artificial gravel packing to act as a downflow filter to the water being sucked into the intake pipe.

When the river or stream is flowing, a self-cleaning effect is maintained, as the stream flow washes accumulated sediment from the surface of the filter bed. Under stagnant water conditions the filter box is placed at mid water level, as this is the cleanest water zone. The unit has been successfully used to abstract water from highly turbid irrigation canals in Sudan, where it is reported to have effectively eliminated the bilharzia cercariae from entering the water supply.

There are obviously several advantages in some form of pretreatment at the point where water is being abstracted from surface sources. The above techniques and devices are given as examples of what can be done. A good deal of innovation, research and development remains to be done in this field. The use of geofabrics and membranes to replace or supplement natural filters has not been touched upon.

3.8.8 *Offtakes for the abstraction of water directly from the water body*

The abstraction of water directly from the water body (river, dam, or lake) may be the most feasible choice in certain situations. Various methods are available for doing this. The intake pipe could simply be fixed at a certain level within the dam or river, and fed to a sump or the intake of a pump. It is desirable at times to selectively abstract water from different levels, or from a specific level within the water body (e.g. from 1 m below the surface). This can be achieved by having multiple level offtakes, or alternatively, by having a floating offtake. Figure 3.19 illustrates one possible arrangement for floating offtakes. It is important to ensure that any intake arrangement is able to withstand the effects of a flood, or be able to be removed to higher ground in the event of a flood.

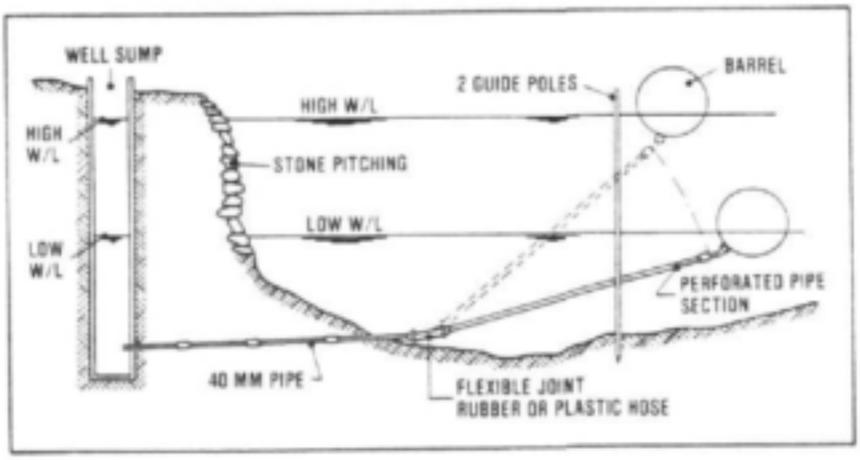


Figure 3.19 : Floating intake for surface water abstraction

4. WATER QUALITY

In the natural environment there is no pure water available for general use. All water, including rainwater, has some impurities. The impurities are commonly in the form of dissolved solids or gases, suspended solids, and micro-organisms. The importance of water as a vehicle for the spread of disease is the main concern in terms of water quality control. However, certain other quality parameters may also be of concern, for example the aesthetics of the water (clarity and taste) and the corrosivity or scale forming potential.

Many studies have been conducted on the effects of water quality on human and animal health, on its general acceptability, and on the useful life of water conveying structures, including pipes, dams, tanks, water heaters and plumbing fixtures. Following such studies, guidelines have been proposed for quality standards to which domestic water supplies should comply.

A guideline value represents the level (a concentration or a number) of a constituent that ensures an aesthetically pleasing water and does not result in any significant risk to the health of the consumer. The quality of water defined by the guideline values is such that it is suitable for human consumption and for all usual domestic purposes, including personal hygiene. In the case of small community supplies, particularly in the rural areas of developing countries, the number of parameters used in assessing and measuring the quality of water intended for public supply must necessarily be limited. Similarly the guideline values given have often to be considered as long-term goals rather than rigid standards that have to be complied with at all times and in all supply systems.

In South Africa three criteria levels or guideline values have been proposed. These are as follows:

i. recommended or working limit within which drinking water is considered to be safe for a lifetime's consumption, or maximum level for no risk;

ii. maximum allowable limit or the maximum level for insignificant risk, and

iii. crisis limit or the maximum level for low risk.

This three tier system allows health authorities to approve water of a poorer quality under certain specific conditions, and makes provision for an upgrading of supplies.

4.1 Quality of common water sources

Various sources of water are used in rural and developing areas, each with differing quality of water in general.

- Rainwater and snow are the purest forms of water naturally available, with low levels of dissolved salts and, if care is taken in collection and storage, low levels of suspended solids and micro-organisms.
- Surface water sources are virtually always contaminated with micro-organisms and suspended solids, but the level of dissolved solids is usually always acceptable for drinking water (except of

course for sea water). The degree of contamination by suspended solids and micro-organisms generally increases from source (mountain streams) to the river mouth. In addition contamination usually increases with rainfall events. However, large expanses of stationary surface waters often undergo a self-cleansing process both in terms of suspended solids and micro-organisms, though the growth of algae in the dam or lake may actually increase the levels of suspended solids. Surface water sources should always be treated before use for domestic water supplies, at least for disinfection.

- Underground water abstracted by means of bore holes, wells and springs is usually of a good quality in terms of micro-organisms and suspended solids, but may have high levels of dissolved solids. Bore holes are usually the safest sources of water for domestic supplies, providing a reliable yield even in times of drought and with no additional treatment of the water required. Some bore holes, however, may be brackish (salty) and hence unpalatable, contain high levels of fluorides or nitrates which have health implications, or be corrosive or scale forming. Wells and springs, while potentially supplying a good quality water, are more easily contaminated by surface storm water run-off or through the use of contaminated containers. When adequately protected, these sources can be of a very good quality. Pit latrines or other sanitation systems located close to the point of abstraction may, however, result in bacteriological contamination of this water, or, in the longer term, a build up of nitrates.

4.2 Microbiological aspects

Emphasis is placed first and foremost on the microbiological and biological quality of drinking water supplies. Ideally, drinking water should not contain any micro-organisms known to be pathogenic (i.e. which cause sickness and disease). Since the most common sources of pathogenic micro-organisms are from faecal material, drinking water should be free from organisms indicative of faecal pollution. Faecal material can contain a variety of bacterial, viral and protozoan pathogens and helminth parasites. Some of these organisms can cause acute diarrhoea, one of the main causes of infant morbidity and mortality in the developing world.

4.2.1 *Bacteria*

To ensure the absence of bacterial pathogens such as *Salmonella*, *Shigella* and *Vibrio cholerae* the water supply should be free of faecal organisms. The primary bacterial indicator recommended for this purpose is the coliform group of organisms. Although as a group they are not exclusively of faecal origin, they are universally present in large numbers in the faeces of man and other warm blooded animals. A subgroup of these coliform organisms, the faecal (thermo-tolerant) coliforms, or in particular *Escherichia coli*, provides definite evidence of faecal pollution.

In South Africa the guideline values for bacteriological quality for drinking water are as follows:

	NO RISK	INSIGNIFICANT RISK	LOW RISK
E. coli/100 ml	0	1	10
Total coliforms/100 ml	0	5	100
Standard plate count/ml	100	1000	10 000

4.2.2 *Viruses*

When drinking water is contaminated with enteric viruses, two diseases may occur in epidemic proportions - gastroenteritis and infectious hepatitus. Viruses may remain viable in the aquatic environment for months and are more resistant to environmental factors than the coliform indicator bacteria. However it is costly and difficult to measure viruses in water, although a virus indicator test (i.e. coliphage test) has been developed which is a quick and low cost alternative. To ensure a water supply that is free from viruses, the source of the supply should be free from faecal contamination. Where this is not possible the following treatment will ensure the inactivation of viruses:

- turbidity of 1 NTU or less;
- at least 0.5 mg/l of free residual chlorine
- at least 30 minutes contact time
- pH preferably below 8.0

The guideline values for viruses in drinking water are as follows:

	NO RISK	INSIGNIFICANT RISK	LOW RISK
coliphages/100 ml	0	10	100
Enteric viruses/10 l	0	1	10

4.2.3 *Parasites* (protozoa and helminths)

There are also certain protozoa, in particular *Giardia* spp and *Entamoeba histolytica*, which if ingested, can lead to giardiasis and amoebic dysentery respectively. These organisms can be introduced into a water supply through human, or in some instances animal, faecal contamination. Coliform organisms are not a good indicator of the presence of these protozoa in treated water because these parasites have a greater resistance to inactivation by chlorine. Some laboratories are geared to test for these protozoa, but it is a better policy to use sources not subject to faecal contamination where possible. In cases where such sources are not available, adequate treatment of the water by filtration should be ensured.

4.2.4 *Transmission of diseases*

Most of the diseases associated with water are enteric of nature and may be transmitted from a sick person (or carrier) to a healthy person via the excreta (faeces and/or urine) of the infected person. The micro-organisms reach the healthy person through contaminated hands, food, milk or water. Insects such as flies and even animals may play a role in this transmission process. Sometimes specific intermediate hosts are part of the life cycle of parasites.

To break the cycle of disease transmission, the following steps should be taken:

i. sanitary handling, treatment and disposal of excreta to prevent personal, soil and water contamination

ii. personal hygiene by washing hands after defecation and before handling food

iii. practice sound pest control and especially prevent fly breeding

iv. treatment and disinfection of contaminated water

v. avoidance of contact with bilharzia infected water

In rural areas educational programmes at schools, clinics, and community meetings should be used as a means of making people aware of these aspects.

4.3 Chemical and physical aspects

There are a number of chemical and physical parameters which should be assessed when determining the quality of a water source. These include turbidity (surface waters), fluorides, nitrates, iron, total salts (ground waters), hardness and stability (all waters).

Turbidity (suspended solids)

Turbidity affects the aesthetic quality of the water, but its primary importance is in relation to water disinfection. The amount of chlorine needed for disinfection often increases as the turbidity increases. Certain suspended solids which are organic in nature may also impart an undesirable taste to the water. The suspended solids are usually clay and silt particles arising from rainfall events, floating plant matter like algae or other water plants, or organic matter from pollution.

Fluorides

Excess fluorides in the diet result in mottling of teeth and bone fluorosis. In some parts of Southern Africa groundwaters contain fluoride levels as high as 15 to 20 mg/l. These levels are a result of the natural geological formations in the area. Some industrial effluents contain high levels of fluorides but by law these may not be discharged into the river courses without sufficient dilution.

Nitrates

While being relatively non-toxic to adults, nitrate is potentially lethal to infants. Fatal methaemoglobinaemia can occur at nitrate concentrations in excess of 10 mg/l. Nitrates occur in underground water from natural formations, from excessive use of fertilizers by farmers, and from leachates from pit latrines and other waste disposal sites.

Iron

While iron in high concentrations is potentially toxic, its aesthetic undesirability manifests well below potentially toxic concentrations. Iron concentrations above 300 µg/l (0,3 mg/l) in water gives rise to discoloration, staining and taste problems. Most groundwaters in Southern Africa contain some iron from the natural geological formations. Iron is also used as a flocculant in water purification, or it can arise from corrosion in the distribution system.

Total salts

Due to osmotic effects of high salt levels, saline water is generally unpalatable. Certain underground waters in the south-eastern, central, northern and north-western parts of Southern Africa are highly saline and thus not fit for human consumption. In addition saline water may be very corrosive or scaling in nature. Total dissolved salts are usually measured by means of the electrical conductivity of the water.

Hardness and stability

Hardness, the presence of multivalent cations (especially calcium and magnesium) is not a problem as far as health is concerned. In fact some hardness in the water is desirable as a protective factor against heart disease. Excessive hardness however, results in an excessive consumption of soap for laundry. Groundwaters may often contain high levels of hardness, particularly in the dolomitic zones. The degree to which water will either corrode a metal pipe or form a scale deposit within the pipe is a measure of the stability of the water. Ideally a water should be just scale forming so that a thin protective coating is deposited on the inside of the pipes, etc.. The stability of the water is determined from measurement of the pH, calcium content, alkalinity, total salts, and the temperature. A parameter, the Langelier saturation Index (LI), can be determined from these measurements. A positive LI value indicates that the water is scale forming, whereas a negative value indicates a corrosive water. Although the LI can be determined by hand calculation, the method is tedious and time consuming. A computer programme is available from the Water Research Commission (PO Box 824, Pretoria, 0001, South Africa) which does the determination automatically, or alternatively graphical methods may be used.

The guideline values for the above physical/chemical quality parameters are given in the table below:

DETERMINANT	UNIT	NO RISK	INSIGNIFICANT RISK	LOW RISK
turbidity	NTU	1	5	10
fluoride	mg F/l	1	1,5	3
nitrate	mg N/l	6	10	20
iron	mg Fe/l	0,1	1,0	2,0
hardness	mg $CaCO_3$/l	20-300	650	1300
electrical cond.	mS/m 25 °C	70	300	400

In addition to the above quality parameters, there may be certain organic contaminants which would be of concern. These include the following:

- pesticides, normally chlorinated hydrocarbons, resulting from farming practices and subsequent run-off into the water courses.
- oil and petrol, from leakage or dumping practices which may enter the water sources by seepage or run-off
- various toxic, from use of improperly cleaned ex industrial containers

These contaminants can usually be detected by the taste or odour they impart to the water. Should such contamination be suspected, the water source should not be used until the nature and source of the contaminant has been determined and a decision made on its safety or otherwise. In some cases treatment to remove the contaminant may be required. However it should be appreciated that the contribution of drinking water as a source of organic matter consumed by humans is very low. Food and air intakes normally far surpass those of drinking water, except in the case of chloroform in chlorinated water, or by accidental consumption as in examples given above.

4.4 Methods of testing water

Certain laboratories in Southern Africa are set up to test water samples, and should be used when good, accurate analyses are required. Many hospitals will also be able to carry out bacteriological analyses. Water laboratories will be found at most water boards, the SABS, CSIR, some of the larger municipalities, some mining houses and some private consultants.

However, a knowledge of methods of analysing water quality is important for three reasons:

- Certain analyses must be carried out immediately the sample is taken, otherwise the parameter to be measured may change;
- The cost of having the samples analysed may not be justified or affordable;
- The results can be interpreted more accurately when some concept of the methods of analysis and their limitations, are known;
- It may not be possible to send the samples to a laboratory within the required time period.

It is not the purpose here to give comprehensive methods on the different chemical, physical and bacteriological analyses of water, but rather to list some of the most common methods and discuss those which can be used for field testing in particular.

4.4.1 *Bacteriological analyses*

This is probably the most difficult analysis to carry out as sterile conditions must be maintained and incubation of the prepared cultures must be carried out. It is important that sterile sample bottles be used for collecting the samples. The bottles can be sterilized by "autoclaving" in a pressure cooker at 15 psi for 20 to 30 minutes, or by heating in a dry oven at 170 °C for 1 hour. The samples, once collected, should be stored in covered cooled containers (cool boxes) at a temperature of 4 to 10 °C.

There are two methods available for counting the number of faecal indicator organisms present in the sample:

i. The multiple tube fermentation technique in which measured volumes of sample are added to sterile tubes containing a suitable growing medium. After incubation (37 °C for 48 hours) gas and acid production is measured. The results give a statistical estimate of the most probable number of organisms present.

ii. The membrane filtration technique in which measured volumes of sample are filtered through a membrane filter (0,45 µm pore size). The micro-organisms remain on the filter which is then incubated face upwards in a suitable growing medium. After 24 hours visible colonies can be counted, and expressed in terms of the number present in 100 ml of original sample.

Of these two methods, the membrane filtration method is more accurate. Equipment for incubation, sample filtration, etc. are fairly expensive, and the person carrying out the analyses should receive adequate training.

A third method which can be used to get an indication of whether contamination has taken place, and to give some indication of the severity of the contamination, is with the use of commercially prepared "dip-sticks". These dip-sticks have a growing medium coated on a stick marked with a grid. The stick is kept sterile until used. It is then brought into contact with the sample and some of the bacteria attach themselves to the growing media. The dip-sticks must then be incubated for 24 hours and the colonies can be counted. It is possible to achieve a reasonable incubation at 35 to 37 °C by carrying the dipsticks close to one's own body.

4.4.2 *Chemical and physical analyses*

Chemical and physical analyses are usually much more rapid than bacteriological analyses. Sample bottles need not be sterile and hence cheaper plastic bottles can be used. The main techniques for field chemical analysis are:

i. colorimetric, in which the constituents in the water react with chemicals to produce a coloured product. The intensity of the colour indicates the concentration of the constituent. This may be done in practice by

- paper test strips
- printed colour comparator cards
- discs and comparators
- field spectrophotometers

ii. titrimetric, in which a selected titrant reacts with the constituent in the water, resulting in the development or disappearance of a colour indicator. The amount of titrant used to reach the end point (i.e. when all of the measured constituent in the water was reacted with the titrant) gives a measure of the amount of the constituent in the water. In practice this is done by

. dropping burettes
. digital titrators
. tablets

iii. specific ion electrometric in which an electrode is able to measure the concentration of a specific ion (e.g. fluoride) directly.

The methods used for the various analyses of interest are as follows:

chlorine (free and/or combined)	: colorimetric - usually with discs and comparator or printed colour cards
turbidity	: spectrophotometric (or turbidimeter)
colour	: colorimetric - usually with colour cards or cubes
fluoride	: specific ion electrode
nitrate	: specific ion electrode or colorimetric
iron and manganese	: colorimetric - usually with colour discs or spectrophotometer
hardness	: titrimetric
conductivity	: conductivity meter

Specific organic analyses should be carried out by a laboratory, but smell can often be used to detect the presence of certain organics, e.g. pesticides, oil or petrol.

5. WATER TREATMENT

5.0 Introduction

The extent to which water for domestic use is treated will be limited by economic and technical considerations. In developing areas complicated treatment schemes are not suitable. In cases where it is required, a better solution may be to exploit an alternative unpolluted source which requires little or no treatment, even when this source is at a greater distance. In some cases a combination of sources provides the optimum solution. For example, in Thailand rainwater collection and storage for drinking water, used in conjunction with a hand pump for domestic hygiene, and dam water for cattle watering, has been found to be a good solution in many rural areas. In this case virtually no treatment is required even though the borehole water is brackish and the dam water is bacteriologically contaminated.

The agency responsible for water quality control in an area should oppose the planning and implementation of treatment processes which the community concerned cannot afford to procure, operate or maintain with its available human and financial resources. As stated earlier, the important question to ask concerning a new water supply is "will the system be sustainable in the long term ?"

Experience worldwide has shown that important design considerations for treatment systems in developing countries are as follows:

- Construction and operation costs should be compatible with the resources and preferences of the users, and limited or no use should be made of imported materials.

- Operation, maintenance and construction (if feasible), should be within the competence of local technical staff or the users. Prior to construction, an assessment should be made of available skills in the community and the water agency.

- A minimum of electrical and mechanical equipment should be used, and this should be sturdy, reliable and preferably available locally.

- The quality of water supplied should not under any circumstances deteriorate below certain acceptable limits during the period of time for which the system has been designed.

- Special steps should be taken to consult the women and to involve them in local management of the scheme, including operation and maintenance, because they are the first users and have a direct interest in keeping it functioning.

- Appropriate systems should be included to monitor the performance of the treatment system.

- Provision should be made to prevent, or deal with, possible deterioration of the quality of raw water or breakdown of the treatment system.

- Treatment processes should be integrated to provide efficient production of high quality water at minimal cost.

- Service from manufacturers and chemical suppliers should be readily available.

- Disposal sites for plant wastes should be readily available.

- Good access to all process units should be provided.

The chances of the successful implementation and use of the proposed scheme is greatly enhanced when these factors are taken into consideration.

TABLE 5.1 GUIDELINES FOR THE SELECTION OF A WATER TREATMENT SYSTEM FOR SURFACE WATER IN RURAL AREAS (adapted from IRC 1987)

Average raw water quality	Water demand m^3/d	Treatment suggested	skills need	capital + op. costs
turbidity < 5 NTU faec coliform 0/100ml bilharzia not endemic	up to 2000	no treatment	nil	nil
turbidity < 5 NTU faec coliform 0/100ml bilharzia endemic	up to 5000	1. rapid filtration or 2. slow sand filtrat	med low	med + low med +v.low
turbidity < 20 NTU faec coliform 1 - 500 per 100 ml	up to 5000	1. rapid sand filtrat + disinfection (Cl_2) 2. slow sand filtrat. + disinf. if poss.	med low	med + med med + low
turbidity: 20-50 NTU faec coliform 1 - 500 per 100 ml	up to 5000	1. sedimentation + rapid sand filtrat. + disinfection (Cl_2) 2. sedimentation + slow sand filtrat. + disinf. if poss.	med med	high + med high + low
turbidity: 50-150 NTU faec coliform > 500 per 100 ml	up to 5000	1. pretreatment (coag floc & sedimentat.) + filtration (slow or rapid sand) + disinfection	high	v. high + high
turbidity > 150 NTU	detailed investigation and possible pilot study work may be required			

5.1 Choice of water treatment

The various treatment processes which may be used to improve water quality are as follows:

- disinfection
- filtration
- sedimentation
- oxidation
- coagulation and flocculation
- more sophisticated processes (ion exchange, adsorption, membrane separation, etc.)

The choice of treatment depends to a large extent on the average raw water quality, the volume of water required, and the design considerations listed above.

Table 5.1 may be used as a rough guide to the selection of treatment processes for rural areas in terms of disinfection and turbidity removal, but it is recommended that each case be thoroughly investigated before making the final choice. In particular, the possibility of using a combination of sources may result in a less costly treatment process being required.

The primary treatment requirement is always to ensure a microbiologically safe supply for drinking purposes. Hence disinfection of the water will usually be the primary treatment objective. Secondary objectives may include clarification of the water, and the removal of potentially harmful chemical components.

5.2 Disinfection

The single most important requirement of drinking water is that it should be free from any micro-organisms that could transmit disease or illness to the consumer. Processes such as storage, sedimentation, coagulation, flocculation, and rapid filtration reduce to varying degrees the bacterial content of water. However, in most cases these processes cannot assure that the water they produce will be bacteriologically safe. Disinfection by chemical addition will frequently be needed. In cases where no other methods of treatment are available or required, disinfection may be the only treatment process utilised before the water is supplied to consumers.

Disinfection of water provides for the destruction or the complete inactivation of harmful micro-organisms present in the water. It is carried out using physical or chemical means. The following factors influence the efficiency of the disinfection process (IRC, 1981):

- The nature and number of the organisms to be destroyed.
- The type and concentration of the disinfectant used.
- The temperature of the water to be disinfected (the higher the temperature the more rapidly will disinfection take place).
- The time of contact; (the disinfection process becomes more complete when the disinfectant remains for a longer period in contact with the water).

- The nature of water to be disinfected; (if the water contains particulate matter especially of a colloidal and organic nature, or other chemical components which react with the disinfectant used - i.e. exerts a demand, the disinfection process will be hampered).

- The pH (acidity/alkalinity) of the water; if chlorine is used, it is considerably more effective at pH 7 than at pH 9.

- Mixing; good mixing ensures proper dispersal of the disinfectant throughout the water, and so promotes the disinfection process.

The following methods may be used for disinfecting water for domestic use:

5.2.1 Boiling

The boiling of water for a few minutes does not necessarily give complete sterilization. Although the majority of bacteria and viruses are rendered harmless very rapidly, it is necessary to boil contaminated water for 10 minutes to ensure its full safety (Mann and Williamson, 1983). Care should be taken to store the boiled water in the same container in which it was boiled if possible, to reduce the possibility of recontamination of the supply from other contaminated containers.

Boiling for 10 minutes alters the taste of water and uses large amounts of fuel, which is often in very short supply. It is therefore not a feasible method for community water supplies. However, in emergency situations or where no alternative exists, the boiling of water may be the only feasible disinfection option.

5.2.2 Chlorination

The ability of chlorine and chlorine compounds to destroy pathogens in water quickly, and their wide availability, make them well suited for use as disinfectants. Their cost is moderate and they are, for this reason, widely used as disinfectants throughout the world.

Effective chlorination of water supplies has in many cases achieved a substantial reduction in those enteric diseases that are primarily water-related. Recent studies, still in progress, have raised the possibility that compounds formed when chlorine reacts with certain organic compounds in water, may cause cancer in man. Due to the number of variables involved no definite evidence is available so far. On the other hand, the disinfecting properties of chlorine are well established and, to date, far outweigh the suggested possible side effects when it is used to safeguard public health.

. *Products used for chlorination*

The following three products that are used for chlorination are readily available and are approved by the South African Department of Health and Population Development for use with drinking water.

- Chlorine gas
- Sodium hypochlorite (Jik, Javel, or other non-perfumed bleaches)
- Calcium hypochlorite
  (Calcium hypochlorite is commonly supplied in South Africa as HTH with 70 % dry granular chlorine).

Chlorine gas

Chlorine gas is normally used at water treatment works because it is cheaper than other chlorine products. The gas chlorination equipment is safe to use by properly trained persons, but special safety features should be included in the design of the chlorination building in which the gas cylinders are housed and in the installation of the equipment. Safety equipment, such as a gas mask, should also be provided. Operating personnel must be well trained in the correct use and precautions to be taken with chlorine gas. For this reason:

chlorine gas is not recommended for small water supplies because any gas leaks will result in a very dangerous situation with the serious injury or death of people located close to the treatment plant.

Where chlorine gas is used there should always be a trained person nearby who knows what to do in the case of a gas leak and can warn people to keep away. This is usually not possible with small water supplies.

Sodium hypochlorite liquid

Sodium hypochlorite is supplied as a liquid in 20 or 25 litre containers. It contains 15 % chlorine (150 g/*l* chlorine) when it leaves the factory, but the chlorine strength gradually reduces to less than 10 % strength during several months storage. This can be prevented by mixing the 20 litres of sodium hypochlorite liquid with 40 litres of water as soon as it is received. This will make 60 litres of sodium hypochlorite liquid which should have at least 4 % chlorine. This solution can be kept for several months without loss of strength provided that it is stored in a cool place.

Household liquid bleach, which can be bought in stores, contains sodium hypochlorite that has already been diluted to 3,5 % chlorine strength. Therefore bleaches can be used to disinfect drinking-water, although bleaches with perfumes, such as spice and pine, should not be used.

Calcium hypochlorite

Calcium hypochlorite is supplied as 70 % dry granular chlorine (trade name HTH). The chlorine strength remains about 70 % for several months provided that the compound is kept in a plastic or glass container with a lid and is stored in a cool place.

Dry chlorine tablets that are suitable for use with drinking water can also be bought. One such product available is the tablets which are specially made for use with the "Flowrite" chlorinator (described in Section 5.6).

Chlorine tablets that are sold in stores for use in swimming pools are generally not suitable for disinfecting drinking water.

These are usually called 'stabilized chlorine' tablets, which means that they contain a stabilizing chemical that could possibly be harmful to people's health if they are used in drinking water for a long period of time.

Dry granular chlorine can be dissolved in water, and a solution with 4 % chlorine strength can be made by mixing 60 g of chlorine (1/3 cup) with one litre of water. However, calcium hypochlorite contains a small amount of insoluble material which, although not harmful to drink, can cause problems in the operation of some chlorination equipment by blocking pipes or valves of dosing pumps. Therefore solutions made up from dry chlorine should only be used with equipment that has been designed to prevent these problems from happening.

Calcium hypochlorite can also cause a white crust (calcium carbonate) to be deposited inside the equipment. This can be removed by cleaning with dilute hydrochloric acid from time to time.

5.2.3 Methods of chlorinating containers of water

This section applies to the chlorination of containers of water, such as buckets, 25 litre containers, 200 litre drums and 1000 litre tanks. It is not necessary to purchase any chlorination equipment, but it is necessary to add a fixed amount of the chlorine to each container every time it is filled with water.

When using sodium hypochlorite, a small quantity of household bleach may be mixed into the water and then it is allowed to stand for at least two hours before drinking. The recommended quantities of bleach for different sized containers are given in Table 5.2. Alternatively, the same quantities of sodium hypochlorite can be used, provided that it is first diluted to 4 % chlorine strength as recommended above.

TABLE 5.2: QUANTITY OF HOUSEHOLD BLEACH RECOMMENDED TO CHLORINATE A CONTAINER OF WATER

Container size	Quantity of bleach	Chlorine dosage (mg/*l*)
10 l bucket	½ teaspoon (2,5 ml)	8,7
20 l container	1 teaspoon (5 ml)	8,7
200 l drum (44 gallons)	¼ cup (50 ml)	8,7
1000 l tank (1 m^3)	1 cup (200 ml)	7,0

Granular chlorine can also be used to chlorinate tanks of water. Ten grams (two medicine measuring teaspoons) of granules per 1000 litres of water will give a chlorine dosage of about 7 mg/*l*. It is recommended that the dry chlorine be first dissolved in water, poured into the tank and the tank then filled with water in order to properly mix the chlorine with the water.

Should the above doses result in a strong chlorine smell and taste in the water, the water may be relatively clean and the chlorine dose could be safely reduced to 50 or even 25 % of those recommended above. However, if there is a possibility of contamination from an ammonia containing stream (e.g. sewage effluent), the smell may be a result of the formation of ammonia-chlorine compounds which may not result in adequate disinfection having taken place. If this possibility exists, the water should first be analysed before reduced chlorine doses are used.

5.2.4 Methods of chlorinating small water supplies

The most suitable chlorination equipment to use depends on each particular case. However, the following points should be considered when choosing equipment:

- The person who is responsible for the equipment must be reliable and must be properly trained in how to operate and maintain the equipment correctly. The person must also make sure that there is always enough chlorine product available. This is very important because other people rely on this person to make sure that the water supply is always safe to drink.

- The chlorine product that the equipment uses must be readily available at a reasonable cost.

- The equipment must be properly designed, strong, easy to operate and require little maintenance. It must be made of materials that will not be corroded (eaten away) by chlorine.

- The equipment must be suitable for the particular use, taking account of the water flow rate, whether the flow rate is constant or varies, the quantity of water used per day, whether the chlorine is dosed into a pressurised pipeline and whether electricity is available.

- The equipment must always dose enough chlorine to disinfect the water, but not too much so that it gives the water a bad taste.

It is necessary to have a large water tank after the chlorinator so that there is enough time for the chlorine to react with the water before people drink the water. The size of this tank depends on the water flow rate, but it should have at least one hour's storage volume.

A number of different ways of dosing chlorine are set out in the CSIR booklet "Disinfection for small water supplies". Some of these are briefly described below.

a) Constant flow chlorinator

This chlorinator ensures a reasonably constant rate of flow of a solution of chlorine (m*l*/min) as the container of chlorine solution gradually empties. (Note that a container with only a hole or a tap at the bottom does not give a constant flow because the rate of flow gradually decreases as the container empties).

The constant flow chlorinator is not suitable for use in cases where the water flow rate varies. Also, the flow of chlorine solution does not stop automatically when the water flow stops. The chlorine solution should be dosed at a point where the water is turbulent, such as at the inlet into the storage tank, in order to mix the chlorine with the water.

The constant flow chlorinator is not sold ready made by any company, but it can easily be constructed by anyone who wants to use it (Fig 5.1).

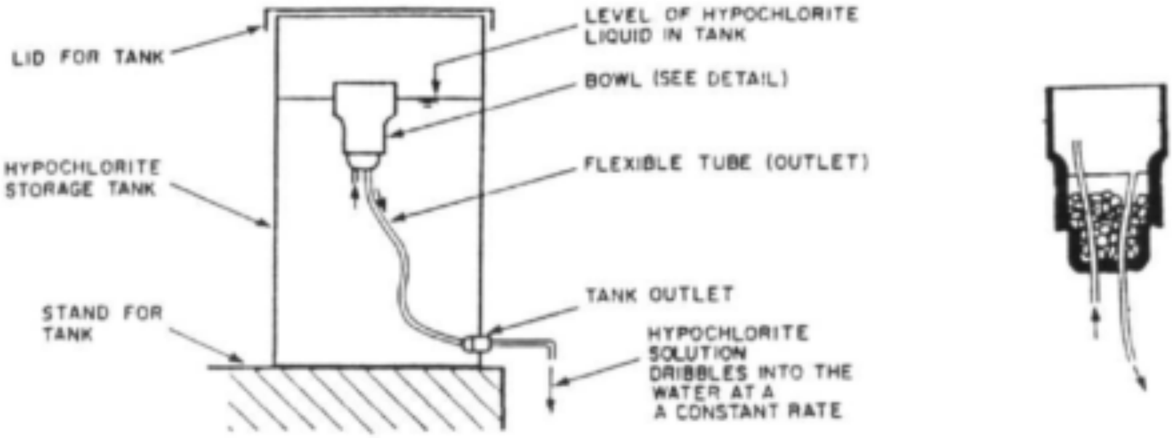


Figure 5.1 Constant flow chlorinator

b) Self-powered chemical doser

This equipment consists of a 150 litre plastic water tank and a 10 litre storage tank for sodium hypochlorite solution. Water flows into the 150 litre tank and, when it is full, a certain amount of hypochlorite solution is automatically poured into the water by a plunger. The 150 litre tank then automatically empties and the cycle starts again.

There are three plunger sizes available of 5, 10 and 20 ml maximum capacity each. The 20 ml plunger can dose up to 5 mg/*l* chlorine using 4 % hypochlorite solution, but it can also be adjusted to give a lower dosage if required. Assuming that it is adjusted to dose 4 mg/*l* chlorine, then the 10 litre hypochlorite storage tank is sufficient to chlorinate 100 000 litres of water before it needs to be refilled.

c) <u>Pot chlorinator</u>

A 12 to 15 litre earthen pot with two 6 mm holes is filled with a mixture of 0,5 kg dry chlorine and 3 kg coarse sand. After the top has been fitted with a water-tight cover (rubber or poly-ethylene), it is suspended one metre below the low water level of the well or water reservoir. The chlorinator will disinfect a well that yields up to 5000 litres per day for seven days.

There are a large number of other such simple devices which have been developed for use in rur . areas. The CSIR (Division of Water Technology) has produced a technical guide describing many of these.

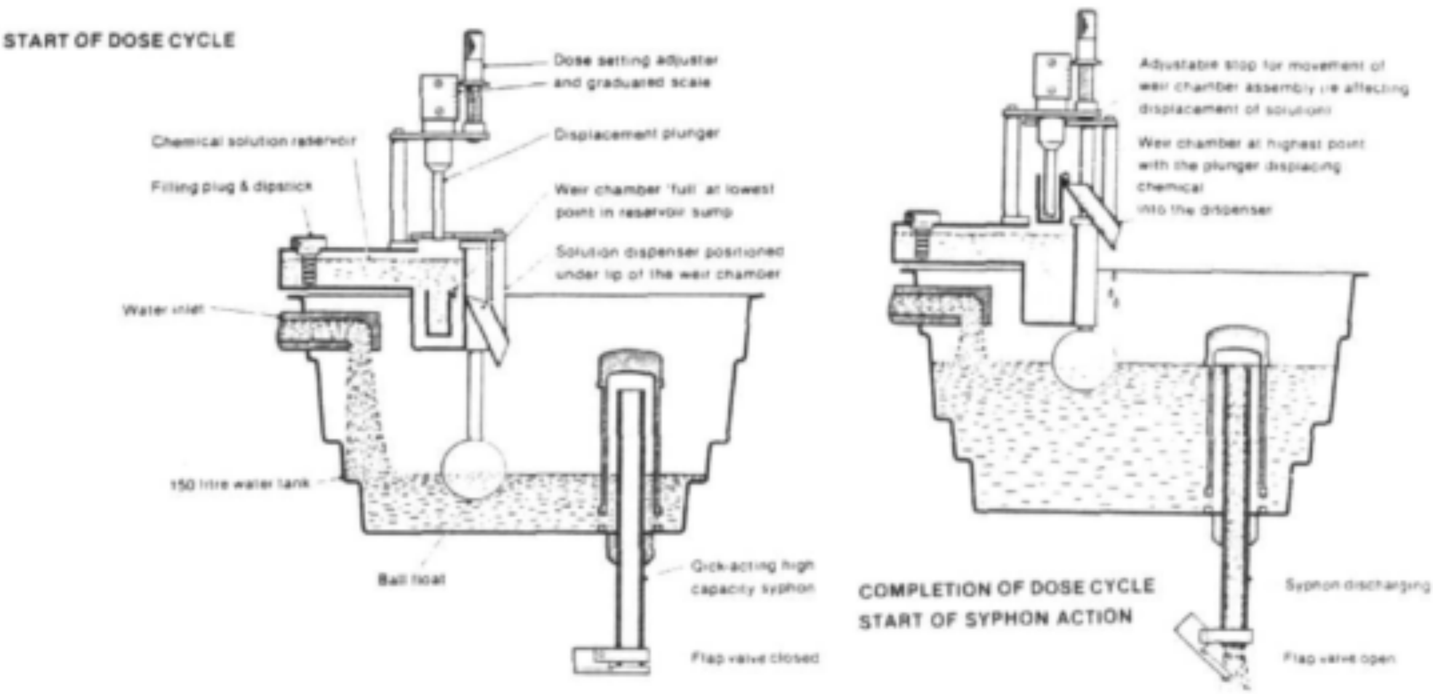


Fig 5.2 Self powered chemical doser

d) <u>Chlorine from salt</u>

Chlorine can be generated from salt on-site using a small chlorine generator. Simple generators employing carbon electrodes may be obtained from swimming pool chemical suppliers. More robust units employing specially coated electrodes and/or an ion selective membrane are commercially available. Units which operate with solar power for use in more remote areas are also available. The chlorine produced is in the form of sodium hypochlorite at concentrations from 0,1 to 5 %. Chlorine produced on-site may often be found to be cheaper than other forms of chlorine if the unit operates efficiently.

Chlorine produced in this way may be dosed into the water by the methods descried above for sodium hypochlorite.

e) Chlorfloc

Chlorfloc is a combination of solid calcium hypochlorite and a flocculating agent. When added to water and stirred it results in both the removal of suspended matter and the disinfection of the water. The suspended matter forms large flocs which settle out to the bottom of the container. The flocs may be removed by filtering through a cloth, or by withdrawing water selectively from the container. With chlorfloc it is possible to treat batches of water from one litre up to ten thousand litres at a time. For one to twenty litres the treatment takes place within ten minutes. For larger batches more time must be allowed for the settling of the flocs. Chlorfloc is a more costly form of chlorine than other forms, but due to its multiple function may be the best solution in some circumstances. A batch water treatment system based on chlorfloc called the "Water Maker" is available for treating larger volumes of water.

e) Flowrite bore hole chlorinator

The "Flowrite" chlorinator was specifically designed for use on bore holes equipped with a hand pump or a small electrically or mechanically driven pump. Within limits variable flows may be tolerated, and on/off operation does not pose a serious problem. Small dry chlorine tablets are inserted into a special container where they are slowly dissolved and the solution combined with the pumped water from the bore hole. The unit was developed in South Africa to treat contaminated ground water during the cholera outbreak in the years 1982 to 1984.

g) Klorman

This is an in-line chlorinator that uses calcium hypochlorite tablets. The tablets (Sanitabs) are available in a cartridge containing 700 grams of calcium hypochlorite. The chlorinator has no moving parts. The chlorine dosage is adjustable and may be set to meet the demand of any specific source. The only operation that is needed is the replacement of the cartridge when the tablets have been fully dissolved.

The Klorman is very easy to install. It is also very simple, durable and easy to operate, which makes it ideal for small and less developed communities. It does require a pressurized pipeline into which it can be connected, and the tablets are more costly than equivalent chlorine in other forms.

h) Other commercial dosing units

There are a number of other chlorine dosing units commercially available. A wide range of dosing pumps are obtainable, some of which may be driven by the hydraulic pressure of the water flow, i.e. they do not require electricity to operate.

5.2.5 Sunlight and ultraviolet rays

Sunshine beamed onto transparent water containers has been found to disinfect the water within a few hours. The wavelengths of the sunlight in the near ultraviolet region (315 - 400 nm) is responsible for the majority of the inactivation of the organisms. Therefore colourless glass or plastic containers are best, and green, orange, yellow and red containers should not be used. Thin walled round shaped containers of 1 - 3 *l* capacity are most suitable and should be exposed for the full day. The 2 *l* clear cooldrink containers which are sold country wide have been found to work well.

Water passed in close contact to ultra-violet (UV) lamps can also be disinfected by this means. The low pressure mercury lamps produce light rays of around 254 nanometers wavelength. The lamps are protected from the water by special quartz or plastic shields. Commercially available systems simple to install and operate. Lamps should last up to 8 000 hours of operation.

5.2.6 Storage

Storage of water can be a treatment process in itself. The number of faecal coliforms and faecal streptococci will be considerably reduced when the raw water is subjected to storage. Bilharzia risk is eliminated if the water is stored for a period of 24 hours or more. Storage also allows sedimentation to take place reducing the settleable solids content of the water. Storage, however, may promote algal growth in the water. Loss of water through evaporation often is another drawback. These effects will be minimized if storage tanks are covered which also would prevent dust, insects, air borne pollution and small animals from contaminating the stored water. If there is a high organic content in the water, certain micro-organisms may use this as food and actually increase during storage.

5.2.7 Disinfection of water supplies in emergency situations

Long-term measures for the provision of safe water for domestic use, aided by personal hygiene and health education, will greatly help protect and promote public health. However, natural disasters like cyclones, earthquakes and floods do occur and sometimes result in complete disruption of water supplies.

These situations call for measures to provide for the supply of safe water on an emergency basis. Often the prime need is for a supply of water which has been adequately disinfected. Although there is no single measure that is a panacea for all situations, the following may be useful to ensure a safe water supply depending upon local conditions and available resources:

i) When the regular water supply system is affected due to a disaster, top priority should be given to putting the system back into operation. Simultaneous action to tide over the situation should include a thorough search for all possible sources of water within a reasonable distance of the affected area. Water from private water supply systems and other sources may be transported by tankers to the points of consumption.

ii) After floods, when the water supply distribution system remains intact, the pressure in the pipelines should be raised so as to prevent polluted water from entering the pipes. As an additional measure the chlorination of the water at the treatment plants may be temporarily raised to a higher rate. However, high-dosage chlorination is recommended only in extreme circumstances or when cleaning out new pipes.

iii) When the only source of water is the contaminated rivers or streams which are in flood, attempts should be made to supply the people who have to use this water with a suitable disinfectant for treating small amounts of water. Chlorfloc or dry chlorine tablets with instructions on their use have been successfully used in the past.

5.3 Filtration

Filtration is a process for the removal of suspended matter (turbidity) from a water supply by means of a physical barrier. Two types of filtration mechanisms are generally used, these being:

. depth filtration - where the suspended matter is removed as the water passes through a deep layer of large particles, usually sand; and

. surface filtration - where the suspended matter is retained on the surface of a physical screen.

Depth filtration will generally have a greater capacity for the removal of suspended matter than surface filtration before cleaning of the filter is required. A number a different filters are used in water treatment, some more suitable for use in rural areas than others.

5.3.1 Slow sand filtration

Slow sand filtration is one of the most effective, simplest, and least expensive water treatment processes and is therefore particularly suitable for rural areas in developing countries. Essentially, this process differs from rapid sand filtration because of its biological nature, its high efficiency, and its suitability for village level operation and maintenance.

The basic process for slow sand filtration is as follows. Water slowly passes through a bed of fine sand at a rate of 0,1 to 0,3 $m^3/m^2/h$. In the process its quality is improved considerably by the filtering of turbidity and by a large reduction in the number of micro-organisms (bacteria, viruses, cysts). Soon after the filtration process begins, a filter skin forms on the surface of the bed. This filter skin, or 'Schmutzdecke', consists of retained organic and inorganic material and a wide variety of biologically active micro-organisms which break down organic matter. This biological activity and other treatment mechanisms extend through the upper layer of the sand bed, to a depth of about 0,4 m. Due to slow water movement and long retention time, slow sand filtration resembles the percolation of water through the subsoil, and the process effectively produces water comparable in quality to groundwater.

After the filter has been producing good quality water for several weeks, the filter skin gradually clogs and cleaning of the filter will be necessary. This is done by scraping off the top few centimetres of the filter bed and then restarting the filtration processes.

The application of slow sand filtration should be carefully evaluated when designing a water supply scheme. When surface water is more readily available than groundwater, slow sand filtration will frequently prove to be the simplest, most economical and reliable method of preparing safe drinking water. In many cases no further disinfection of the water will be required, although chlorination may be desirable as a back-up.

5.3.1.1 Mode of action of slow sand filters

The raw water is fed gently onto the filter bed and percolates downwards. Suspended matter in the raw water is deposited on the surface of the filter bed. This layer of organic and inorganic material increases the friction loss through the bed. The water level therefore rises gradually until it reaches a predetermined value, usually not more than 100 cm above the sand layer. The bed must then be taken out of service and cleaned.

The slow sand filter does not act by a simple straining process. It works by a combination of straining and microbiological action of which the latter is the more important. The mode of operation is complex, and the purification of the water takes place not only at the surface of the bed but for some distance below. Three zones of purification in the bed may be distinguished. Firstly, the surface coating, secondly the autotrophic zone existing a few millimetres below this, and thirdly the heterotrophic zone which extends some 30 cm into the bed.

1st stage = acts as an extremely fine-meshed strainer

2nd stage = decomposes plankton and the filtrate becomes oxidized by chemical reaction

3rd stage = microbiological filtration (also in the other two stages)

In order to guarantee good microbiological filtration, attention should be paid to ensuring:

- favourable conditions for biological activity in the water above the sand (including the schmutzdecke), i.e. no disinfectants;
- slow filtration rate ($0.1\ m^3/m^2/h$); and
- favourable raw water quality (pretreated by sedimentation only, no chemical additives like chlorine, etc.).

5.3.1.2 Performance of slow sand filters

The performance of slow sand filters is summarized in Table 5.3.

TABLE 5.3: PERFORMANCE OF SLOW SAND FILTERS

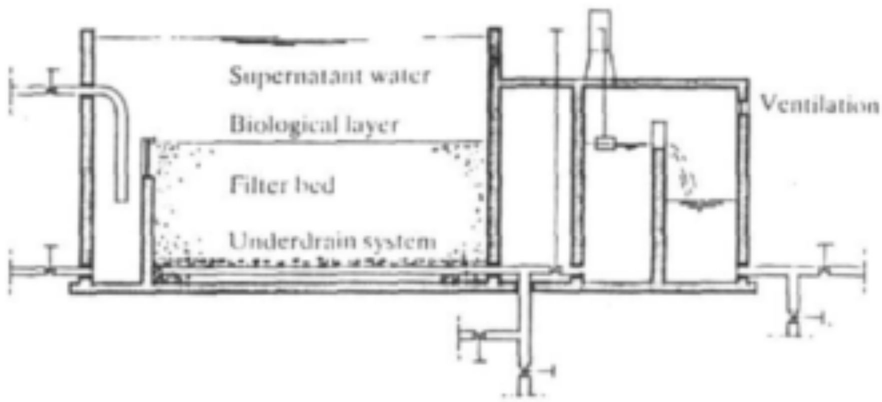
Water quality parameter	Purification effect of slow sand filtration
Colour	30 to 100 % reduction
Turbidity	Turbidity is reduced to less than 1 NTU
Faecal coliforms	Between 95 and 100 % and often 99 to 100 % reduction in the level of faecal coliforms
Cercariae	Virtual removal of cercariae of schistosoma, cysts and ova
Viruses	Virtually complete removal
Organic matter	60 to 75 % reduction in COD
Iron and manganese	Largely removed
Heavy metals	30 to 95 % reduction

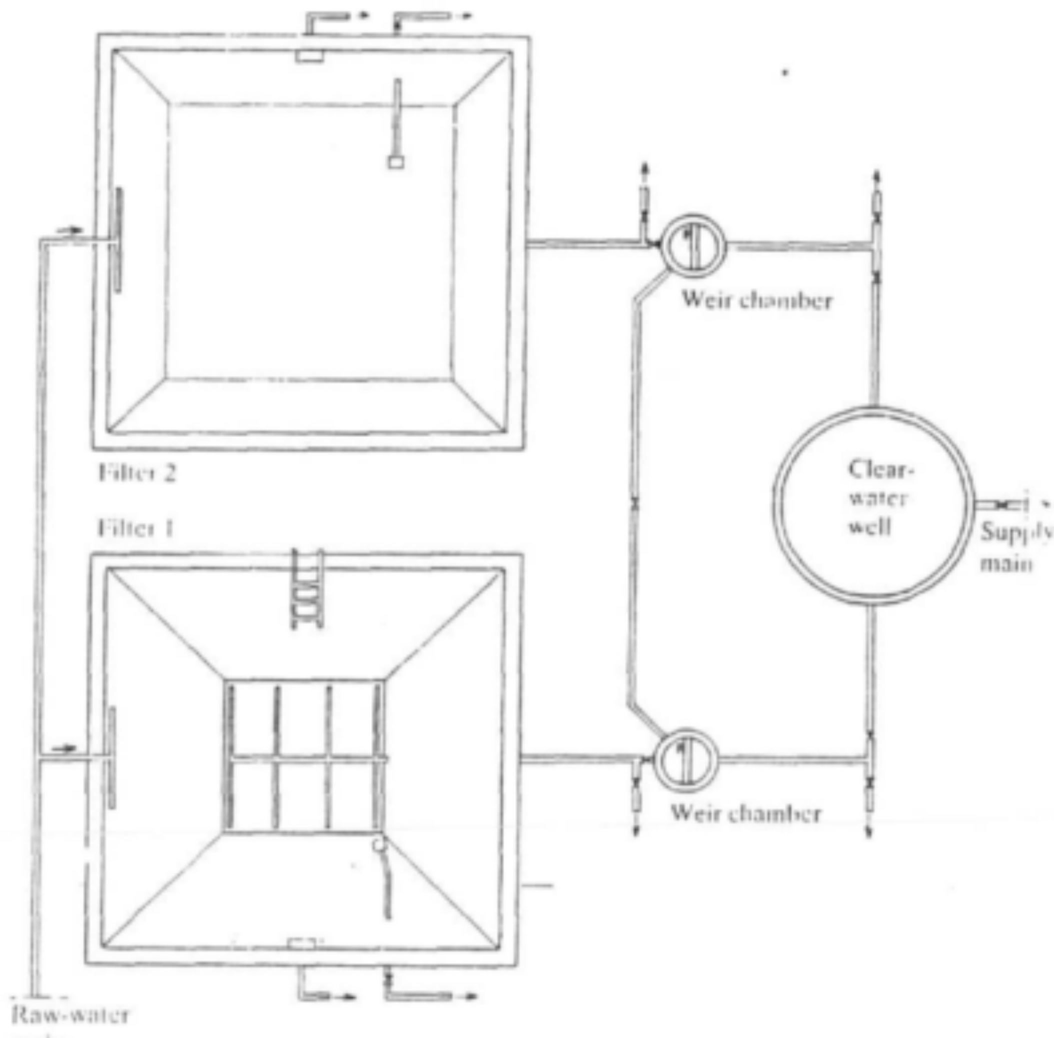
5.3.1.3 Design criteria for slow sand filters

TABLE 5.4: DESIGN CRITERIA FOR SLOW SAND FILTERS IN RURAL WATER SUPPLY

Design criteria		Recommended level
Design period		10 to 15 years
Period of operation		24 h/d
Filtration rate in the filters		0,1 to 0,2 m/h
Filter bed area		5 to 200 m² per filter, minimum of two units
Depth of sand layer:	Initial	0,8 to 0,9 m
	Minimum	0,5 to 0,6 m
Specification of sand:		
Effective size		0,15 to 0,30 mm
Uniformity coefficient		< 5 preferably below 3
Height of underdrains including gravel layer		0,3 to 0,5 m
Maximum height of supernatant water		1 m

Figures 5.4 and 5.5 show the basic components of slow sand filters.





Figures 5.4 and 5.5 : Basic components of slow sand filters

5.3.1.4 Advantages of slow sand filters

Quality of treated water

No other single process can effect such a combined improvement in the physical, chemical, and bacteriological quality of normal surface waters as that accomplished by slow sand filtration. The delivered water does not support aftergrowth in the distribution system, and no chemicals are added, thus obviating one cause of taste and odour problems.

Cost and ease of construction

The simple design of slow sand filters makes it easy to use local materials and skills in their construction. The cost of imported materials and equipment may be kept to almost negligible proportions, and it is possible to reduce the use of mechanized plant to the minimum and to economize on skilled supervision. Design is easier, little special pipework or equipment is required, instrumentation can be almost completely eliminated, and a greater latitude in the screening of media and the selection of construction materials can be permitted. Only when a high price has to be paid for land and when expensive superstructures are necessary for protection against low temperatures is the capital cost of slow sand filters likely to equal or exceed that of comparable rapid filters.

Cost and ease of operation

The cost of operation lies almost wholly in the cleaning of the filter beds, which may be carried out either mechanically or manually. In developing countries and elsewhere where labour is readily available, the latter method will be used, in which case virtually the whole of the operating cost will be returned to the local economy in the form of wages.

No imported chemicals or other materials are needed for the process, though in many cases chlorination is practised as an additional safeguard. However, chlorination would be equally necessary with any other form of treatment and, in general, the dosages required to disinfect water treated by slow sand filtration are less than those needed to disinfect water treated by other methods.

No compressed air, mechanical stirring, or high-pressure water is needed for back-washing, thus there is a saving not only in the provision of plant but also in the cost of fuel or electricity.

The operator of a biological filter requires far less training and skill than does his colleague in charge of a rapid gravity filter, and less supervision and support (e.g. laboratory testing of chemical quality) are called for. Slow sand filters automatically accommodate minor fluctuations in raw water quality, temperature, and climatic conditions and can stand short periods of excessive turbidity or demand without breaking down.

Conservation of water

In water-short areas, biological filters have the additional advantage of not requiring the regular flushing to waste of wash water. In the case of pressure or rapid gravity filters, which need cleaning every day, this wastage represents some 2 to 3 % of the total amount treated. Reclamation of the wash water may be practical in some places but represents an additional expense.

Disposal of sludge

Sludge storage, dewatering, and disposal are less troublesome with slow sand filters than with mechanical filters particularly when the latter contain chemical coagulants. Since the sludge from biological filters is handled in a dry state there is virtually no possibility of polluting neighbouring watercourses, and the waste material is usually accepted by farmers as a useful dressing for their land, the mixture of sand and organic matter being especially suitable for conditioning heavy clay soils.

5.3.2 Roughing filtration

Sometimes a more limited treatment than conventional rapid filtration can be used for treating the raw water. This can be accomplished by using gravel or plant fibres as filter material in a filter system. In the upflow mode, three layers would be used having grain sizes of 10 - 15 mm, 7 - 10 mm and 4 - 7 mm from the bottom upward, and with a simple underdrain system. This coarse ("roughing") filter will have large pores that are not liable to clog rapidly. A high rate of filtration, up to 20 m/h may be used. The large pores also allow cleaning at relatively low back-wash rates since no expansion of the filter bed is needed. The back-washing of roughing filters takes a relatively long time, about 20 to 30 minutes.

Horizontal flow-roughing filtration is a treatment process that is based mainly on sedimentation, although with time, biological activity may also play a role. A horizontal flow-roughing filter consists of a rectangular box, usually 1,0 to 1,5 m deep, with the raw-water inlet on one side and the outlet on the other. The box is usually divided into three compartments packed with crushed stones of graded sizes from coarse to fine (Figure 5.6). Design criteria are given in Table 5.5. Experience with these filters to date is limited, but promising. Although turbidity removal will depend on local conditions, particularly on the type of turbidity, it will generally be greater than 70 % and values of 90 % have been reported. The suspended solids removed from the raw water will slowly build up in the filter. After some time, this will reduce the efficiency of the filter and ultimately lead to complete clogging. Efficiency can be restored by fully opening the underdrain system, and flushing out the deposits by fast drainage and a water spray on the surface. Nevertheless, some solids may remain and make subsequent fast drainage less effective and it may therefore become necessary to clean the filter material after several years. Cleaning can be carried out by removing and washing the filter material, and then replacing it.

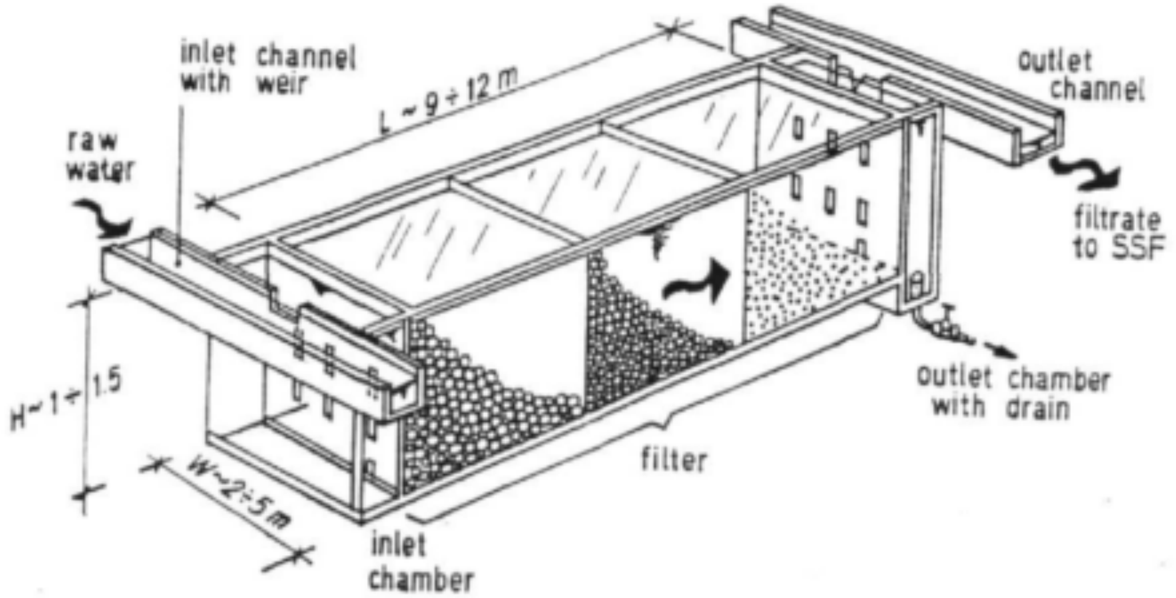


Figure 5.6 Horizontal roughing filter (HRF)

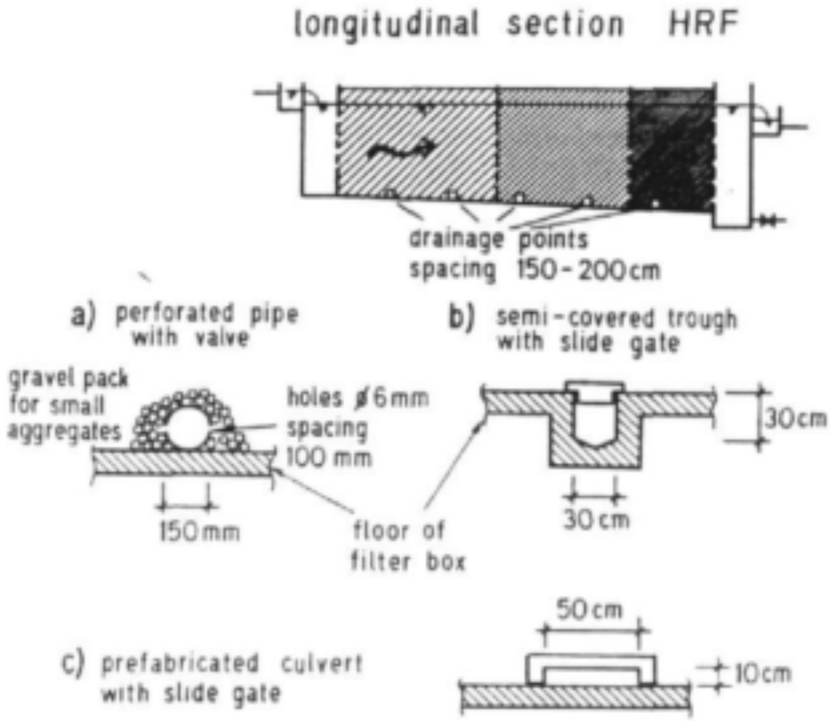


Figure 5.7 : Drainage systems for cleaning HRF media

TABLE 5.5: PRELIMINARY DESIGN GUIDELINES FOR HORIZONTAL FLOW-ROUGHING FILTERS

Parameter	Average suspended solid concentration in raw water	
	High (150 mg/*l*)	Medium (100-150 mg/*l*)
Horizontal flow (m/h)	0,5 - 0,75	0,75 - 1,5
Depth (m)	1,0 - 1,5	1,0 - 1,5
Width (m)	1,0 - 5,0	1,0 - 5,0
Length of filter media (m):		
First compartment (15 - 25 mm)	3,0 - 5,0	3,0 - 4,0
Second compartment (8 - 15 mm)	2,0 - 4,0	2,0 - 3,0
Third compartment (4 - 8 mm)	1,0 - 3,0	1,0 - 2,0

5.3.3 Rapid filtration

For rapid filtration, sand is commonly used as the filter medium, but the process is quite different from slow sand filtration. This is so because much coarser sand is used with an effective grain size in the range 0,4 to 1,2 mm, and the filtration rate is much higher, generally between 5 and 15 $m^3/m^2/h$ (120 to 360 $m^3/m^2/day$). Due to the coarse sand used, the pores of the filter bed will be relatively large and the impurities contained in the raw water will penetrate deep into the filter bed. Thus the capacity of the filter bed to store suspended impurities is much more effectively utilized and even very turbid river water can be treated with rapid filtration. For cleaning a rapid filter bed, it is not sufficient to scrape off the top layer. Cleaning of rapid filters is effected by back-washing. This means directing a high-rate flow of water back through the filter bed whereby it expands and the sand is scoured. The back-wash water carries the trapped suspended solids out of the filter. The cleaning of a rapid filter can be carried out quickly; it need not take more than about one half an hour. It can be done as frequently as required, if necessary, two or three times per day. This means that a rapid sand filter can treat water in which the suspended solids levels may vary from low levels to very high levels without adversely affecting the production of clean water.

5.3.3.1 Applications of rapid filtration

There are several different applications of rapid filtration in the treatment of water for drinking water supplies.

In the treatment of groundwater, rapid filtration is used for the removal of iron and manganese. To assist the filtration process, aeration is usually required as a pretreatment to form insoluble compounds of iron and manganese.

For water with a low turbidity such as is frequently found in lakes and sometimes in rivers, rapid filtration should be able to produce a clear water without any pretreatment. The final water will, however, still contain pathogenic bacteria and viruses. A final treatment such as chlorination is then necessary to obtain a bacteriologically safe water.

In the treatment of river water with high turbidity, rapid filtration may be used as a pretreatment to reduce the load on the following slow sand filters, or it may be used for treating water that has been partially clarified by coagulation, flocculation and sedimentation. Again final chlorination is required to ensure that the water is bacteriologically safe.

5.3.3.2 Types of rapid filters

Rapid filters are usually built open to the atmosphere with the incoming water passing down through the filter bed by gravity. The sand bed is usually of a homogeneous material, although some stratification of the material will take place during filter backwashing (the lighter fine sand settling on the top, and the heavier coarse sand settling first on the bottom.

For certain operating conditions, other rapid filters than the open gravity type are better suited. The most important are: pressure filters, upflow filters and multiple-media filters.

Pressure filters are of the same construction as gravity-type filters but the filter bed together with the filter bottom is enclosed in a watertight pressure vessel. The driving force for the filtration process is the water pressure applied to the incoming feed water which can be so high that almost any desired length of filter run is obtainable. Pressure filters are commercially available as complete units. They can be used in modular configurations so that upgrading at a later stage is very simple. The pressure drop across the filters is generally higher than that across a gravity filter, requiring appropriate pumps unless sufficient static head is available.

Upflow filters provide for a coarse-to-fine filtration process. The coarse bottom layer of the filter bed filters out the major part of the suspended impurities, even from a turbid raw water, without an excessive increase of the filter-bed resistance due to the large pores. The overlaying fine layers have smaller pores but here also the filter resistance will increase only slowly as only fine impurities make their way through to the upper layers. Upflow filters are therefore reputed to have a much larger dirt-holding capacity than downflow filters.

In upflow filters, sand is used as the single filter medium. They are frequently used for the pretreatment of water that is further purified by down-flow gravity-type rapid filters or by slow sand filters. In such cases, upflow filters can give excellent results and may be well suited for use in small treatment plants. However provision must be made for adequate back-washing as the dirt must be carried upwards through the whole filter bed to remove it.

One drawback is that the allowable resistance across an upflow filter is not more than the submerged weight of the filter bed. With sand as the filter material, the available resistance head is about equal to the thickness of the bed. For very turbid river water the length of the filter run and the allowable rate of filtration are thus very limited. In addition the suspended matter in the raw feed water may clog and block the nozzles which distribute this water to the base of the filter.

Multiple-media filters are gravity-type downflow filters with the filter bed composed of several different materials that are placed coarse-to-fine in the direction of flow. For small-size rapid filters it is common to use only two materials in combination: 0,3 to 0,5 m of sand with an effective size of 0,4 to 0,7 mm as the under layer, topped by 0,5 to 0,7 m of a material with a specific gravity lighter than that of sand and with an effective size of 1,0 to 1,6 mm. Suitable materials for the upper layer include anthracite, pumice and crushed coconut husks. The filter therefore has the advantage of a coarse to fine configuration as with an upflow filter, but without the limitations of upflow filters. As a final treatment multiple-layer filters can give excellent results and, when suitable materials are available locally, application in small treatment plants is well worth considering.

Examples of the above types of filters are shown in Figures 5.8 to 5.11.

5.3.3.3 Autonomous self-backwashing gravity filter

An innovative design of a gravity sand filter is the autonomous gravity filter which is being used in both rural areas, and in some industrial water treatment processes. The advantage of the autonomous filter is that it can be left to operate on its own without concern of the filter blocking. Once the head loss across the filter increases to a fixed value, the filter automatically goes into a backwash cycle and cleans the filter bed. No electricity is required for the filter to operate valves, etc., and because of the simple, robust design, very few problems due to breakdowns are likely to be experienced. Figure 5.12 illustrates the filter.

5.3.3.4 Design considerations

For the design of a rapid filter, three parameters need to be selected:

. grain size of the filter material
. depth of the filter bed
. rate of filtration

These parameters may often be more easily selected by comparison with existing plants treating the same or a comparable water, or by carrying out pilot scale experimental evaluations. Design equations do exist but cannot always be relied upon because of the wide difference in the quality and type of solids found in the waters to be treated.

Example : Once a filtration rate has been selected, the size and number of filters can be determined. Assuming a water use of 40 litres/person/day, the required water for 10 000 people would be 400 m^3/day or 40 m^3/hour for a 10-hour day operation. With a filtration rate of 5 m/h this calls for 8 m^2 of filterbed area. This may be provided by three circular filters of 2.5 m diameter each, giving a filter area of 4.9 m^2 per filter with one filter as reserve.

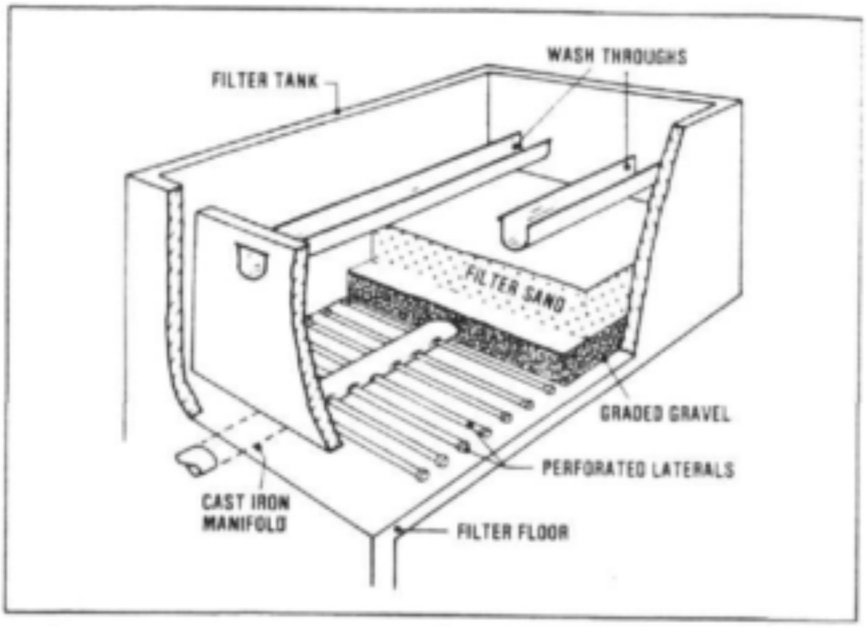
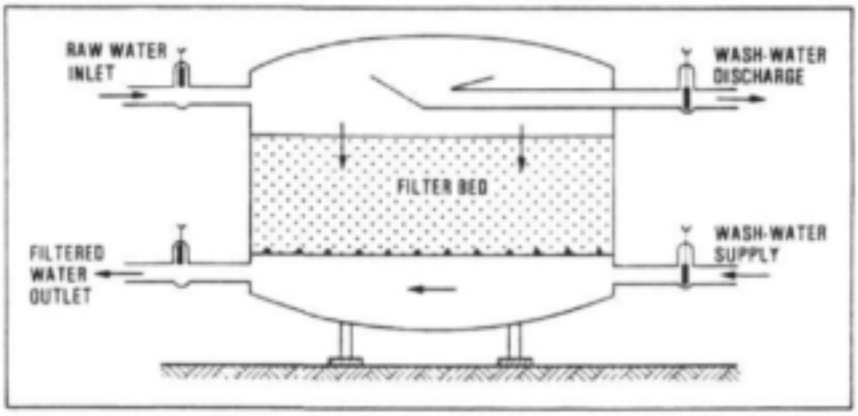
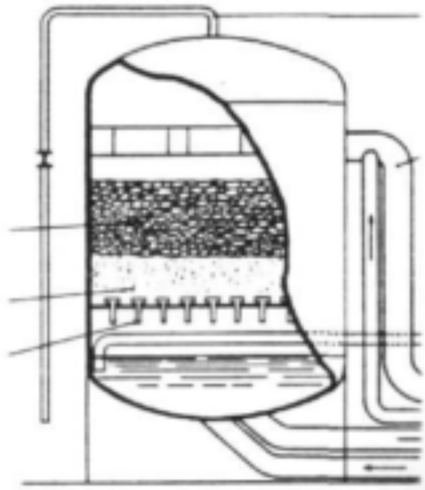


Figure 5.8 : Rapid gravity downflow filter



(a)



(b)

Figure 5.9 : Pressure filters a) single media, b) dual media

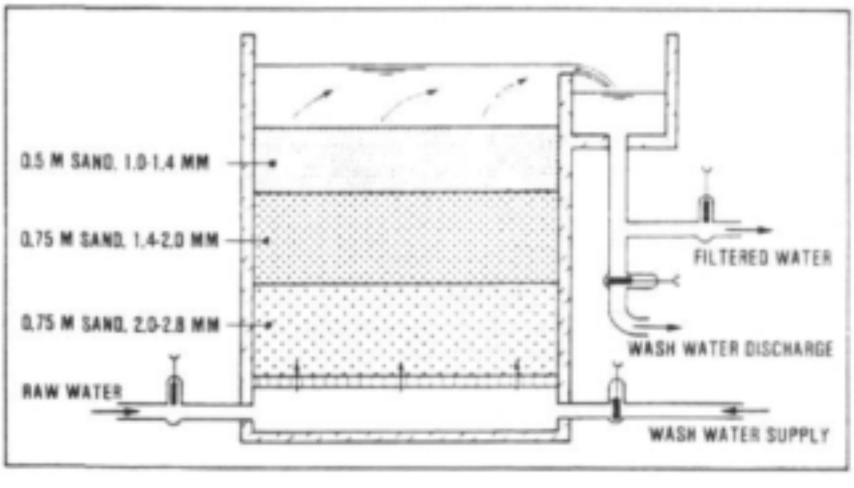


Figure 5.10 : Upflow filter

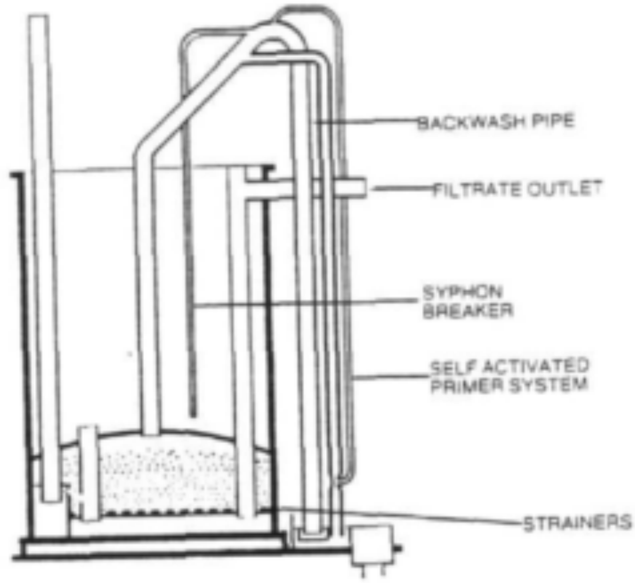


Figure 5.12 : Autonomous rapid gravity filter

Grain size and depth of filter media: The underdrain system is usually made of perforated laterals covered with graded layers of gravel, broken stones or hard bricks chipped to the desired size. The sand to be used in the main filter bed should be graded using suitable sieves to give a uniformity coefficient of 3.0 or less. Effective grain sizes for sand filters used as prefilters or final filters should be 0,8 mm to 1,2 mm; and 1,0 mm to 1,5 mm for iron and manganese removing filters. For prefilters and final filters the sand bed thickness should be 1,0 to 1.2 m and for iron and manganese removing filters 1,5 m. In the event that sand cannot be obtained, similar materials may be used, such as crushed stones or bricks, crystalline calcium-carbonate, dolomite, etc.. These should be graded to a size about 40 % larger than the sizes mentioned above. In some instances burned rice husks and crushed coconut shells have given acceptable results. Before the filter is commissioned it should be back-washed for about half an hour to clean the filter material.

Backwashing : If possible, filtered water should always be used for back-washing. Filtered water can be stored in an elevated tank for this purpose, or the effluent from other operating filter units of the filtration plant can be used directly ("self-wash arrangements"). The velocity of the upward water flow should be sufficient to produce an expansion of the filter bed. For a filter bed of sand (S.G. = 2,65 g/cm^3) typical back-wash rates giving about 20 % expansion are:

Temp (°C)	effective grain (mm)	0,4	0,5	0,6	0,7	0,8	0,9	1,0	1,1	1,2
		(back-wash rate $m^3/m^2/h$)								
10		12	17	22	28	34	40	47	54	62
20		14	20	26	33	40	48	56	64	73
30		16	23	30	38	47	56	65	75	86

These values should be used with care and if possible, pilot tests should be carried out to confirm the required flow rates.

Filter rate control is normally achieved by one of three ways:

- Constant rate filtration whereby each filter has an individual rate controller designed to maintain a constant flow through the filters by reducing the outlet pressure as the head-loss increases;

- Variable inflow or outflow evenly distributed between filters by flow splitting, whereby each operating filter receives the same flow of water, but the total volume fed to the filters may vary;

- Declining rate filtration whereby the rate of filtration decreases as the head loss across the filters increases.

Declining rate filtration design is much simpler than for controlled-rate filters and is certainly worth considering for small water treatment plants in developing areas.

5.3.4 Surface filtration with ceramic filters

There are several types of ceramic filters, such as pressure filters, non-pressure filters and filter pumps, and there is a wide range of ceramic media having different pore sizes. The heart of any of these is the filter candle, and the method of getting water through the candle is only a matter of convenience. Only clean water should be used with ceramic filters, otherwise, with cloudy or turbid water, the candles clog very quickly.

Coarse-grained candle filters are useful in removing suspended matter, helminth ova, cercariae, and cysts. They may only partially be effective in removing the smaller disease organisms, and consequently water should always be chlorinated or otherwise disinfected after passage through a coarse-grained or industrial-type filter.

Porcelain filters will remove all disease organisms usually found in drinking water except for viruses. Since viruses cannot survive for prolonged periods without a host, the filtered water can be rendered safe by storage over a period of time (at least 24 hours). After a prolonged use, however, the bacterial growth will penetrate and pass through the filter. This breakthrough can generally be prevented by cleaning and boiling the filter at least once a week, even if the filter does not clog. If a filter gets coated or clogged, it should be scrubbed under running water with a stiff brush free from soap, grease, or oil, and then boiled for 15 to 20 minutes.

Filter candles can be mounted in a gravity-type filter, which consists of two reservoirs with the candle or candles attached to the upper one. Water is simply poured in at the top, trickles through the ceramic candles, and is stored for use in the lower compartment. Another mounting is made where piped water is available under pressure. The candle is mounted in a pressure case which is attached directly to the water system, filtered water being drawn from the filter as needed. A third type is fitted with a hand pump. The suction tube is put into a vessel of water, and the pump is operated like a bicycle pump, the filter candle being inside. The filtered water is discharged through a second tube. Any of these systems is satisfactory if suitable filter candles are selected.

5.4 Coagulation and flocculation

The substances that frequently are to be removed by coagulation and flocculation, are those that cause turbidity and colour. Turbidity may result from soil erosion, algal growth or animal debris carried by surface runoff. Colour may be imparted by substances leached from decomposed organic matter, leaves, or soil such as peat. Both turbidity and colour are mostly present as colloidal particles. Colloidal particles are midway in size between dissolved solids and suspended matter. Colloids are kept in suspension by electrostatic repulsion and hydration.

The electrostatic repulsion between colloidal particles effectively cancels out the mass attraction forces that would otherwise bring the particles together. Certain chemicals (called coagulants) have the capacity to compress the double layer of ions around the colloidal particles which reject neighbouring particles. This reduces the electrostatic repulsion, and thus enable the particles to come together and join (i.e. to flocculate). The flocs so formed can grow by joining with more and more particles in the same way. When the flocs are of a sufficient size and weight they can be removed by settling or filtration.

Generally, water treatment processes involving the use of chemicals are not so suitable for small community water supplies, and should be avoided if possible. Chemical coagulation and flocculation should only be used when the required water quality cannot be achieved with another treatment process which requires no chemicals (for example slow sand filtration).

Coagulation and flocculation will usually be required when the suspended solid load would cause filters to block too rapidly, when space limitations or when flocculation facilities have already been provided and preclude the use of slow sand filters, or when the colloidal matter present in the water is too fine to be removed even by slow sand filters. Some plants have been designed to dose coagulants only when the incoming turbidity is greater than some predetermined value.

5.4.1 *Coagulants*

Alum (aluminium sulphate - $Al_2(SO_4)_3.nH_2O$ - is by far the most widely used coagulant, but iron salts [e.g. ferric chloride - $FeCl_3$, or ferric sulphate - $Fe_2(SO_4)_3$] can be used as well, and in some instances have advantages over alum. A significant advantage of iron salts over aluminium is the broader pH range for good coagulation. Thus, in the treatment of soft coloured waters where colour removal is best obtained at low pH's, iron salts may be preferred as coagulants. Iron salts should also be considered for coagulation at high pH's, since ferric hydroxide is highly insoluble in contrast to aluminium salts which form soluble aluminate ions at high pH's. Alum is mostly used for coagulation at medium pH's (6,0 to 7,5). Synthetic organic polyelectrolytes have become available as coagulants but are generally not economical for small water supply systems, nor are they always readily available.

For good coagulation, the optimal dose of coagulant should be fed into the water and properly mixed with it. The optimal dose will vary depending upon the nature of the raw water and its overall composition. It is not possible to compute the optimal coagulant dose for a particular raw water. A laboratory test called the "jar test" is generally used for the periodic determination of the optimal dose.

5.4.2 *Natural coagulants*

People have traditionally used both plants and soil materials for domestic treatment of highly turbid water. For example, horse beans, lentils and helba have been used as coagulants. Also used are ful masri (*Faba vulgaris*), doleb (palmfruits of *Borasses* sp.) and oshar shrub (*Calotrpis procera*), while ground nuts are being used with a clarifying clay soil (these are powdered seeds of *Moringa oleifera*). The juice from the leaves of *Moringa oleifera* has also been found to inhibit the growth of *E. coli*, *Micrococcus pyogenes*, and *Bacillus subtilis*. This has been attributed to the absorption capacity of the montmorillonite composing the material.

The drawback of using these coagulants is the ignorance about their constituents and their general effect on hygiene, as well as their scientific application, i.e. dose required, pH influence, degree and time of mixing, etc. which are not very well known.

5.4.3 *Rapid mixing*

Rapid mixing aims at the immediate dispersal of the entire dose of chemicals throughout the mass of the raw water. To achieve this, it is necessary to agitate the water violently and to inject the chemical in the most turbulent zone, in order to ensure its uniform and rapid dispersal. The requirement for rapid mixing is based on the property of the coagulant which results in very rapid hydrolysis of the coagulant (within a few seconds). Coagulants which have hydrolysed are less available to destabilise the colloids which the treatment process is aimed at removing. The destabilization of colloids is also very rapid, and hence optimum results are obtained when the coagulant is mixed with the water very rapidly (within one or two seconds).

Many devices are used to provide rapid mixing for the dispersal of chemicals in water. Basically, there are two groups, hydraulic rapid mixing and mechanical rapid mixing.

5.4.3.1 Hydraulic rapid mixing

For hydraulic rapid mixing, arrangements used are: channels or chambers with baffles producing turbulent flow conditions, overflow weirs, and hydraulic jumps. Utilization of a hydraulic jump downstream of a Parshall flume is one of the most practical methods used in developing areas. This provides the additional advantage of flow measurement in the Parshall flume. Rapid mixing may also be achieved by feeding the chemicals at the suction side of pumps. With a good design, a hydraulic mixer can be as effective as a mechanical mixing device.

5.4.3.2 Mechanical rapid mixing

With mechanical mixing the power required for agitation of the water is imparted by impellers, propellers or turbines. Generally, mechanical rapid mixers are less suitable for small treatment plants than hydraulic ones since they require a reliable and continuous supply of power, as well as regular maintenance.

5.4.4 *Flocculation*

Flocculation is the process of gentle continuous stirring of coagulated water for the purpose of forming larger flocs through the aggregation of the minute particles present in the water. It is thus the conditioning of water to form flocs that can be readily removed by settling or filtration. The efficiency of the flocculation process is largely determined by the number of collisions which can be induced between the minute coagulated particles per unit of time. As for rapid mixing, there are both mechanical and hydraulic flocculators.

5.4.4.1 Mechanical flocculators

Here the stirring of the water is achieved with devices such as paddles, paddle wheels or rakes.

5.4.4.2 Hydraulic flocculators

Again hydraulic systems are to be preferred to mechanical ones. The flow of the water is agitated by small hydraulic structures which cause a stirring action. Typical examples of hydraulic flocculators are channels with baffles, flocculator chambers placed in series, gravel bed flocculators, and hydraulic jet mixer type flocculators. Baffled channels are the most common and can be either the over-and-under type or round-the-end type. Figure 5.13 shows the baffled channel type flocculator design.

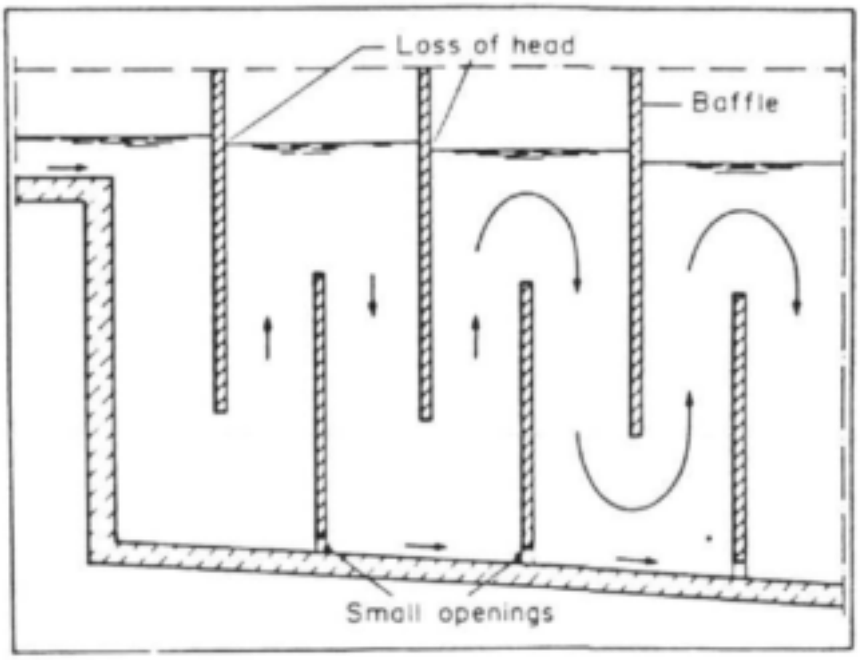


Figure 5.13 : Baffled channel type hydraulic flocculator

5.5 Sedimentation

5.5.1 *Process description*

Sedimentation is the settling and removal of suspended particles which takes place in static or slow flowing basins. Turbulence is negligible and particles having a mass density (specific weight) greater than that of water will settle to the bottom of the settling basin.

5.5.2 *Types of sedimentation tanks or settlers*

Two basic types of sedimentation processes are generally used: the horizontal flow type and the upflow clarification type. Upflow type clarifiers work well under conditions of relatively constant hydraulic loadings and raw water quality. Horizontal flow type sedimentation tanks are more tolerant to shock hydraulic and water quality loads, mainly due to a longer detention time. Operation and maintenance costs are also low.

5.5.2.1 Horizontal flow sedimentation tanks

Horizontal flow settling tanks include both the rectangular type where the average horizontal flow rate remains constant along its full length, and the radial type where the horizontal flow decreases as the water flows towards the outer periphery. Common design criteria for horizontal flow sedimentation tanks are:

surface loading :	20 to 80 $m^3/m^2.d$
effective water depth :	3 m minimum
mean flow velocity :	0.25 to 0.5 m/min
detention time :	2 to 4 hours

5.5.2.2 Tilted plate and tube settlers

An improvement in settling efficiency can be obtained by the installation of extra bottoms (trays) in a settling tank. The space between such trays being small, it is not possible to remove the sludge deposits manually with scrapers. Hydraulic cleaning by jet washing would be feasible but a better solution is the use of self-cleaning plates. This is achieved by setting the plates at a steep angle of 40° to 60° to the horizontal. The most suitable angle depends on the characteristics of the sludge which will vary for different types of raw water. The slope must be steep enough to allow the sludge to accumulate and then slide down the sloping plate. Such installations are called tilted plate settling tanks.

Instead of tilted plates, closely packed tubes may be used. These can easily be made of PVC pipes, usually of 3 to 5 cm internal diameter and sloping about 60° to the horizontal. For large installations commercially available tube models can have merit. A low cost alternative is the use of sheet plastic which has been welded at regular intervals to form a matrix of linked tubes. When expanded a lightweight, flexible sloping tube matrix results.

The possibility of increasing the efficiency of a tank through the installation of tilted plates or tubes may be used with great advantage for raising the capacity of existing settling tanks. Where the available tank depth is small (less than 2 m), the installation of the tilted plates or tubes is likely to meet with problems. In deeper tanks, they can be very advantageous.

In considering the expansion of existing facilities by the addition of tilted plates or tubes, it is important to remember that more sludge will be generated and so additional removal facilities may be required. Inlet and outlet pipe sizes and weir capacity should also be checked to see if they can carry the increased loading.

5.5.3 *Design of sedimentation tanks*

Sedimentation tanks are designed to reduce the velocity of the water flow so as to permit suspended solids to settle out of the water by gravity.

The raw water (of rivers) contains impurities of three physical kinds:

- Particles large enough to be strained out of the water, or that will settle gravitationally in still water (sedimentation);

- Particles of microscopic or colloidal form that will not settle in still water and are too small to be strained out (filtration or flocculation and sedimentation is required to remove these substances);

- Substances held completely in solution, i.e. dissolved in the water, which can be removed by chemical treatment only.

5.5.3.1 Factors affecting sedimentation efficiency

Factors which affect the efficiency of the sedimentation process include the following:

- settling velocity
  - mass density of suspended particle
  - shape of suspended particle
  - mass density of the fluid
  - viscosity of the fluid

- drag force
  - shape of suspended particle
  - velocity of the fluid
  - viscosity of the fluid
  - mass density of the fluid

- concentration of suspended solids in the fluid (settling hindered by wall effect)

The only factor that is altered by sedimentation is the fluid velocity. The smaller the size of the particles to be removed, the smaller the velocity of the fluid needs to be. The reduction in flow velocity required depends on the nature of the particles and the required efficiency of sedimentation (e.g. gritty, granitic or volcanic sediments, being heavier, need less flow velocity reduction to deposit them than fine lateritic topsoils).

The efficiency also depends on the design of the sedimentation tank:

- inlet and outlet must be constructed so that short-circuiting is prevented;

- agitation of settled solids in the sludge zone must be prevented.

The efficiency of a sedimentation basin will affect the processes following the basin, e.g. poor performance of the sedimentation process will affect the performance of any sand filters following.

5.6 Oxidation

Oxidation is utilized for various purposes in water treatment. In particular oxidants are used to remove or destroy undesirable tastes and odours, to aid in the removal of iron and manganese, for disinfection, and to help improve clarification and colour removal. Oxygen, chlorine and potassium permanganate are the most frequently used oxidizing agents, and the use of each is discussed below.

For small water treatment systems, it is recommended that chlorine be considered before other oxidants since chlorine will normally be used for disinfection too. If the use of chlorine for oxidation is not practical, then the use of air or potassium permanganate could be evaluated on an economic basis. Aeration requires only a capital investment and perhaps ongoing pumping costs, as opposed to ongoing chemical costs for chlorination and the use of potassium permanganate. Generally, aeration is preferred above potassium permanganate for oxidation unless high levels of manganese are to be removed. In that case, the use of potassium permanganate is usually necessary. Furthermore, if intermittent tastes and odours are a problem, potassium permanganate is preferred to aeration from an economic point of view. Chemical feed equipment for dosing potassium permanganate requires a smaller capital expenditure than aeration equipment. In addition, the chemical oxidant would be used only on an intermittent basis, so operation and maintenance costs would be at a minimum.

Oxidation is recommended as a treatment process for hydrogen sulphide and odour removal, and as an aid in iron and manganese removal. As a treatment step, oxidation should be applied before the other treatment steps of flocculation, settling, and filtration, as any oxidised products will then be removed from the water in these steps. One of the most common needs for oxidation in rural water supplies is for the removal of iron from borehole water. This can be accomplished by simple aeration procedures whereby the water passes over a charcoal bed before being filtered. The aeration procedure converts the iron into its insoluble form so that it can then be removed in the filtration step. In areas where the pH of the water is very low, it will be necessary to add a chemical for pH correction in addition to the aeration step to ensure the efficient removal of the oxidised iron.

5.6.1 *Chlorine* (see also section 5.2)

Chlorine is a strong oxidant, and is used to treat the raw water entering a treatment plant for the oxidation of unwanted organics, the oxidation of iron and manganese, and for the inactivation of algae. The systems which can be employed for dosing chlorine are the same as those for dosing chlorine as a disinfectant.

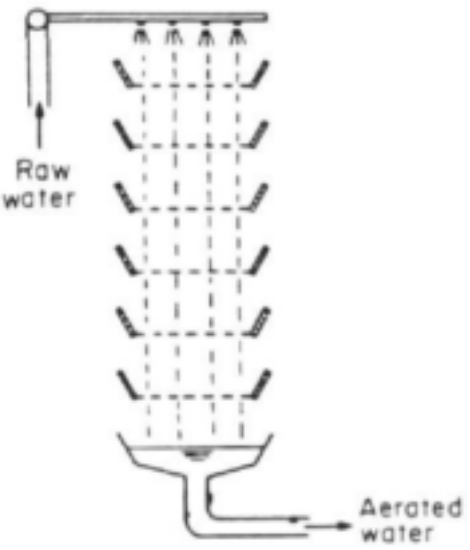
Chlorine does at times form undesirable by-products in its reaction with certain organics. By-products of concern are the ones which give rise to unpleasant tastes in the water, and the possible cancer causing chlorinated organics, particularly trihalomethanes (THMs). The reaction of chlorine with phenols results in the formation of chlorophenols, some of which impart an unpleasant taste to the water. The reaction of chlorine with natural humic substances, as well as algal byproducts, can result in the formation of THMs.

5.6.2 *Aeration*

Aeration is often used to treat groundwater having too high an iron and manganese content. The atmospheric oxygen brought into contact with the water through aeration will react with the dissolved ferrous and manganous compounds, changing them into insoluble ferric and manganic oxides or hydroxides. These can then be removed by sedimentation or filtration. It should be noted that the oxidation of the iron and manganese compounds is not always readily achieved, especially when the water contains organic matter. Chemical oxidation may be required to boost the process in such situations.

The following methods have been used to achieve an intimate contact between water and air as needed for aeration:

- waterfall aerators whereby the water is dispersed through the air in thin sheets or fine droplets. A number of different systems for achieving this are in use (figure 5.14);
- bubble aerators whereby the water is mixed with fine air bubbles which have been introduced into the water under pressure or by suction (figure 5.15);



Multiple tray aerator

This type of aerator consists of 4-8 trays with perforated bottoms at intervals of 30-50 cm. Water enters the upper tray and trickles down at a rate of about 0.02 m^3/sec per m^2 of tray surface. Trays are made of any suitable material, (small diameter pipes, parallel wooden slats, asbestos cement plates, etc.). The trays can be filled with gravel or charcoal to provide finer dispersion of the water.

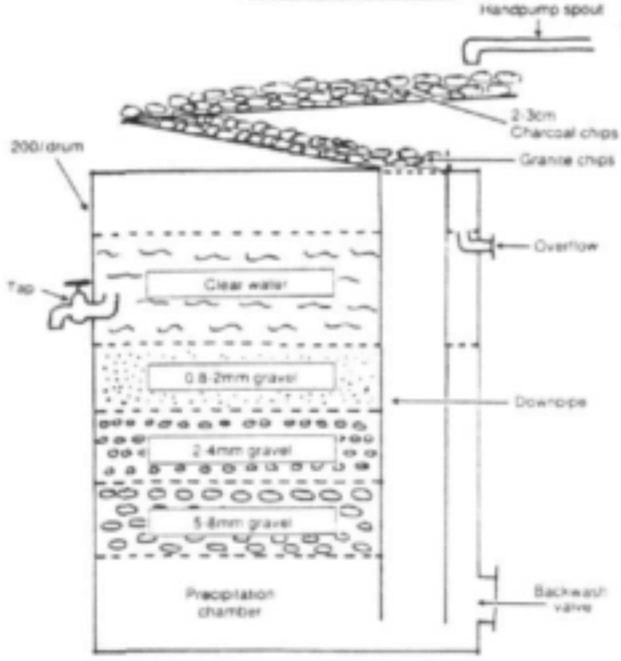


Figure 1 Section through the iron removal plant

Simple 200 l drum type aeration/iron removal plant for handpumps

Figure 5.14 : Water-fall aerators for oxidation by aeration

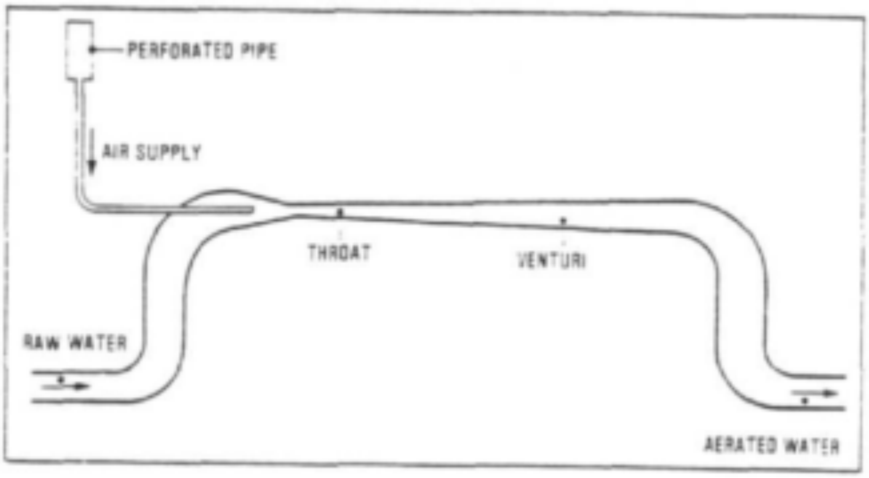


Figure 5.15 : Bubble dispersion type aerator

5.6.3 *Potassium permanganate*

Potassium permanganate has frequently been used for oxidation in water treatment processes. It is a powerful oxidising agent, and is rapidly consumed in waters containing organic material. The commonly used dosage is 0,5 mg/*l*. As a disinfectant, potassium permanganate may possibly be effective against the cholera vibrio, but it is of little use against other disease organisms. However, it is commonly used for the oxidation of iron and manganese in water, particularly when other oxidation methods are ineffective due to the organic complexes existing in the water. Methods of adding potassium permanganate to the water are the same as those used for adding chlorine solutions.

5.6.4 *Ozone*

Ozone, produced by passing a high voltage discharge through dried air, is also a powerful oxidant. However it requires a large capital investment in sophisticated equipment, and is therefore not applicable for use in rural areas. However, in more recent times a simple apparatus has become available which, it is claimed, can produce a mixture of chlorine, ozone and other oxygen species by means of electrolysis of salt in a membrane separated cell. This combination of oxidants could be useful for both oxidation and disinfection applications.

5.7 Special treatments to remove problematic constituents from water

5.7.1 *Nitrate removal*

In South Africa the recommended limit for nitrate in drinking water is 6 mg/*l*. When waters have a high nitrate content, various methods are available to reduce this to within the recommended limits. It should be noted that the nitrate limit for drinking water is based on the effects high nitrates may have on infants ($<$ 3 years). Therefore in situations where no alternative source can be obtained, it should be the aim firstly to provide sufficient treated water to satisfy the needs of drinking water for just the infants in the community who rely on this water source. It will not be necessary to treat all of the water for the removal of nitrates.

5.7.1.1 Physical-chemical methods

- **Ion exchange**: Investigations carried out at Aroab in South West Africa (Namibia) demonstrated that ion exchange is feasible for the removal of nitrate in the production of drinking water for small communities. Small household ion exchange units (similar to household water softeners currently on the market) are now commercially available. Table salt (NaCl), used for regeneration (once or twice a month), is relatively inexpensive.

- **Reverse osmosis**: Reverse osmosis systems are very expensive, especially on a small scale, and although they can be justified if desalination of the product water is also required, they cannot seriously be considered for nitrate removal.

- **Distillation**: Desalination by distillation has been applied in many technical process variations and refinements, but the cost of thermal energy makes large-scale distillation uneconomical.

  Owing to rising fuel costs, solar energy is becoming increasingly attractive. Solar distillation is used extensively in the USA and Australia, and should also be applicable in the hot South African climate. Since the yield per exposed surface unit area is rather low (about 1 to 6 litres per m^2 per day) the capital outlay for solar stills is rather high and, at this stage, justifies only small-scale use (such as for households or small communities).

  The National Institute for Water Research (1973), now known as the Division of Water Technology, CSIR, developed a design for a greenhouse type solar still that is very easy to build and maintain (Van Steenderen, 1977). Extensive tests in South West Africa have shown it to be reliable.

5.7.1.2 Biological methods

- **Algal ponds**: Nitrate can be assimilated by algae and thereby removed from solution in the water. Cultivation of algae is a slow process, very much dependent on temperature and light intensity. Algal ponds must be shallow and exposed to light. The problem of separating the algae from the water before the water can be utilised presents an additional problem. It appears, at this stage, that algal ponds are not very suitable for the production of denitrified potable water.

- **Denitrification:** Bacteria also utilise nitrates in the water in their metabolic process. In particular, certain groups of bacteria, among them *Thiobacillus denitrificans*, utilise nitrate as a source of oxygen under anoxic conditions, converting the nitrate to nitrogen gas or nitrous oxide. The main disadvantage of biological denitrification for the production of potable water is that the presence of biological sludge residues, colour and micro-organisms, make further treatment necessary to render the water suitable for human consumption. The water can, however, be used for stock watering.

5.7.2 *Defluoridation*

When water supplies contain excessive fluorides, the growth of the bones and teeth of consumers can be adversely affected. This is often evidenced by the teeth of people using the water being mottled with a permanent black or grey discoloration of the enamel.

At present, three defluoridation methods have proved possible. The first two methods employ ion exchange media, i.e. activated alumina or bone char, which remove the fluorides as the water percolates through them. The media are periodically regenerated by chemical treatment when they become saturated. In the third method, the fluorides are removed by aluminium flocculation and settling. The activated alumina ion exchange method is the most efficient, but also costly. The use of crushed bone or bone char has been successfully used in a number of rural applications, and is very cost effective due to the availability of the raw materials. Flocculation with alum is limited to reducing the fluoride content to about 15 mg/l, which is still excessive for drinking water. However it could be utilised as a first step when the natural fluoride content is very high.

5.7.3 *Desalination*

Desalination of water for domestic purposes is an attractive solution to domestic supplies when the water available is highly saline (>1 500 mg/l total salts), and when alternative fresh supplies are too far away for economic utilisation. A number of rural areas in Southern Africa are endowed with saline groundwater resources. A few methods are available for desalinating water, even in rural areas.

Rainwater harvesting and storage is a means of obtaining an improved quality of water for domestic use, but is normally insufficient for the long dry periods when little or no rain falls. In particular recurring droughts in Asia, Africa and other world regions have placed an emphasis on the need for low cost yet reliable small scale desalination equipment to produce potable water from the saline ground water sources.

The age-old greenhouse type solar still (figure 5.16) has been tried and tested in many parts of the world. In particular the greenhouse type still has shown considerable potential for rural areas due to its simplicity. However, this still does suffer from problems of low efficiency and relatively high capital costs. Subsequently the cost per litre of fresh water obtained is relatively high.

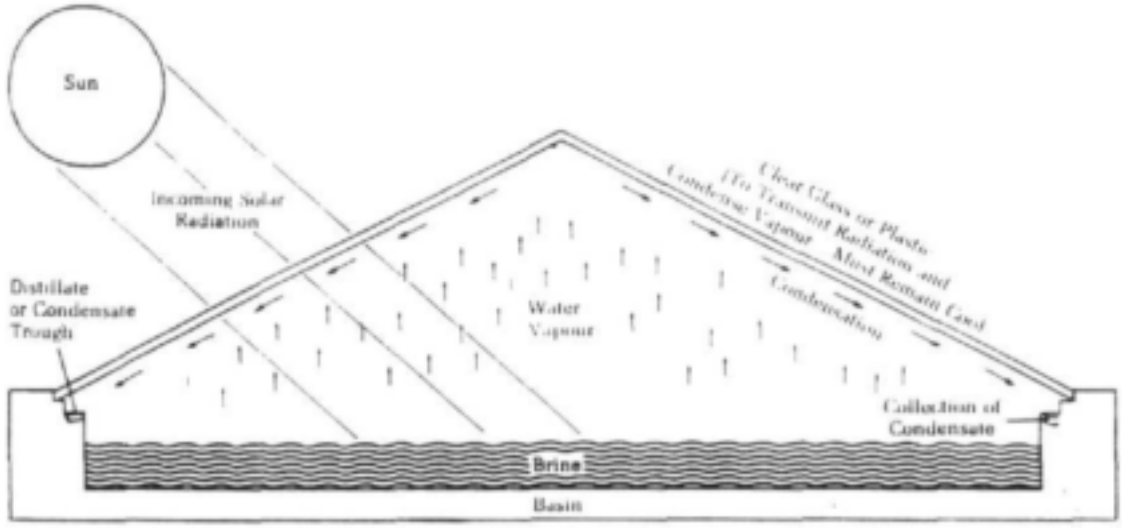


Figure 5.16 : Greenhouse type still for solar distillation of saline water

Many alternatives to the greenhouse still have been attempted, but most of these are too complex or costly for use in rural areas. However, one innovation which does show promise is the cloth type still. Cloth or wick stills can be assembled in a multi-effect configuration which results in a much improved efficiency. There still exists certain restraints on the use of these stills in rural areas due to their complexity and possible scaling problems.

Reverse osmosis membrane systems have been developed for small scale and household use. The so-called "tap water" units which rely on a low pressure to desalinate are not suited to the high salinity waters (3000 mg/l and more) generally encountered in the problem regions. The alternative RO systems for desalinating brackish water sources require some additional pumping mechanism to pressurise the water on a continuous basis to 200 psi (15 bar) or more.

A number of factors must be considered when selecting the best technology for desalting water in a rural area. Further developments are required to cater for the specific needs of rural populations.

5.7.4 *Algae removal*

Rural dwellers will generally choose alternative water sources if their closest sources become contaminated with excessive contaminants like algae, e.g. rainwater, springs, or ground water. Where alternatives are not sufficient, simple treatment methods are available to treat algae laden waters. These methods are low cost and low maintenance alternatives which are well suited to rural areas. Four methods of interest are presented here, these being slow sand filtration, horizontal roughing filtration, in line filters and oxidation with chlorine.

Algae entering a slow sand filter with the raw feed water will be captured in the schmutzdecke and upper sand layer. The algae may also proliferate in the supernatant water. These algae will be beneficial to the treatment process if they are in moderate numbers. Algae add oxygen to the water and filter out certain nutrients and even some metals.

However algal blooms have created problems in many slow sand filters. When the algae are of a free-floating filter blocking variety, short filter runs result. In other cases, oxygen consumption at night by large numbers of algae has created anaerobic conditions in the filters, leading to unpalatable effluents (Visscher et al 1987).

Techniques to prevent or control troublesome algae growth in slow sand filters include pre-treatment, shading, chemical treatment, biological methods and manual removal. Top feeding fish, such as tilapia, may be of value in controlling algae growth in slow sand filters, but under no circumstances should bottom-feeders be introduced. Manual removal may be a suitable method of removing filamentous algae from a filter.

Algae is removed to a large extent in a horizontal roughing filter (HRF). It is also not necessary to cover or shade a HRF as the water remains within the gravel and is not exposed to the surface.

Cartridge filters and filter screens which are commercially available can be very effective in removing algae, particularly filamentous algae. However, the filters will block fairly rapidly under conditions of high algae loads, and the replacement of cartridges or the cleaning of screens may be expensive and tedious. There is nevertheless, one in line filter unit which is useful particularly in the case of irrigation water for removing algae and other particulate matter which may block sprayers or drip systems. This is the Filtomat automatic self-cleaning filter (figure 5.17). Raw water entering the filter under pressure passes through a perforated plastic cylinder which acts as a coarse grid and strains the large particles from the water. The water then enters the lower compartment which is enclosed by another perforated plastic cylinder, lined with a fine corrosion resistant screen. As the solids accumulate on the screen, they form a bed which increases the pressure difference between inlet and outlet water pressures. At a predetermined pressure difference the cleaning mechanism is actuated and the screens are flushed clean. The filter unit can thus be connected in the supply line of a water supply or irrigation system and left to operate on its own.

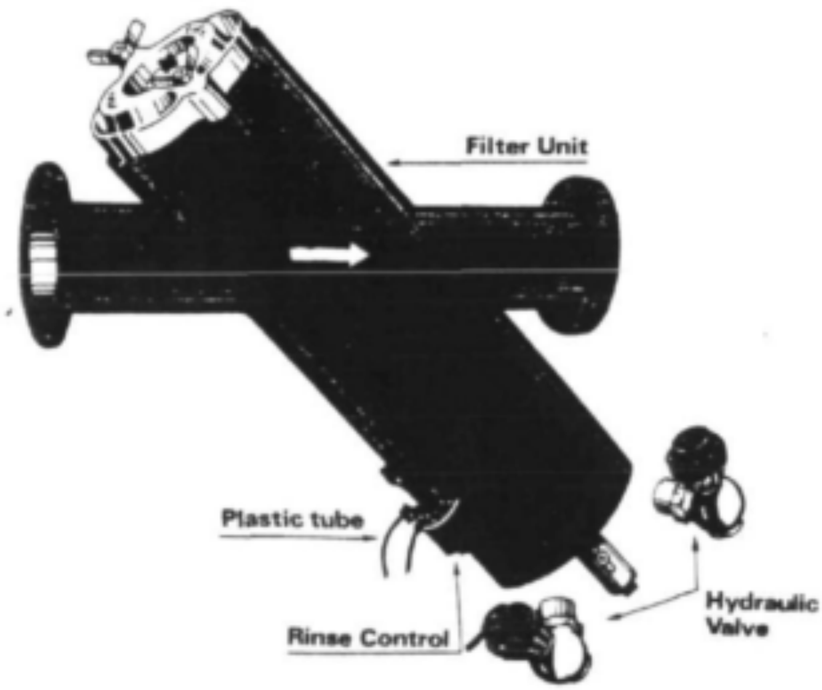


Figure 5.17 : Filtomat self cleaning filter

Chlorine and other oxidants can be used to inactivate algae cells by attacking the cell walls and reacting with the internal contents. Having inactivated the cell it is more easily removed from the water by conventional or other treatment processes. The use of chlorine for algae control may result in the formation of unpleasant tastes or odours in the water when certain species of algae are present.

5.7.5 *Organics (pesticides, oils)*

When the water source has an excessive amount of certain organics, it may be necessary to implement a specific treatment step to reduce the organic content. Problem organics are pesticides from agricultural practices, oils form waste dumps or spills, or general industrial type pollution. Usually a conventional treatment process will remove a percentage of the organics from the water. Where this is insufficient, or where the organics removal is the only treatment required, alternative methods will need to be used.

The method most suitable for organics removal is activated carbon adsorption. The water is passed through a bed of carbon granules where adsorption of the organics into the fine pores of the carbon particles, takes place. The capacity of the carbon to adsorb organics depends on a number of factors, including the type of carbon used and the type of organics present. A rough range in which many compounds fall in terms of adsorption onto activated carbon is 20 to 150 mg of the organic compound per gram of carbon. The adsorption rates for finer carbon granules (powdered carbon) will be higher. Once saturated, the carbon can be discarded, or sent to a facility where it can be regenerated by heat treatment.

Other methods for removing organics include the desalination methods described above, ultrafiltration (similar to reverse osmosis but using a membrane with larger pores) and oxidation. Oxidation methods will not remove organics, but rather convert the existing organics to other more acceptable compounds.

5.8 **Package treatment plants**

A number of package type treatment plants are commercially available. These plants usually include the following unit processes combined into a complete and compact unit:

- coagulation and flocculation
- filtration
- disinfection

Such plants are therefore designed to treat surface water which is contaminated with both suspended solids and micro-organisms. These plants are available in standard modular sizes and hence the capacity of the treatment plant can be built up by incorporating a number of modules in parallel. It is important to assess the operation and maintenance requirements for such plants to ensure that the community utilising the plant will reasonably be able to operate and maintain it.

6. PUMPING

6.0 General

Whenever water has to be raised from a lower level to a higher level, some type of pump or water lifting device needs to be employed. In order to raise the water, energy is required to drive the pump. The cost of pumping or lifting water is closely related to the rate at which power is used (i.e. the energy requirement in a given period). Note that the energy requirement is then the product of power and time. For example, a power of say 5kW expended over a period of 6 hours represents an energy consumption of 30kWh. Conversely, if water passes from a higher level through a turbine at a lower level, energy is generated at the turbine. This is the basis of hydropower, which is in essence a pump working in reverse.

For either of the above cases, the relationship between energy (a measure of the work done) and volume of water is given by the following formula:

$$E = \rho g V H \text{ (Joules)} \qquad \ldots\ldots\ldots 1$$

where E = the hydraulic energy in Joules
V = the volume of water in m^3
H = the head in metres
ρ = the density of water (1000 kg/m^3)
g = the gravitational acceleration (9,81 m/s^2)

This formula can be expressed in kilowatt hours (kWh), the energy units commonly used in electrical definitions, as follows:

$$E = \frac{9{,}81\ VH}{3{,}6 \times 10^3} \text{ kW.h} \qquad \text{(where : 3,6 MJ = 1 kW.h)}$$

$$= \frac{VH}{367} \text{ kW.h} \qquad \ldots\ldots\ldots 2$$

Fuels of various kinds have their potency measured in energy terms. For example petroleum fuel or diesel oil have a gross energy value of about 36MJ/litre, which is 10kWh/litre.

In order to select the right size of pump or pump-turbine and motor-generator it is important to know the <u>power</u> requirement (P). Power is the rate of energy supplied/generated, or the energy required/generated per unit time. The formula relating power to the rate of flow of water (Q) is given by the formula:

$$P = \rho g Q H \text{ Watt} \qquad \ldots\ldots\ldots 3$$

where P = the hydraulic power in Watt
Q = the flow rate in m^3/s
H = the head in metres
ρ = the density of water (1000 kg/m^3)
g = the gravitational acceleration (9,81 m/s^2)

This formula can be expressed in kilowatt (kW) thus:

$$P = \frac{9.81\,qH}{10^3} \quad \text{kilowatt,} \qquad \text{where } q = \text{flowrate in } l/s$$

or $$P = \frac{qH}{102} \quad \text{kW} \qquad \ldots\ldots\ldots\ldots \quad 4$$

The above formula (4) is a particularly useful formula. It gives both the hydraulic power potential of a particular hydropower site, or conversely the theoretical hydraulic power requirement of a particular pumping site.

It must be realised that the above equations are theoretical power requirements. Because pumps and turbines are not 100% efficient, more power will be required to drive the pump in practice (or less power will be generated by a turbine). The ratio between the theoretical and actual power requirement is a measure of the efficiency of a pump:

$$\text{actual power required} = \frac{P \times 100}{\text{efficiency } \%}$$

As an example, a pump is required to pump 50 l/s at a total head of 55 m, the hydraulic efficiency at this duty being 72 %.

The theoretical power requirement would be:

$$\frac{50 \times 55}{102} = 26.96 \text{ kW}$$

Therefore the actual power required for the pump is:

$$\frac{26.96 \times 100}{72} = 37.45 \text{ kW}$$

Each component of a pumping system has an efficiency (or by implication, an energy loss) associated with it. The system or total efficiency is the product of multiplying together the efficiencies of all the components. For example, an electrically driven centrifugal pump consists of an electric motor (efficiency typically 85%), a mechanical transmission (efficiency if direct drive of say 98%), the pump itself (optimum efficiency of say 70%), and the suction pipe system (say 95% efficient). Then the overall system efficiency will be as follows:

$$0.85 \times 0.98 \times 0.7 \times 0.95 = 0.55 \quad \text{(i.e. 55\% efficient)}$$

Each pump has its own performance characteristics, and hence its own optimum operating conditions. This is particularly true of centrifugal pumps. Therefore when a pump is specified for a certain operating condition, and is operated under different conditions, the efficiency can be expected to be reduced even further. Other losses in efficiency arise when operating a diesel engine, which is used to drive a pump, at higher altitudes and or temperatures. Generally there is a 1 % reduction in power for every 2.8 °C above 29 °C, and there is a 3.5 % reduction in power per 300 m above 150 m above sea level. As a rough and useful guide, therefore, the power formula (4) above should be adjusted to give a power requirement estimate based on 50 % efficiency as follows:

$$P = \frac{qH}{50} \quad kW$$

Note that the water output of a rotary pump is almost proportional to the rotating speed. Hence by adjusting the pump rotational speed, a pump can be matched with its working or design requirement. Since most engines run at a fixed rotational speed, the pump rotational speed can be adjusted by fitting different sized drive pulleys to the engine and pump respectively. The relationship between speed of pump, speed of engine, and the diameter of the pump and engine pulleys is as follows:

$$Np = Ne.De/Dp$$

where

Np = pump speed in rpm,
Ne = engine speed in rpm,
Dp = diameter of pump pulley,
De = diameter of engine pulley.

Various types of pumps and lifting devices are used for raising water. Some are used for irrigation purposes where low heads and large flow rates are required, others for domestic supplies where high heads and relatively small flow rates are normal. These pumps can be powered by a wide variety of devices known as prime-movers. Various pumps, water lifting devices and prime-movers are discussed in the following sections.

6.1 Types of pumps

The distinction between types of pumps lies not so much in the mode of application of the pump, but in the principles of operation of the pump. Pumps may be divided into the following three major categories:

positive displacement pumps
turbo pumps
direct lift devices

6.1.1 *Positive displacement pumps*

Water is for most practical purposes incompressible. Therefore water can be displaced by drawing a close fitting piston through a pipe full of water, or by pushing a solid object into the water so that the water around it is displaced in the direction of least resistance. Pumps working on this principle are known as positive displacement pumps.

Most pumps fall into this category. However, since a number of these fall into turbo pump category with particular characteristics, they have been separated out here for ease of reference. Hence positive displacement pumps in this sense refer to:

piston pumps,
diaphragm pumps, and
rotary spiral screw pumps,

with the sub-category of turbo pumps being addressed separately.

Positive displacement pumps are designed to deliver water from a source to a point at a higher elevation or against pressure. This is accomplished by pressurising the water and feeding it into the delivery pipe under pressure.

6.1.2 *Turbo pumps (Rotodynamic pumps)*

The so called turbo or rotodynamic pumps propel water using a spinning impeller or rotor. All of these types of pumps have characteristics which gives them a limited range of speeds, flows and pumping heads in which good efficiency can be achieved. Hence manufacturers produce a range of pumps to cover a wide range of heads and flows. Pumps in this category include:

centrifugal pumps,
multi-stage submersible centrifugal pumps, and
jet pumps.

6.1.3 *Direct lifting devices*

These water lifting devices include buckets, levered scoops, windlasses, and rotary bucket lift systems. They are usually operated by man or animals, although they could be mechanised in certain cases. The flow from any of these devices is a function of the volume of each bucket, and the speed at which the buckets are raised. The flow is usually intermittent, or at best pulsationary. A number of such devices have been developed over the world during the last centuries. Devices still in use include:

the shadouf, and dall,
Archimedian screw,
Persian wheel,
chain or rope pumps,
Mohtes, and
bucket hoists.

6.2 Hand-dug well and tube well pumps

There are a number of simple types of pumps and lifting devices that have been specifically designed and developed for use with wells and tubewells. These pumps may be classified into two broad groups: shallow pumps which lift the water by creating a negative pressure relative to atmospheric pressure and which are limited to pumping depths of around 5 metres (the limit to which atmospheric pressure can lift water is 7 metres at sea level); and pumps which lift water by creating a positive pressure relative to atmospheric pressure and are generally used to lift water from greater depths. The advantage of a shallow lift suction pump is that there are no moving parts below the pumphead, greatly reducing maintenance procedures (no lifting of pump assembly from well or tube well). Their limitation is obviously the depth from which they can raise water (typically between 3 and 6 metres). Most well pumps therefore are of the positive pressure variety.

Buckets winched down to the water level were the most common technology for lifting water from open wells. They have the advantage of being simple and therefore easily maintained. The disadvantage is that buckets are often contaminated by handling, thereby polluting the well, and this method is time consuming and slow. There are a number of relatively simple technologies that overcome to some extent these problems.

6.2.1 *Chinese tube chain pump*

Developed in China for manual or bullock-powered operation, these pumps have the advantage of simplicity in terms of both concept, and construction and maintenance capability. For an idea of the principal of these pumps see Figures 6.1

Accurately cut or rubber sealed washers make a tight fit in a small section of steel or uPVC pipe located at the lower end of a larger diameter riser pipe. By having only a short section of pipe where the washes fit tightly, the power required to drive the pump is reduced and wear on the washers is minimised.

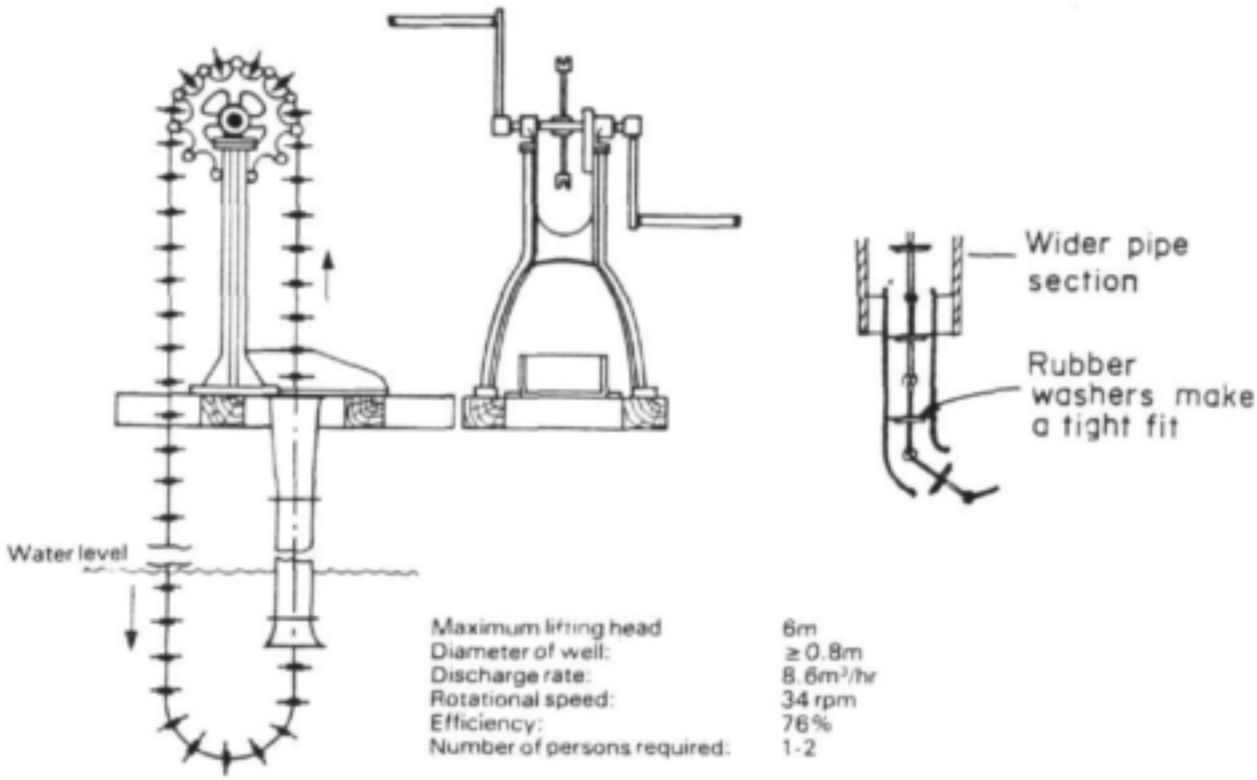


Figure 6.1 : Chinese chain and washer pump

6.2.2 *The Blair pump*

This is a positive displacement pump designed specifically for low lifts (about 6 metres). The limit to which the water can be physically lifted without the use of levers is about 15 metres.

The rising main cum pushrod is manufactured from galvanised iron, or uPVC which is lighter than galvanised iron and is buoyant when submersed in water. It is easily assembled and can be manufactured, with the exception of the two foot valves and one spring, from off-the-shelf plumbing fittings. The pump is designed to be self lubricating since the piston does not fit tightly into the pump cylinder. Two possible configurations for use in wells and tube wells are shown respectively in Figures 6.2 and 6.3.

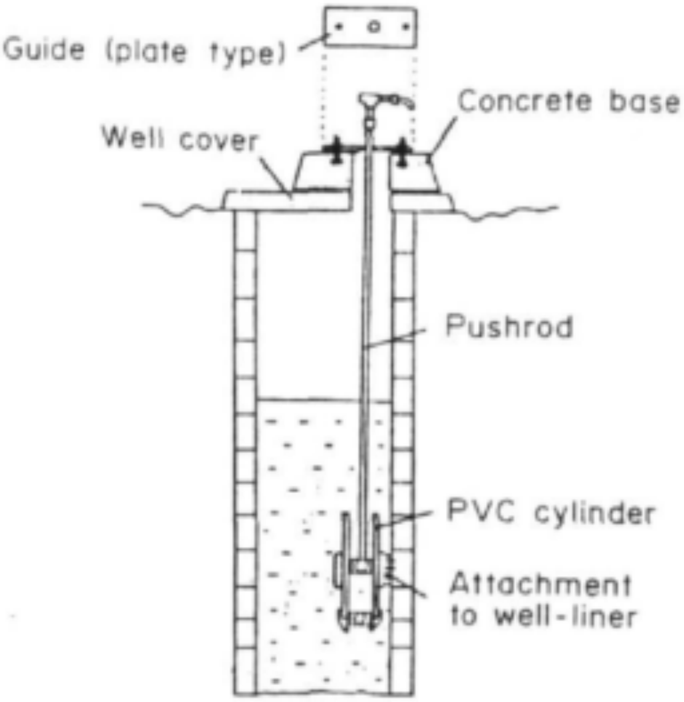


Figure 6.2 : Blair pump on a hand-dug well

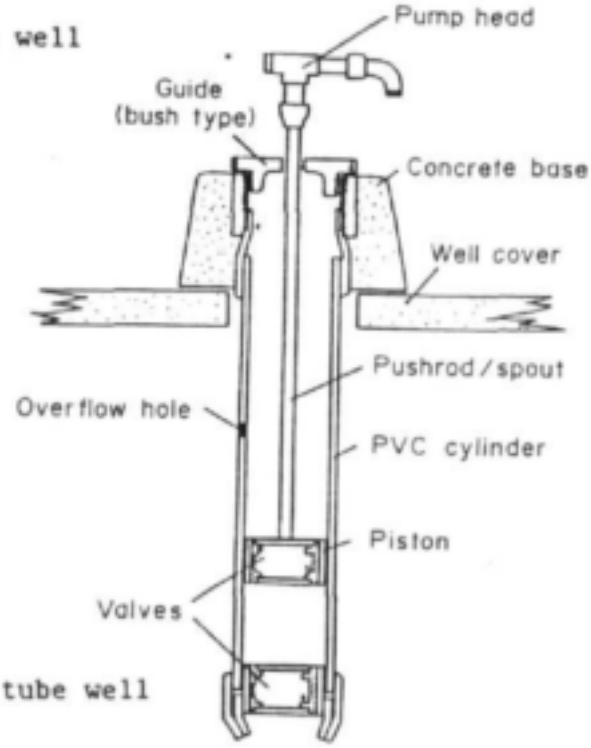


Figure 6.3 : Blair pump on a shallow tube well

6.2.3 *Tube well bucket*

The concept of using a small diameter long bucket for use with tube wells has been developed by the Blair Research Laboratory and modified and improved by Mr C L Louw, Centre for Social Research and Documentation, University of Zululand for use with tubewells in Northern Kwazulu. The design can be readily modified for use with hand dug wells, that have been covered with a suitable lid. The 75 mm diameter bucket has a simple poppet valve at its base located above a hole of about 25 mm diameter. The valve opens as the bucket is immersed in water and closes as the bucket is raised. When the bucket is lowered into a basin at the top of the well the valve is pushed open and water escapes via the bottom hole. The water from the base must be directed via a hose pipe to fill water containers. The capacity of the bucket is about 12 litres. The design of the bucket and poppet valve, whilst simple, is critical to the successful operation of the tube well bucket.

6.2.4 *Off-set handpumps*

Considerable advantage in terms of water quality is claimed by Jenkins (1984) in off-setting the pump from above the well or tube well (see Figure 6.4). By removing the possibility of contamination of the water of the well or tube well by waste water, he claims that in a project in Nepal during the 1970's coliform counts in hand dug wells previously equipped with simple buckets improved from 10 000 per 100 ml to 10 - 15 per 100 ml after introduction of off-set handpumps. In addition flexibility and ease of removing down-hole components in the well or tube well are seen as advantages with off-set handpumps. The type of pump which may be employed in this design could be a suction pump, or a diaphragm pump which do not require a straight riser pipe or push rod.

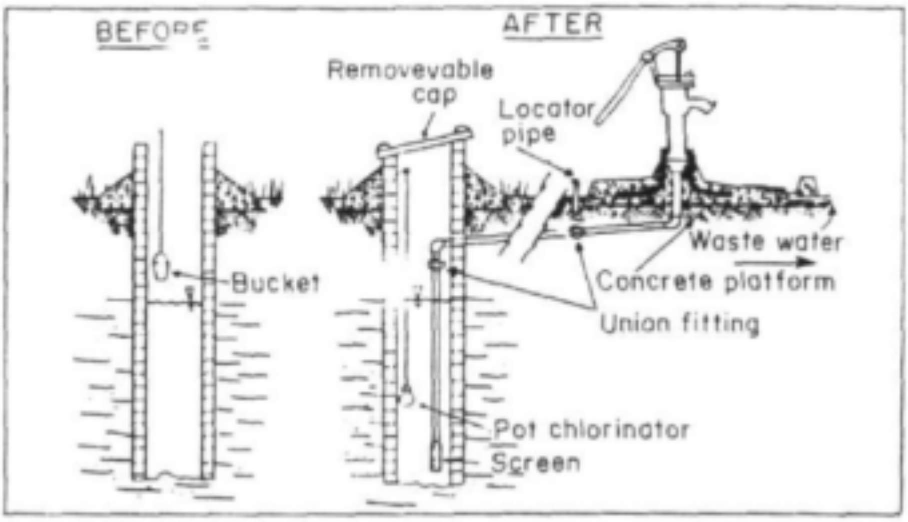


Figure 6.4 : Offset handpump on a well

6.3 Deep well and borehole pumps

A number of pumps have ben developed for use with deep wells and boreholes. Groundwater at many locations in Southern Africa is too deep for shallow well and tube-well pumps (> 20 m), and narrow boreholes are too small for many of these pumps. Even where groundwater is shallower than this, seasonal variations and particularly drought events can lead to the level of the water table dropping significantly. Hence pumps which can operate from deeper depths are usually the pump of choice for recovering groundwater resources. In some areas the groundwater table is at a depth of 80 m or more, and the choice of possible pumps which can be used in such situations is limited. A large number of people in rural areas rely on groundwater as their primary water source, and it is important that a pump best suited to the application is selected in each case.

6.3.1 *Rotary spiral screw pumps*

The principle of operation of this pump is that a spiral rotor is rotated inside a close-fitting stationary casing called a stator. As the rotor turns, it increases the pressure of the water by creating a progressive cavity which 'screws' the water in the direction of flow. These types of pumps are used in a number of applications, the most well-known being the rotary borehole pumps which are common in Southern Africa. A cross section of this pump is shown in Figure 6.5.

These pumps, which are referred to as helical rotor, progressive cavity, screw or worm pumps, deliver water in a steady stream. The flow rate can be adjusted by adjusting the speed of rotation, with a maximum speed under normal circumstances of 1200 rpm. These pumps can be easily primed from the delivery pipe and have good suction characteristics. The chief manufacturer of all types of rotary screw pumps in South Africa is Mono Pumps (Pty) Ltd. Diesel driven engines and electrical motors are suitable for driving this type of pump, with some being designed for hand operation. Internal losses in rotary pumps is usually higher than in reciprocating pumps through slip (internal leak-back). Since slip increases with increasing pressure, rotary pumps are less suited to high pressure systems. When these pumps are used at depths of 75 m or more, a gear device should be provided at the drive to the power requirements when starting the pump.

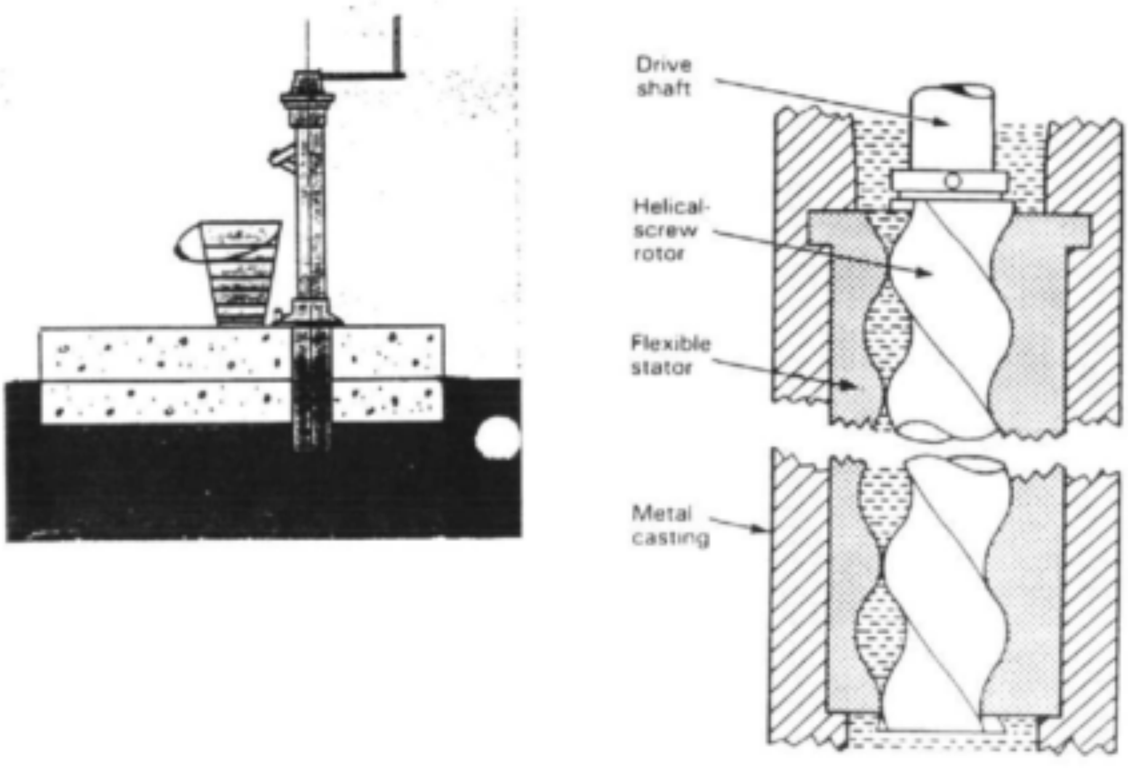


Figure 6.5 : Rotary spiral screw hand pump (Mono)

6.3.2 *Reciprocating piston pump*

Reciprocating piston pumps are the most common form of displacement pump. A common example of a borehole piston pump is shown in figure 6.6. Water is sucked into the cylinder on the up-stroke, and the water above the piston displaced out of the pump. On the down-stroke the lower check valve is held closed, and the water in the cylinder is displaced out through the piston ready for the next up-stroke.

Certain of these pumps known as net positive suction head pumps (or suction pumps) rely on atmospheric pressure to push water into the suction pipe of the pump. The pump reduces the atmospheric pressure on the water in the suction pipe and the atmospheric pressure on the water outside the suction pipe pushes the water up. The difference in level between the pump and the level of the water to be pumped is determined therefore by the negative suction pressure exerted by the pump in the suction pipe relative to atmospheric pressure. Normal atmospheric pressure at sea level is about 1 bar = 100 kPa. This pressure is theoretically sufficient to support a column of water of 10,19 metres under a perfect vacuum (zero pressure). In practice since no pumps are perfect, atmospheric pressure limits pumping depths to 7 metres at sea level (or 6 m at 1100 m elevation). Many pump suctions can only effectively handle suction depths of about half those quoted above due to leaking valves and seals. Therefore applications of suction pumps are restricted to shallow depths. They are sometimes referred to as shallow pumps. A typical hand operated shallow pump is shown in Figure 6.6. The pumping mechanism consisting of a suction valve and plunger, is located above ground level. This considerably simplifies maintenance procedures. The USAID "Batelle" pump and the UNICEF "New No. 6" pump developed in Bangladesh are examples of hand operated suction pumps. The more recent SWS Rower Pump is a direct action extremely simple modification to these pumps.

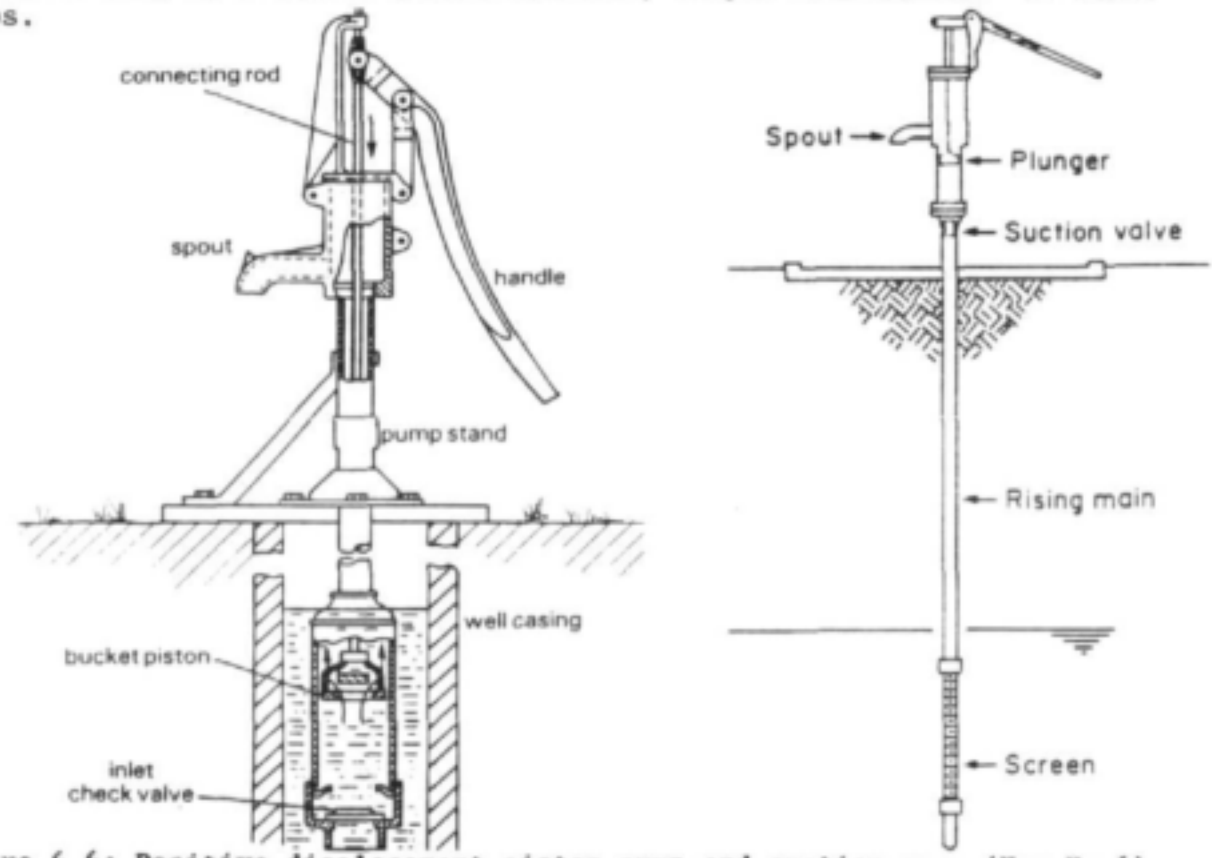


Figure 6.6: Positive displacement piston pump and suction pump (New No 6)

Many of the internationally most widely used borehole and well hand pumps are of this type, including the Blair pump (Zimbabwe) - see Section 6.2 - the India Mark II, the Afridev (manufactured by Mono Pumps, England, and a South African version "Safridev" may soon be available), National Pumps (South Africa), Van Reekum (Netherlands), Consallen (England), Maldev (Malawi), Nimric (Hammanskraal, South Africa), Dempster (Nebraska, USA), Volanta (Netherlands) and Climax (Peacehaven, South Africa). The "Catalogue of Equipment for Small Water Supplies" (1986) by the then NIWR of the CSIR contains a more comprehensive guide to many makes of this type of pump.

Reciprocating pumps, although low in efficiency, are well suited to pumping against variable heads, including heads greater than 75 m.

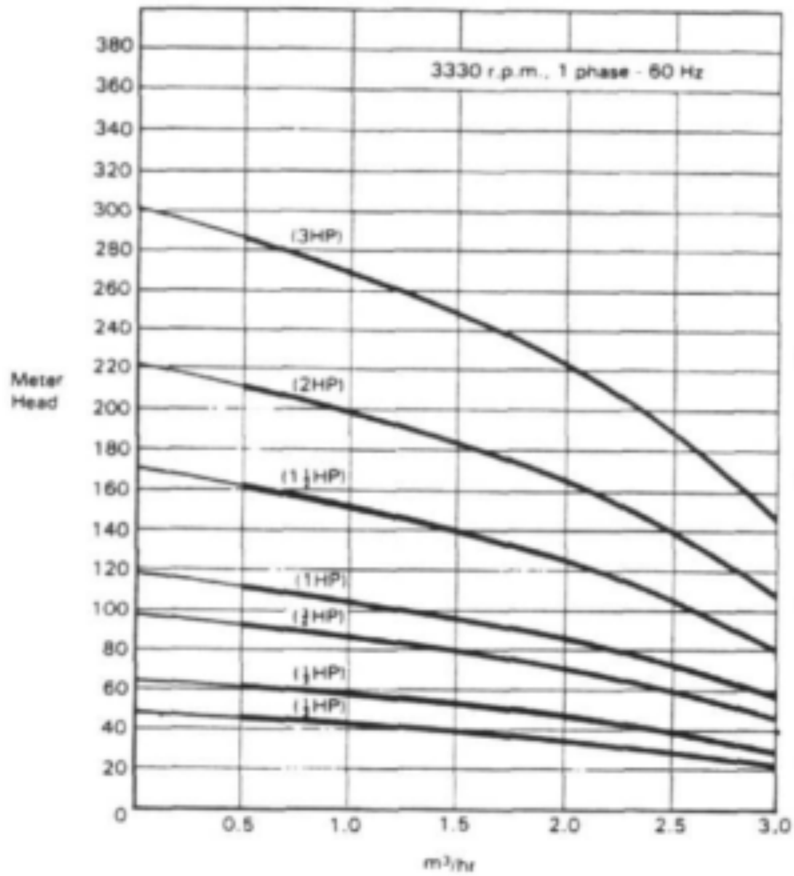
6.3.3 *Submersible centrifugal pumps*

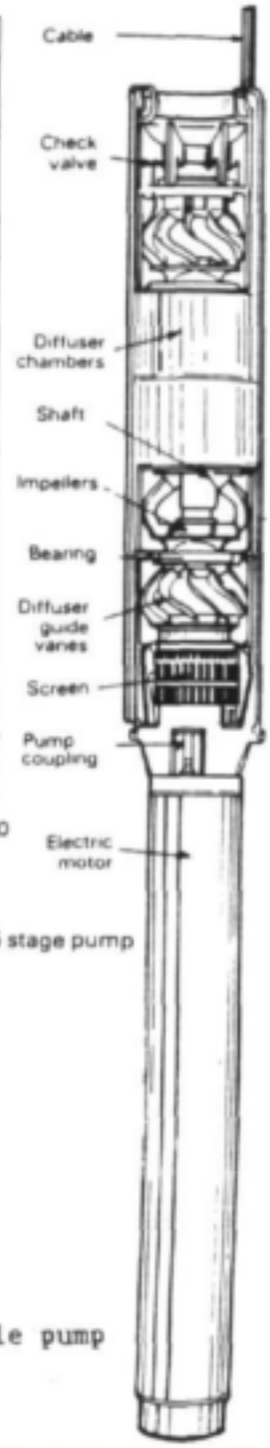
Centrifugal pumps use impellers to displace fluids like water by momentum rather than positive mechanical travel. Hence output is dependent on impeller speed and on discharge head. Typical operation speeds range from 1500 to 3000 rpm. Low speed centrifugal pumps can be expected to wear less and last longer than high speed pumps. Performance charts should be consulted when selecting pumps, and the pump manufacturer should be consulted before making the final choice of the pump, and the size of the engine or motor.

Submersible borehole pumps are equipped with either DC or AC (single or three phase) electric motors, which are directly coupled to the pump and encased integrally with it. A typical borehole pump of this type is shown in Figure 6.7.

Readily available submersible pumps are manufactured in South Africa by Mono (known as the Mono Gould pumps), Jacuzzi (Italy), Grundfos (Denmark) and McDonald (Iowa, USA). Many of the submersible borehole pumps on the market at present are fitted with Franklin electric motors (USA). Submersible borehole pumps, particularly when driven by DC electric motors, have been found to be ideal when incorporated into solar-pumping systems, because at low power output the pump motor will still turn (low starting torque). Because of the fluctuating electrical power output of solar-voltaic panels, minimum power requirements to start the pump (starting torque) rule out the use of many other types of borehole pumps. With submersible pumps water delivery rate is approximately proportional to speed of rotation, which is governed by the power supplied, which in turn is proportional to the solar irradiation. Thus the more the sun shines, the more water will be pumped.

Submersible pumps for boreholes are recommended when the borehole is high yielding, the yield reasonably stable, and groundwater conditions fairly well known. The multistage configurations result in high pumping heads being possible. Submersible pumps are relatively more efficient than other borehole pumps, and are easier to install as no long shaft is required. It must be borne in mind however, that the output of these pumps can change significantly with fluctuating borehole drawdown levels or increased discharge pressures. See also comments under 6.4.1.





How adding stages increases the power requirements and the head with 5 stage pump (part section) on right (based on data published by Grundfos)

Figure 6.7 : Electrically driven submersible borehole pump

6.3.4 *Diaphragm pumps*

Pumps that employ the deflection of a flexible diaphragm to displace liquid are known as diaphragm pumps.

However, the principle of a deflecting diaphragm has been successfully employed in the Vergnet foot-pump (France) which has now been copied by Merrill Pump (England). The principle of the diaphragm borehole pump is that the foot pedal (or handle) forces water down the closed pipe in to the diaphragm or elastic sleeve, which then expands. The increasing volume of the diaphragm forces water out of the cylinder, up the rising main and out of the spout. When pressure is taken off the foot pedal (or handle), the diaphragm contracts, the delivery valve closes and water enters the cylinder through the foot valve (Figure 6.8). The cycle is then repeated. This type of pump is not common in Southern Africa at present, although it is widely used in Francophone Africa.

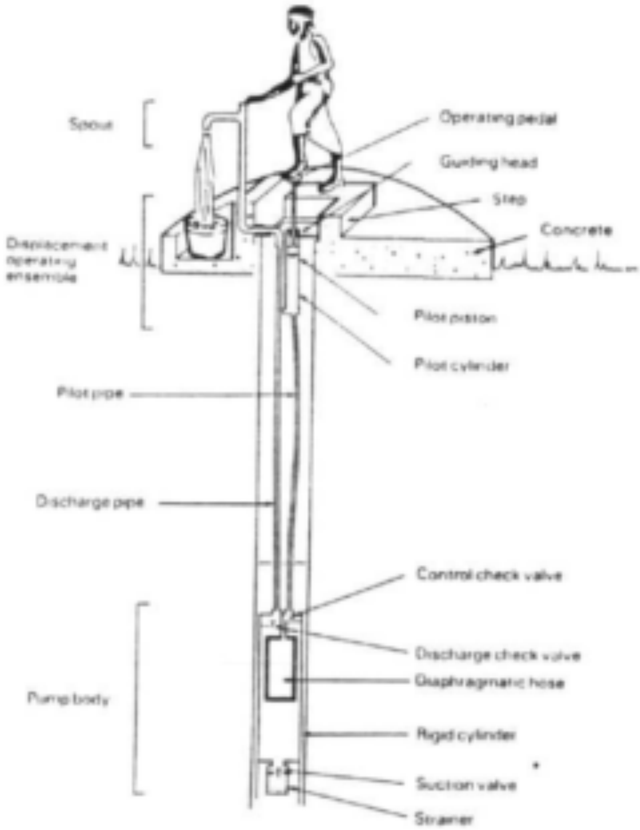


Figure 6.8 : Deep well diaphragm pump (Vergnet foot pump)

A pump which has more recently become available and which fits into this category is the Pulsa Pump which operates on the principle of inertia. Flexible compressible balls are placed in a chamber at the bottom of the borehole with a standard type footvalve for letting water into the chamber. A discharge pipe connects this chamber to surface where a plunger and cylinder arrangement exists. The water in the discharge pipe is pressurised by depressing the plunger on surface, resulting in the flexible balls in the chamber compressing. On release of the pressure (return of the plunger) the balls expand, forcing the water in the chamber back up the discharge pipe. When the balls are fully expanded, the inertia of the water moving up the discharge pipe causes a negative pressure in the chamber and the footvalve opens, allowing fresh water into the bottom of the chamber. This also displaces water out of the discharge pipe at surface. The pump operator develops a rhythm of pumping which matches the characteristics of the pump. The pump is simple, with most working parts on surface, and only one connecting pipe down the hole.

6.3.5 *Jet pumps*

Jet pumps are inherently less efficient that the other types of pumps listed in this section, but this has not prevented them from being used in many boreholes. The principle of the pump is to create an increased vacuum in the delivery pipe, thus enabling water to rise from deeper depths than a conventional centrifugal pump. The vacuum is created by forcing a high velocity jet of water through a nozzle (figure 6.9). The second basic component of the jet assembly is a venturi tube (diffuser). Its function is to convert the increased velocity, created by the nozzle, into increased pressure. The increased pressure in the suction pipe enables the water to be raised from a greater depth than is possible with a standard suction pump. Raising water from depths of up to 70 metres is possible with jet pumps, although in general, the higher the lift, the lower the efficiency of the pump. For a surface mounted pump and motor, water can be extracted from depths of around 10 to 20m satisfactorily.

Energy and hence efficiency are lost at each stage where pressure is converted to velocity and vice versa. In deep wells the jet assembly (nozzle and diffuser) is located down the well in a closed compartment called the suction chamber. Another advantage of the jet pump is that it can reliably run on a mixture of air and water without losing its prime.

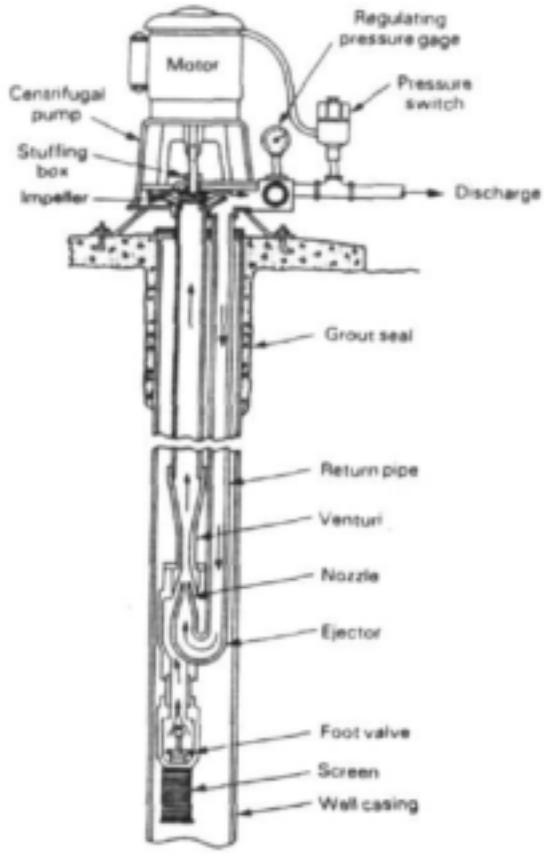


Figure 6.9 : Borehole jet pump installation

Apart from the suction arrangements described above, the rest of the pump on the delivery side is designed and constructed as a conventional centrifugal pump. Deep well jet pumps are distinguished by the fact that there are two pipes running from the pump down to the intake water level, the smaller one is the pressure pipe supplying a pressurized jet of water, the larger is the suction pipe to raise the water from the well or borehole. Jet pumps with DC electric motors may be used in conjunction with photo voltaic cells for solar pumping systems.

6.4 Pumps for surface water pumping

The principles of operation of borehole pumps are usually equally applicable to pumps in this category. The most common pumps used for surface water pumping are centrifugal pumps driven by electric motor or a diesel engine. However reciprocating pumps and diaphragm pumps are also used in a number of specific applications.

6.4.1 *Centrifugal pumps*

These operate by rotation of impellers or screws within a pump body or casing. Large size screw type mixed-flow axial pumps are suitable for large flows at low pumping heads. Where high pumping heads are to be dealt with, multi stage pumps are used; each stage of the pump comprising casing and impellers which progressively raise the pressure and hence the hydraulic potential energy of the water. They can be installed in various ways according to local conditions. The axis of the pump can be mounted vertically or horizontally. The suctions can either be above the level of the water to be pumped or below (submerged suction). In each situation basic data of lengths, heights, size of pipe etc. are required in order to predict the operating characteristics of the selected pump and to design the pump installation.

Each centrifugal pump has a maximum efficiency corresponding to a particular value of discharge Q, head H, and speed N. The curves drawn for discharge versus H, N and efficiency *e* are known as characteristic curves. The most suitable pump for a particular application is one whose efficiency is maximum at the operating values of head and flow rate.

Pump speeds of up to 1500 rpm should be chosen for raw water pumping, with speeds up to 2900 rpm for clean water with no suspended solids, or for pump openings of 200 mm or more. The capacity of the pump is directly proportional to the speed, but pumping head varies with the square of the head.

Because of the importance of matching the pump with the actual operating conditions, head losses should be calculated as realistically as possible without the safety margin normally included in the calculation for selecting pipe dimensions. Friction losses should never exceed the static head at any point in the discharge line as the operating point of the pump will then be very difficult to determine correctly.

There are many manufacturers of centrifugal pumps including, Sulzer (Switzerland), Stewarts & Lloyds (South Africa), Salweir ({Pty) Ltd of Benoni, South Africa. A sectional view of a centrifugal pump is shown in Figure 6.10.

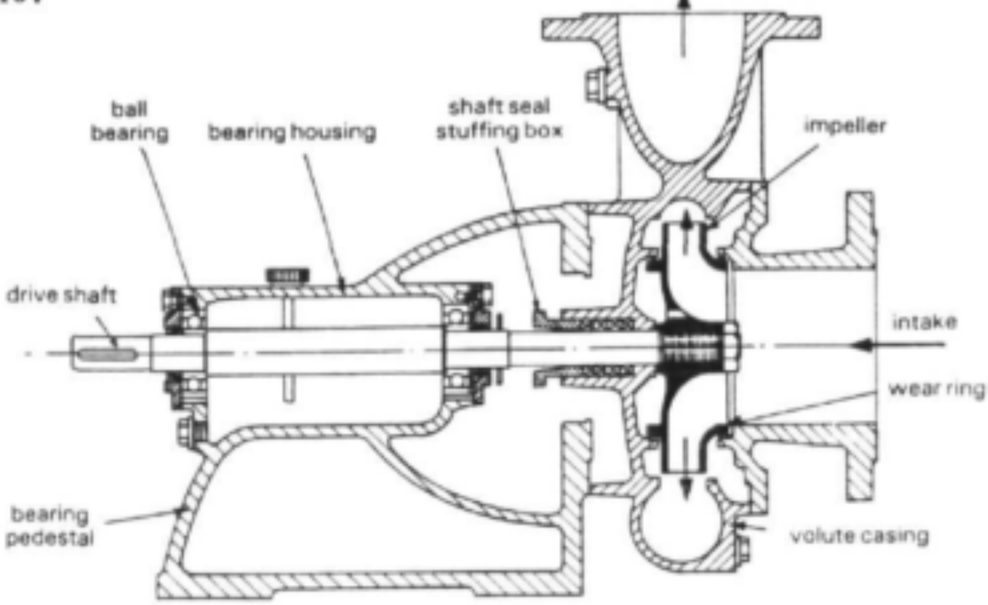


Figure 6.10 : Centrifugal pump

Submersible centrifugal pumps are commonly used for sewage pumping installations and in other situations where liquids have to be pumped out of an underground sump. The pump and its engine is lowered beneath the surface of the water to be pumped. These pumps have been specifically designed to deal with solids encountered in sewage or high silt loads without blocking. By employing multi-stage impellers and diffusers of glass reinforced plastic supported in stainless steel bowls, corrosion resistance is enhanced and abrasive water handling capabilities improved. Replacement of impeller stages is relatively less costly and simple in these submersible multi-stage pumps. See also comments under 6.3.3.

6.4.2 *Reciprocating pumps for surface pumping*

The so-called "plunger pumps" manufactured by National Pumps in simplex (single cylinder), duplex (double cylinder) models, with either single acting or double acting (pumps on both upward and downward strokes), and triplex, single acting models are in principle reciprocating piston pumps.

These pumps were noted for their reliability, and were very popular in South Africa up to the early 70's. They are mainly belt-driven by electric or diesel engines at relatively low speeds (75 to 250 revolutions per minute).

Reciprocating pumps are still a good choice in situations where a high pressure is required, and a low volume of water is being pumped.

A relatively simple reciprocating pump known as the Devol Pump, operates on the natural energy of a small stream. The pump is driven by a simple turbine, which is driven by a flow of water from a stream. It has been developed by a New Zealand company Devol Engineering. The pump requires only a low head of water, usually acquired by building a small dam across a stream. A minimum depth of 25 mm of stream water running through the turbine is required for it to operate properly.

6.4.3 *Diaphragm pumps for surface water pumping*

Because of their ability to handle high solids concentrations, diaphragm pumps have been used to pump sewage, sludges and thick industrial products such as paint. Modifications of this principle are the stainless steel rotary lobe pump and the peristaltic type pumps. Diaphragm pumps are commonly used for the dosing of chemicals at water treatment plants.

Semi-rotary pumps, more commonly known as wing pumps or vane pumps (figure 6.11), are commonly used to dispense paraffin and fuel from 200 litre oil drums. A wooden handle is pumped one quarter of a revolution, backward and forward. On each stroke liquid is positively displaced from the casing of the pump by movement of a flap or wing inside the casing. Water is drawn in through a lower brass (check valve) and displaced through an upper brass valve in the same manner as the reciprocating piston pump described in Section 6.3.3. Wing pumps are manufactured by Stewarts & Lloyds (South Africa) and can be used for pumping water. They are, however, not as durable as most hand pumps and cannot be subjected to prolonged and sustained operation.

6.5 Types of water lifting devices

In addition to the wide variety of pump types described in the previous section, there are a number of water lifting devices, some of which have been in use for thousands of years. Many of these have been designed to be driven by animal power. In Southern Africa the use of animal power is limited to mainly ploughing. However, in most Third World countries of Africa, Asia and South America, extensive use is made of animal power in agriculture and transportation. An ox of mass 400 kg can exert a continuous power output of approximately 0,4 horse power (200 Watt) when moving at a speed of 0,7 metres per second on a level surface (Stern, 1979). This may seem insignificant when compared to diesel or petrol engines which are rated in kilowatt. However, when one considers the power potential equation 10.4, the power of one ox (300 Watt) has the potential to raise 1 litre of water per second to a head of 30 metres. Thus in 1 hour the ox can potentially raise 3600 litres of water to a height of 30 metres (neglecting inefficiencies of the mechanical and pumping device). In practice about 75 % of the potential is available, say 2700 litres of water per hour. In order to pump the same amount of water using photo voltaic panels, approximately 20 panels of 40 Peak Watt capacity each would be required. The comparison is made only to stress the enormous potential power supply that is being ignored in South Africa.

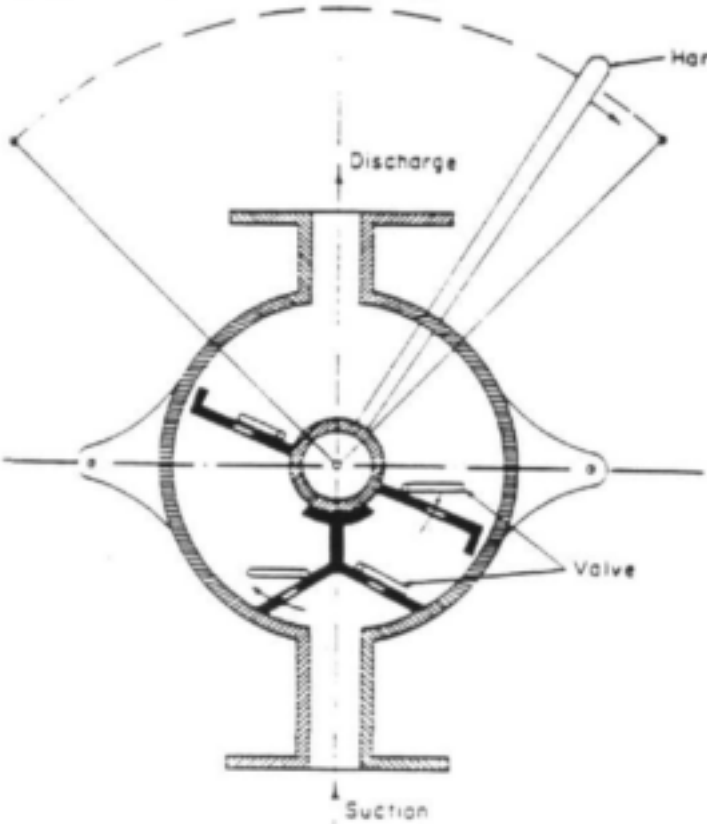


Figure 6.11 : Hand operated vane pump

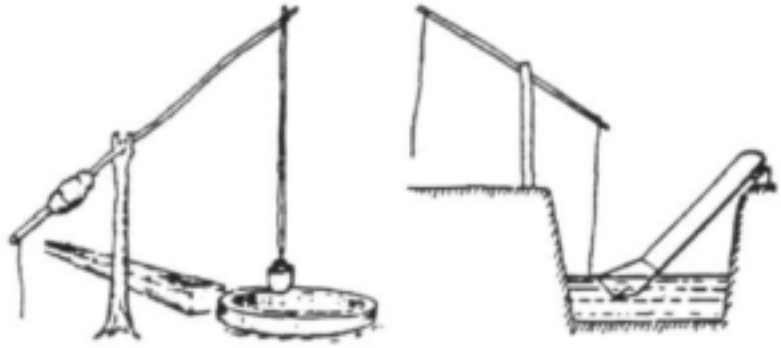
Some of the more common water lifting devices in use throughout the world are used in irrigation schemes to lift water from irrigation canals into the fields.

6.5.1 *The 'Shadouf'*

The word *shadouf* is an Arabic word. It describes the bucket, lever and counterweight system, commonly used throughout the Middle and Far East to raise a bucket, or animal skin vessel filled with water from a well or irrigation channel. A typical *shadouf* is shown in Figure 6.12.

6.5.2 *The 'Dall'*

The word *dall* is used in India to describe a similar device to the shadouf. The basic modification is that a hinged scoop is employed to raise the water in place of a bucket. A typical *dall* or *auge* is shown in Figure 6.12.



Shadouf Indian 'dall'

Figure 6.12 : Typical shadouf and Indian dall

6.5.3 *Archimedian screw*

This is reputed to have been invented by Archimedes in about 200 BC. It is still used in Egypt and India for shallow lift irrigation. Archimedian screws are used in South Africa for low lift applications in sewerage treatment works. A cross section of an Archimedean screw is shown in Figure 6.12. A modification of this principle is the coiled pipe pump described by Reimer (1985). This type of coil pump has, however, not been found to be very efficient due to problems associated with the rotating water tight joint that is necessary between the rotating coil and the delivery pipe. A paddle driven coil pump is shown in Figure 6.14.

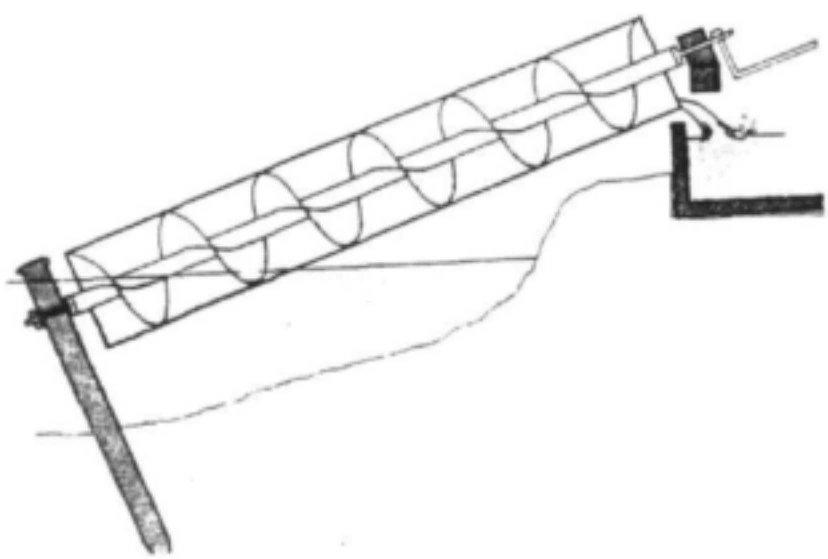


Figure 6.13 : Archimedian screw

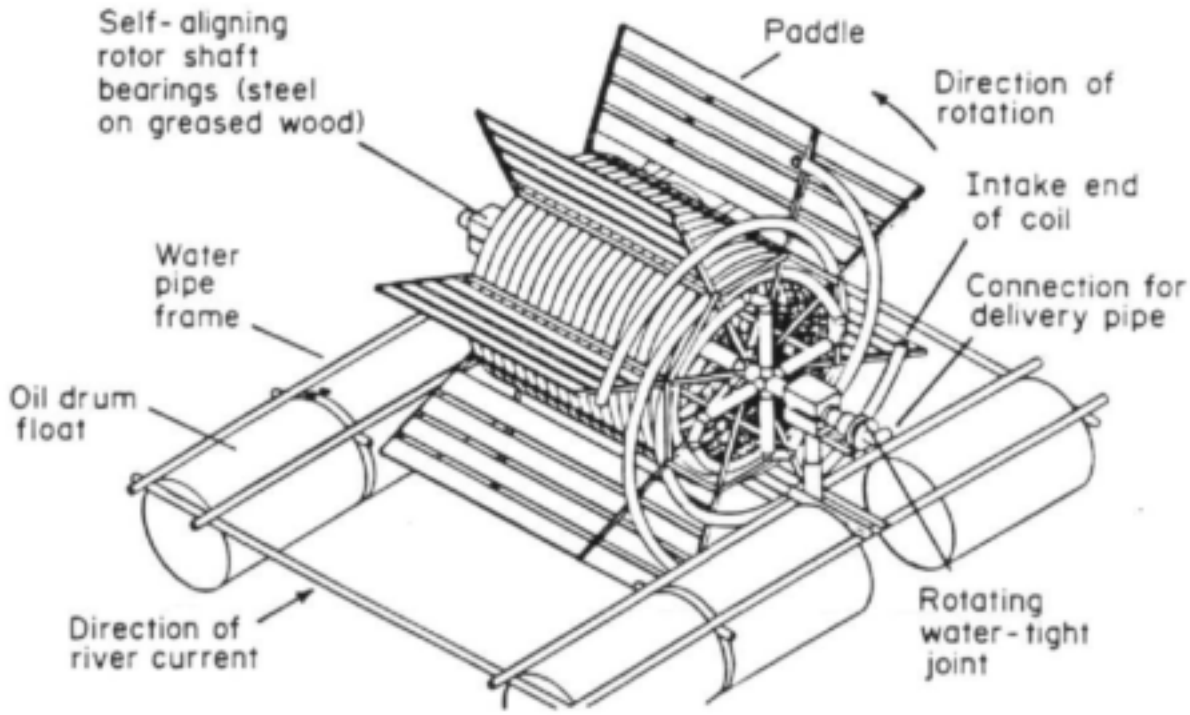


Figure 6.14 : Paddle driven coil pump

6.5.4 *Persian wheel*

The Persian wheel or *Saqia* has been designed to be powered by an animal harnessed to a beam which rotates in a horizontal circle. The horizontal rotary motion of the crown wheel is transmitted to vertical rotary motion of a water wheel by means of a lantern wheel. The principle is illustrated in Figure 6.15.

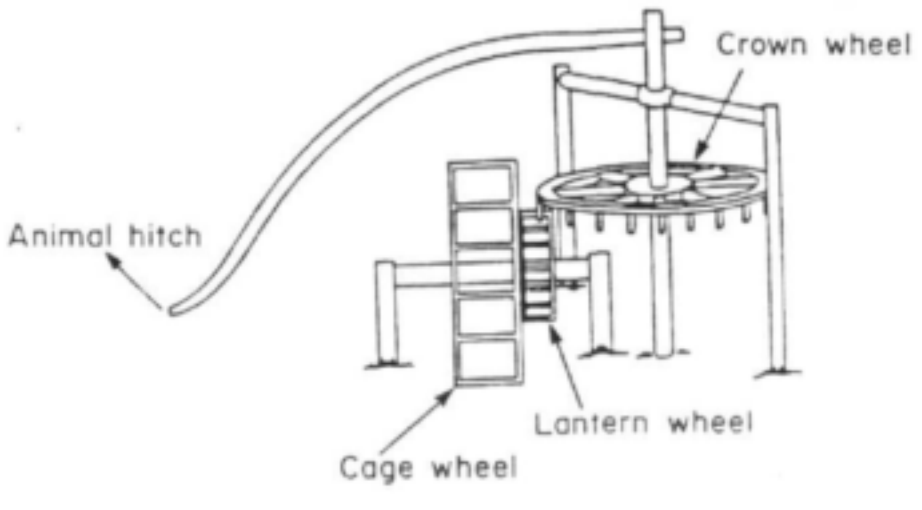


Figure 6.15 : Animal driven Persian wheel

6.5.5 *The 'Mhote'*

The *Mhote* was devised in India for withdrawing water from deeper wells using animal power. The animal moves in a linear direction pulling a rope, which is passed over a pulley. The water is raised vertically from the well by means of a bucket or animal skin vessel. When fully raised, the water is tipped into a launder or channel.

6.5.6 *The water wheel*

Water wheels are most commonly associated with micro hydropower schemes, in which the hydraulic energy of a moving stream of water is used to rotate the water wheel, the mechanical power delivered at the shaft of the water wheel is transmitted by means of gears or belts to machines or grindstones, thus performing useful work. Water wheels can also be used to drive a pump or to lift water directly. Two successful paddle driven water pumps have been constructed and are used for irrigation purposes on the Zambezi river. (ATI, P O Box 11070, Dorpspruit 3206). The paddle wheels of 5 metres diameter 2,4 metres wide revolve at a speed of 2,25 rpm due to the speed of flow of the river (about 1,7 m/s). The shaft of the paddle wheel is connected hydraulically, or mechanically by means of chain and sprocket, to a set of positive displacement rotary screw type Mono pumps. The continuous hydraulic output of the pump, as measured operating at 860 rpm, is 2,4 kW after mechanical and hydraulic inefficiencies have been taken into account. This is sufficient to pump 48 kilolitres per hour to a head of 18 metres continuously. Maintenance requirements are minimal.

In the Bulwer district of Natal, a water wheel driven by a stream is coupled directly to a pump which pumps water for irrigation to a storage reservoir at a higher elevation.

6.5.7 *The hydraulic ram*

The hydraulic ram utilizes the kinetic energy of a flowing stream of water to pressurize air stored in an air vessel. The energy stored in the compressed air is then used to force a smaller volume of water through a delivery pipe to a higher level than the water in the supply stream. A cross section of a hydraulic ram is shown in Figure 6.16.

Water flows down the inclined drive pipe into the body of the ram and through the delivery valve into the air vessel, compressing the air in this vessel. Most of the water (approximately 80 %) spills out of the waste valve, until the pressure on the rubber disc valve causes it to close. The impulse of this sudden closure causes a shock wave which further compresses the air in the vessel to a pressure in excess of the hydrostatic pressure of the water at the level of the ram. The delivery valve then closes, the waste valve reopens. The water trapped in the air vessel is pushed out through the delivery pipe by the pressure of the compressed air. The cycle then repeats itself.

The performance of the ram is determined by the working head on the drive pipe and by the vertical height to which the water must be raised. The volume of water and pipe sizes must also be taken into account when designing a ram installation. Hydraulic rams are manufactured by a manufacturer in Howick, Natal, or may be self-built using a suitable manual. Simplified hydraulic rams can be constructed in any engineering workshop. The maximum height to which a ram will lift water is between 100 and 150 metres depending upon size of ram and hydraulic conditions. Maintenance consists of checking and adjusting the spring pressure on the waste valve to achieve the correct frequency of the pulse. The air in the air vessel must be replaced periodically by draining the air vessel. Air becomes dissolved in the water under pressure. Hydraulic rams perform with little maintenance and no fuel for considerable periods of time. The continuous pulsing noise can be a source of annoyance and care should be taken in the siting of the installation to minimize this.

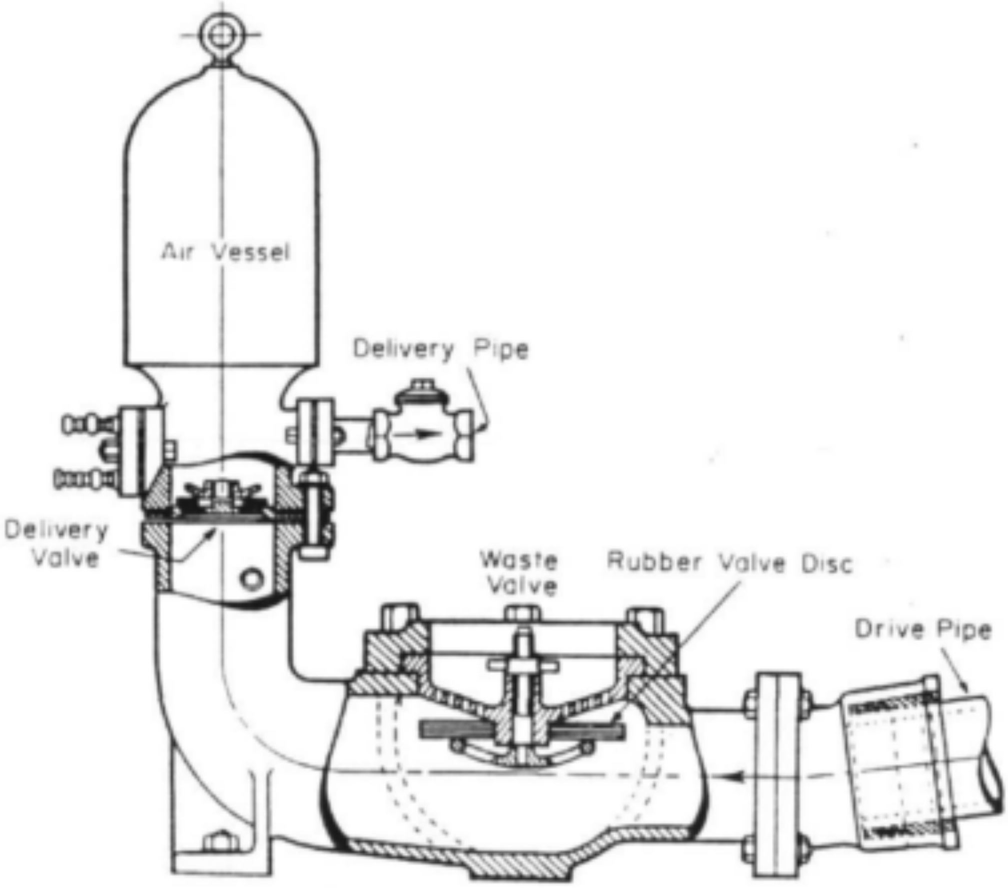


Figure 6.16 : Hydraulic ram

6.6 Prime movers

As mentioned previously, the machine or device that is used to power the pump is termed a prime mover. In turn the prime mover requires an energy source that it can convert into useful modes of energy in order to drive the pump. The sources of energy that are available for pumping water include:

hydraulic energy
wind energy
solar energy
animal and man power
chemical energy (stored in fossil fuel)
electrical energy from the national grid

Of the above, the last two only are non-renewable sources of energy. They are, however, the most popular at the present time for technological and economic reasons. Since a large percentage of rural water supplies rely on hand pumps, the energy source of man power (derived from food) must be considered as a significant one. The question of animal power has been briefly dealt with in the preceding section 6.5.

Various types of prime movers commonly used in water pumping schemes will be discussed.

6.6.1 *Manpower*

The energy source for manually operated pumps is chemical energy stored in food. One of the major needs identified early in the International Drinking Water and Sanitation Decade was the need for reliable, affordable and appropriately designed hand pumps. The World Bank, UNDP together with a large number of consultants such as the Intermediate Technology Development Group (ITDG) launched on a concerted effort to research and develop hand pump technology. The results of some of this work are reported in various World Bank Technical Papers, for example Technical Paper No 29 (1984) and Technical Paper No 48 (1985). One objective of the programme was to develop and evaluate the concept of the VLOM (Village Level Operation and Maintenance) hand pump. VLOM is an ideal and as such none of the pumps, of which many have been described in Chapter 6, have actually realised the VLOM ideal. Most of the problems associated with the VLOM concept are socio-political and not necessarily economic or technical. However, VLOM remains an important ideal. The pumps themselves must be evaluated against other criteria of acceptability and appropriateness, which include:

performance
reliability
durability
hydraulic efficiency
safety
affordability etc.

Many studies have been done on hand pumps worldwide. In South Africa, Wiseman and Eberhard (1988) carried out a study on handpumps and windpumps in KwaZulu and the Transkei.

The pumping power of man-powered pumps is discussed by O'Hea (1983). As would be expected, potential power is inversely proportional to the duration of the effort. Table 6.1 gives the potential power of a person of 60 kg mass. The potential power given are what could be expected from a person pedalling an ergometer (stationary bicycle).

TABLE 6.1: POTENTIAL POWER OF A PERSON (60 kg mass)

DURATION OF EFFORT	POTENTIAL AVERAGE POWER (watt)
0	1000
2 minutes	200
1 hour	100
10 hours	75

<u>Note</u> : These power figures are intended to represent approximately what may be expected from a person (who is used to the task) pedalling an ergometer in the open air but without the cooling effect of being on a moving bicycle. To repeat, they are intended as a standard, and are not invalidated by individual performances different from them.

The use of pedal pumps has been developed in the Vegnet foot pump (see Section 6.3.4). Mono Pumps have also developed a pedal pump, designed to operate at about 60 rpm, which drives a progressive cavity pump mounted on the pedal frame.

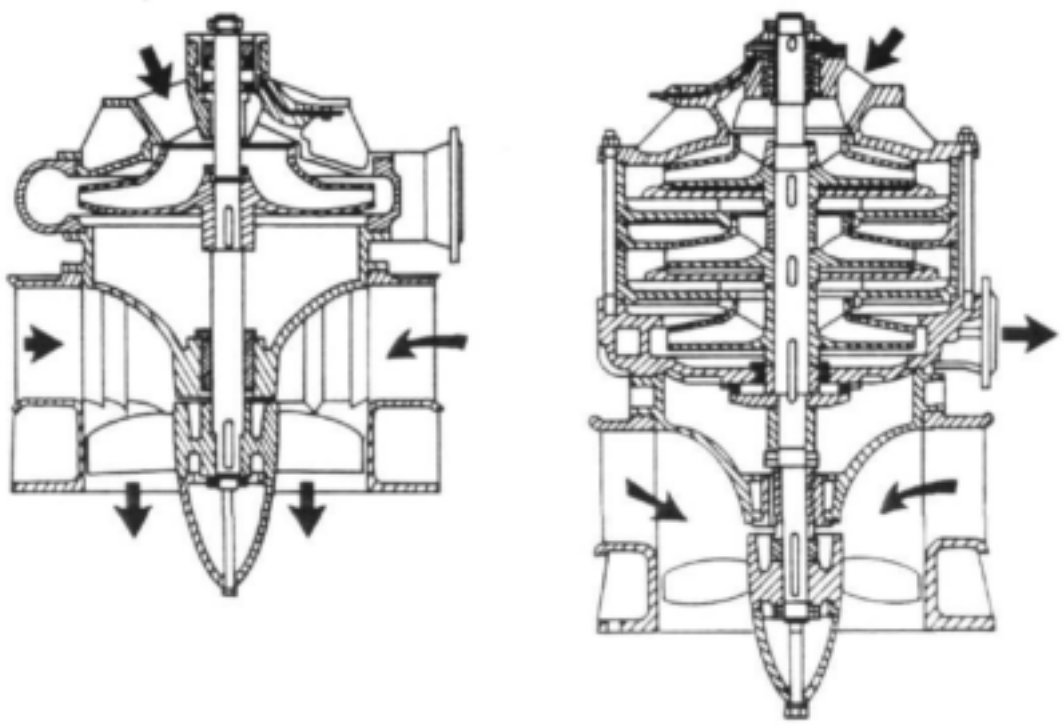
According to an ITDG panel (Appropriate Technology, 1982), "pedal-driven pumps offer the possibility of increasing a person's (power) output by perhaps 50 to 100 %, compared with hand pumps". The use of pedal-driven pumps is virtually unknown in South Africa at present.

6.6.2 *Draught animal power*

The power of draught oxen has been briefly discussed in Section 6.5.4. The potential for using animal power for water lifting is as yet untapped in the rural areas of Southern Africa.

6.6.3 *Hydraulic power*

As already noted water has both potential energy, due to its relative height above a datum, and kinetic energy due to the momentum of a flowing stream. Hydro-electric turbines, water wheels and hydraulic rams all utilize hydraulic energy, and have been discussed in the previous sections. There is one further item of technology that relies on hydraulic energy as its prime source of energy for pumping water. This is the Chinese Water Turbine Pump described by Shen' Lunzhang (1987) and depicted in figure 6.17.



single stage multi-stage

Figure 6.17 : Chinese water turbine pump

The water turbine pump is a combination of a hydraulic turbine and a centrifugal impeller mounted on the same axial shaft. It can be used for a wide range of water pumping applications, for hydro-electricity generation, or coupled to direct drive mechanical devices. Invented during the 1940's, the water turbine pump was further developed during the 1960's in a range of sizes for applications ranging from small streams to large rivers and tidal estuaries. They are manufactured in a range of sizes with head ratio's (discharge head to driving head) of 4 : 1 or 6 : 1, with discharge rates between 4 and 500 litres per second and hydraulic power outputs of 0,5 to 250 kW. There were over 60 000 sets installed in China up to 1987. The efficiency of the water turbine pump is better than an electric pump, because energy losses associated with energy conversions in the transmission lines, electric motor and pump are eliminated. Capital investment can be recovered in one or two years when compared to diesel pump operation.

As yet, this technology has not been applied in South Africa.

6.5.4 *Wind power*

Unlike micro hydro power which is a virtually untouched source of energy in Southern Africa, wind power, particularly for pumping water from boreholes, is a well developed technology. Wind energy potential varies from region to region and is also very site specific. Wind data for a region are normally compiled by the various Meteorological offices, which are coordinated by the Department of Environment Affairs. Wind speeds, directions, seasonal variations and frequencies are all of importance in assessing potential wind power sites. However, since mechanical power output is directly proportional to the cube of the wind speed, average wind speeds are of primary importance in making potential wind power assessments. In practice a windmill can never be more than 50 % efficient in converting wind energy to useful mechanical energy (Sorensen, 1979).

Wind energy can also be converted directly into electricity by means of a wind turbine. Problems associated with this form of energy conversion are the storage of electrical energy and conversion from DC to AC electricity. Windpumps can be driven either by means of a reciprocating mechanism, used to power reciprocating piston pumps (see Section 6.3.2), or rotary drives when used in conjunction with rotary spiral screw pumps (see Section 6.3.1). The former are the more common type of windpump employed in South Africa and are manufactured in South Africa by Climax, Southern Cross, Jooste Cylinder and Pump Co., and Nimric. Rotary windpumps are manufactured by Climax, SNS Rotors and Monomore (Pty) Ltd. The biggest manufacturer of windpumps in Africa is Climax, a division of Dorbyl Engineering, which has manufactured more than 200 000 units ranging in sizes from 8 foot (2,5 m) diameter sails to 14 foot (4,4 m) diameter sails.

In order to function reliably, windpumps require regular maintenance and greasing. Furling, braking and reefing mechanisms are essential to protect the sails from damage during strong winds. This is all the more important with the larger diameter sails. Other problems encountered with windpumps are the sudden changes in pumping rates caused by variations in wind energy. These sudden changes result in water hammer shocks in the delivery lines, especially for high lift pumps.

Some research work has been carried out in recent years at the Department of Mechanical Engineering at Wits and Natal Universities in reviving the ancient Cretan windpump. A Cretan windpump differs from a fixed sail windpump in that the number and pitch of the sails can be adjusted to suit wind conditions, in the same manner as a sailing boat. Another type of wind power head suitable for water pumping is the Savonius rotor, which comprises two half cylinders mounted vertically-opposed on a direct drive vertical shaft. Gearing and transmission mechanism are greatly simplified. The advantage of the Cretan and Savonius wind-heads are low cost of construction. This advantage is off-set by reduced efficiency, reliability and in the case of the Cretan windpump, durability.

6.6.5 *Solar power*

Direct conversion of sunlight into electricity by means of photo voltaic panels, has become an attractive proposition for water pumping in high solar intensity regions of the world. Unfortunately at this time due to the high import cost of the photo cells, and low efficiencies in both the photo-voltaic panels and in the electric motor/pump sub-systems, this technology is uneconomical, except perhaps in extremely remote conditions. The theoretical maximum efficiency of solar energy to electrical energy using crystalline silicone cells is 23 %. In practice far lower efficiencies have been achieved. Crystalline silicone technology has so far achieved a maximum efficiency of 5,5 % and these efficiencies have been recorded under optimum conditions. Efficiency is highly sensitive to temperature of the cells and as temperatures rise (as they do in practice), efficiency decreases markedly. Manufacturers of solar cells rate their panels in peak watts (Wp) (peak power) output. This output is rarely, if ever realised under normal operating conditions. When the solar panels are used in conjunction with motor pump sub-systems, further energy losses are incurred. The nett effect is that sub-system efficiencies, depending upon type of pump, motor and suction arrangements employed, typically have average daily efficiencies of between 25 % and 35 % (as measured by hydraulic output to electrical input; Mc Nelis, 1987).

In practice to achieve reasonable hydraulic output, large numbers of solar panels are required, which means the capital cost of the solar pumping system becomes very expensive. According to Mc Nelis (1987), photo-voltaic pumps are typically cost-competitive, compared to diesel powered pumps for rural water supply applications up to approximately 500 m^3/day (e.g. 25 m^3/day pumped to a head of 20 metres). This corresponds to a 800 Wp (peak watt) array or 20 panels of 40 Wp. Of course economic comparisons must be made by the designer for the specific equipment, site and cost parameters of each individual pumping installation, in order that meaningful comparisons can be made.

Overall systems efficiencies (solar energy to hydraulic energy) are in the order of 2 % or 3 %.

6.6.6 *Petrol, diesel power and the National Electricity Grid*

In areas remote from the national electricity supply grid it is often necessary to employ some form of diesel or petrol powered pump. The diesel or petrol engine can be directly coupled to the pump by means of a direct drive shaft or by means of belts and pulleys; or the engine can be linked to a generator which produces electricity, which in turn would be used to drive an electric motor coupled to a pump. Obviously direct linkage of the engine to the pump will result in less energy mode conversions and will consequently be more energy efficient. However, the conversion into electricity may be necessary or desirable, depending upon pump types, pumping configuration and operational considerations.

Economic comparisons need to be made between the use of petrol/diesel engines and other forms of power generation, particularly making use of the national electricity grid. This may involve extending the transmission lines to the pumping site. Where extension charges are not significantly large, electricity from the grid will provide a substantially cheaper source of energy for pumping, and other applications than diesel or petrol. In addition, electric motors are more reliable, much simpler to operate, and require substantially less maintenance than petrol engines and diesel engines.

A small to medium size electric motor will convert electricity to mechanical energy with an efficiency of between 70 % and 80 %. By comparison a diesel engine can convert chemical energy in the fuel to mechanical energy with about 40 % efficiency, whereas a petrol engine has an efficiency of about 25 %. Note we are here ignoring primary loss of efficiency in the generation and transmission of electricity from the power station, where the conversion-transmission efficiency of electricity from coal is about 30 %.

Diesel engines are as a rule more reliable than petrol engines. One of the most popular diesel engines for pumping water is the English Lister diesel engine which comes in a range of sizes from 3,3 kW to 94,0 kW continuous power output (speeds range from 1000 to 2500 rpm). Other stationery diesel engines available in South Africa include Robin, and Kubota Ltd, both manufactured in Japan. Small petrol engines suitable for pumping are available from Honda, Robin and Koshin Ltd, all of Japan; power outputs vary between 1 kW and 7,5 kW depending upon model and speed.

Electric motors are manufactured by many companies including GEC (USA), Siemens (West Germany) and AEG (South Africa). The range of sizes and technical specifications is vast and technical information can be obtained from the suppliers.

Belt driven pumps should be chosen whenever an engine or motor drive is selected. They are easier to operate and maintain than direct drive pumps and allow easy modification of the pump capacity by changing the drive pulley.

6.7 Management and planning aspects

One of the most important issues in pump technology is the maintenance question. Frequent inadequacies of maintenance programmes in developing countries has stimulated research and development of nearly maintenance free pumps, even though these are more costly. Experience has been that 30 to 80 % of pumps may be inoperative at one time. Experience has also shown, however, that despite what manufacturer's literature may claim, the maintenance-free pump does not exist. In fact where better quality pumps are installed, breakdowns often take considerably longer to repair than pumps where breakdowns are more frequent.

Historically the principle of pump maintenance has been to repair the pump once it has broken down, rather than to carry out scheduled preventative maintenance. However, by involving community management in the care and maintenance of pumps on a regular basis, substantial dividends can be achieved. An effective pump maintenance programme will result in :

- longer pump life
- less pump breakdowns
- cheaper pump repairs
- shorter breakdown periods
- reduced pressure on centralised maintenance units

There are three possible approaches to the aspect of pump provision and pump maintenance. These are :

i) Total village self-reliance in the manufacture and maintenance of pumps. This could be a possibility in the case of certain low-lift pumps. However, for village manufacture an adequately equipped workshop is required to ensure a high standard of finish on the manufactured product.

ii) Partial self-reliance, with factory made pumps, but with villagers at least partially responsible for maintenance. Emphasis has been given to this approach internationally during the past decade with the aim of producing the so called "VLOM" type pumps (i.e. village level operation and maintenance). When this approach is adopted, it is vital to make a considerable investment in community development, establishing responsibilities for the maintenance programme, and offering appropriate training.

iii) Elimination of all village responsibility. This approach has been applied often in the past. Pumps in this case must be heavy duty units with all bearings and mechanical parts totally enclosed, protected from dirt, and self lubricating. Many handpumps are designed with this application in mind. Such pumps, however, may cost as much as 8 to 10 times more than the simplest pumps available to do the job. Breakdowns which are sure to occur will take considerably longer to repair than with other approaches. It may be necessary to adopt this approach when the community is transient, or when physical conditions are such that only a high level of sophisticated technology will be adequate for doing the required pumping function.

This section has dealt with pumps, water lifting devices and energy prime movers. In virtually all water pumping systems water is lifted from a source, and must be pumped into a storage tank or reservoir. In the next section, storage will be dealt with, both in terms of storage requirements and various types of reservoir construction.

7. STORAGE AND DISTRIBUTION

The aspects of water storage and distribution are generally the most costly parts of a water supply scheme. Hence any savings in these aspects will usually result in significant savings for the whole project.

The elements of water storage and distribution systems are usually some or all of the following:

- bulk storage
- bulk water transmission
- intermediate storage
- distribution networks
- terminals

It is necessary to establish the basic criteria on which the design of the above elements depend before discussing each of these elements individually.

In order to provide a reliable, easily accessible water supply at adequate pressure and in the quantities required which will be functional for a reasonable number of years, it is necessary to estimate the future conditions which the water distribution system will be required to satisfy. These conditions are for some future population size and water usage pattern. The parameters used in the design process, for example the "design average daily water demand", the "design life", and the "design population", must be estimated as accurately as possible for the designer to make quantitative assessments in sizing and designing the water supply system. For rural populations this is not always as straight forward as in other areas. Populations may remain stagnant, or even decrease, due to the attraction of possible better conditions and opportunities in the urban areas. Alternatively, especially in rural areas close to major urban centres, the population may mushroom virtually overnight.

7.1 Estimation of water demand

Water demand is a function of the accessibility of the water source to the consumer, what the water is used for, and the number of consumers. There have been many studies conducted on water consumption amongst different sectors of the population in Southern Africa. The range of daily water consumptions lies between 8 litres and 300 litres *per capita* per day. In most rural communities where water has to be collected in containers and carried to the home for domestic purposes, but where extensive use is still made of natural water courses for washing clothes and personal hygiene, water demand is low (typically 10 *l*/c/d). Depending upon the distance from the water source to the house and whether or not a tap is provided in the yard or inside the house the figures given in table 7.1 can be used as a guide in estimating daily *per capita* water demand:

TYPE OF WATER SUPPLY	TYPICAL WATER CONSUMPTION (*l*/capita/day)	RANGE (*l*/capita/day)
communal water point (e.g. village well, public standpost)		
- at considerable distance (> 1000 m)	7	5 - 10
- at long distance (500 - 1000 m)	12	10 - 15
- at medium distance (250 - 500 m)	15	12 - 20
- at close distance (< 250m) :		
village well/handpump	20	15 - 25
communal standpipe	30	20 - 50
yard connection (metered)	40	20 - 80
yard connection (unmetered) (one tap at each household)	50	25 - 120
house connection (metered)		
- single tap	60	30 - 100
- multiple tap	150	70 - 250

Table 7.1: Typical domestic water usage

<u>*Notes*</u> *Sometimes, the number of households (families) in a community is easier to determine than the number of individuals, and the domestic water use can then be computed using an estimated average size of family.*

A spring can be considered as similar to a standpost, especially when it has been protected.

A climb of more than 60 m with a full container over a shorter distance should be considered as being similar to walking a considerable distance (> 1000 m)

The introduction of water flush toilets, baths, washing machines, etc. would increase the *per capita* daily water demand to anything between 60 litres and 250 litres *per capita* per day.

Other non-domestic water requirements which often have to be catered for include the following:

public facilities (high values for flush toilets):		
schools	10 - 15	litres/pupil per day
clinics	20 - 25	litres/patient per day
hospitals	220 - 300	litres/bed per day
bus stations	5 - 10	litres/user per day
agriculture (excluding irrigation):		
- cattle	25 - 35	litres/head per day
- horses/mules	20 - 25	litres/head per day
- sheep	15 - 25	litres/head per day
- pigs	10 - 15	litres/head per day
- poultry	15 - 25	litres/100 per day

When developing a water source it is important to estimate the number of persons or households who are likely to utilise the particular water source during its design life. The population growth is projected over the years representing the life span that the system is designed to operate for. Realistic figures for the annual rate of population growth (around 3,1 % p.a. in South Africa at present, but may be less in the rural areas as a result of migration to urban areas) need to be applied, together with some factor to take into account increased *per capita* consumption as residents become used to an improved supply. A certain degree of judgement is therefore needed in arriving at a "design average daily water demand" for the whole water supply system. The following example will illustrate this calculation:

Example

A spring is presently used by 20 households, the average number of persons per household from a survey was found to be 8,5 persons per house. The spring is to be protected and over the next 15 years it is anticipated that 80 % of the households will have a tap in the yard. What is the present daily water demand of the spring and what will it be 15 years hence?

* Present population (P) = 20 x 8.5 = 170 persons

* Projected population (P_1) in 15 years at a growth rate of 3.1 % p.a.:

  $P_1 = P(1 + r/100)^n$ where r = annual growth rate in percentage p.a.
  n = design period

  $$\begin{aligned} P_1 &= 170(1 + 3.1/100)^{15} \\ &= 170 \times 1.58 \\ &= 269 \text{ persons} \end{aligned}$$

* Present average daily *per capita* consumption based on half the houses being more than 200 m away from the spring and half less than 200 m

  $$= \frac{15 + 30}{2}$$

  $$= 22.5\ l/c/d$$

* Projected average daily *per capita* consumption based on 80 % of houses with yard taps and 20 % being more than 200 metres away from spring

  $$\begin{aligned} &= (0.8 \times 50) + (0.2 \times 15) \\ &= 43\ l/c/d \end{aligned}$$

* The present daily water demand is therefore estimated as

  $$\begin{aligned} &= 170 \times 22.5 \\ &= 3825\ l/d \end{aligned}$$

* The projected future water demand in 15 years is estimated as

  $$\begin{aligned} &= 269 \times 43 \\ &= 11\ 567\ l/d \end{aligned}$$

Consumption patterns have a significant daily variation; for example, with large communities in which a significant number of homes have house connections, water is consumed up to 17 hours per day. In smaller villages with no (or very few) private connections, the bulk of the water is drawn between 6h00 and 9h00, and again between 16h00 and 18h00. To cater for this, peak factors for the distribution system and storage requirements should be determined (based on the expected demand pattern) to ensure an adequate supply during peak demand periods. The following estimated factors can be used for cases where at least 75% of the homes have yardtaps or house connections. (Note that the peak factor can be reduced to 1,1 for the pipe distribution system when sufficient terminal storage is provided).

Population	Peak factor
10 000	1,5
<5 000	2
< 200	2,5

7.2 Estimation of storage requirements

There are fundamentally two reasons for providing storage in a water supply system. The first is to provide balancing storage where inflow and outflow rates differ, and the second is to ensure an emergency supply of water for a reasonable period of time in the event of a breakdown of some element of the water supply system.

7.2.1 *Balancing storage*

It rarely, if ever, happens that the rate of supply and demand for water will be the same. Domestic water demand shows a definite daily cycle with two peak periods (one in the morning and another in the evening). This peak factor (ratio of peak flow too average daily flow) will differ from one community to another. The figures given above are the figures suggested for design purposes, but it is preferable to determine the expected peak factor based on the expected demand pattern. For example, where a community draws their water primarily between the hours 06:00 to 09:00, and 16:00 to 18:00, the peak factor would be $24/5 = 4.8$. There may also be seasonal cycles in water consumption, with greater demand for water during the summer.

Where the installed capacity is unable to cater for the peak demand, the demand curve will flatten out, resulting in the actual peak demand being limited to the supply capacity of the system, extended over a longer period of time. This may be of some inconvenience to residents, but will not lead to a water shortage.

In instances where the rate of supply is more or less constant, as for example with supply from a spring, the storage requirement can be determined by plotting the accumulated daily water supply and the accumulated daily water demand. The required storage is the maximum difference between the latter and the former (see Section 3.3.4).

In instances where the rate of supply is erratic, or for only part of the day, as for example with supply from a windmill or other pumped water supply, it is necessary to provide sufficient storage to accumulate the required volume of water delivered over the pumping cycle. When the pumping cycle corresponds with the peak demand periods of the day, the balancing storage requirement may be less than for a constant water supply. The same method as used for determining the storage capacity above can be used for erratic or non-constant supplies.

An important consideration for a piped water supply scheme is that the closer the storage is located to the end of the pipelines, the cheaper the pipe distribution system can be. For terminal storage, the distribution system will only have to deal with average flows.

7.2.2 *Emergency supply storage*

In the event of a major breakdown or natural disaster, reserve storage capacity should be provided to ensure a supply of potable water for a period of time. The period of time for which provision must be made depends on the level of risk which the design provides for. Risk is a function of a number of factors, including the following:

- reliability of supply;
- availability of alternative sources;
- cost increment of reducing the risk; and
- consequences of supply failure.

The level of risk acceptable to the users and the availability of funds will determine the final storage capacity to be provided. A figure of 24 hours supply is often used as a rough guide.

7.3 Types of storage reservoirs

Storage reservoirs or tanks can come in a wide range of sizes and types of construction, depending upon the size of the water supply system.

7.3.1 *Open reservoirs*

In many places storage reservoirs for raw water supplies can be open bodies of water or dams. Due to the fact that the water surface is open to the atmosphere and sunlight, pollution and water loss through evaporation do take place. The growth of algae and water weeds can be a particularly serious problem. For these reasons open storage is not recommended for treated or potable water supplies. There is also the added risk of animals or people getting into an uncovered reservoir.

7.3.2 *Reinforced concrete reservoirs*

Reinforced concrete (RC) is the material most commonly used on reservoirs of 50 000 litres capacity upwards. The design and construction of reinforced concrete reservoirs must be carried out and supervised by an engineer. A different set of criteria, namely resistance against cracking of the concrete, are used in the design of water holding structures, as compared to other RC structures. Water holding structures are designed to the provisions of the British Code of Practice CP 2007. The provision of suitably designed water tight construction joints, is an integral part of the design of RC reservoirs. Most RC reservoirs are designed with RC roofs, supported if necessary upon internal columns. In recent years floating plastic sheet membranes have been successfully employed for covers on a number of large reservoirs in the Transvaal.

7.3.3 *Ferro-cement reservoirs*

The use of ferro-cement technology, which was initially developed for moulding the hulls of boats and ocean going yachts, has found increasing application in the past decades for the construction of water storage tanks. The technology is ideal for construction covered tanks with storage capacity ranging from 4 000 litres to 50 000 litres. Reservoirs of up to 150 000 litres capacity have been successfully built using this technique (Watt, 1978), although in the larger sizes it is usually not possible to provide a roof integral with the rest of the construction.

The technique is basically one of providing a mould (formwork) onto which reinforcing in the form of fencing wire, weldmesh or reinforcing steel is affixed. Next a layer of diamond mesh wire is fixed, onto which successive layers of a strong plaster mix (1 : 3 or 1 : 4, cement : sand) are applied and allowed to cure. The mould is then removed and the inside plastered in the same fashion.

The technique whilst sounding simple, requires a good deal of skill in the selection of the correct plaster mix, application and curing. It is an easy matter to construct a tank which will leak, but not so easy to construct a tank that is 100 % watertight, as it should be.

The use of a composite structure using a permanent masonry mould and a ferro-cement skin has been found to be successful on tanks of 100 000 litres capacity, resulting in savings of nearly 50 % as compared to RC construction.

The technique has been used widely throughout Southern Africa for constructing spring water storage tanks and rainwater storage tanks in rural areas. The technique of constructing a 4100 litre tank is well described in the video and technical guide of the Division of Water Technology, CSIR (1988), and has the potential for transference to personnel without high levels of technical skill.

7.3.4 *Sectional steel plate, bolted reservoirs*

Storage tanks from sectional steel plates can be constructed either at ground level or as elevated tanks using preformed plates which are bolted together on site with rubber gaskets. Such tank arrangements can be either circular or rectangular/square. Elevated tanks of this type are widely used by the South African Railways and other public works departments. Capacities of these preformed steel tanks range from about 6750 litres to 450 000 litres. Some manufacturers supply steel plates with glass fused to the inside steel surface for corrosion resistance and against attack by aggressive chemicals and effluents.

7.3.5 *Prefabricated water tanks*

Prefabricated water tanks of storage capacity between 4000 and 5500 litres are manufactured in South Africa in a range of materials. A comparison of retails prices of standard 4500 litre tanks was made in December 1987. These prices are given below, together with the retail price (November 1988) of a 5500 litre moulded polyethylene tank :

Galvanized iron tank	-	R653 (1987)
Fibre glass tank	-	R699 (1987)
Moulded polyethylene (5500 litres)	-	R926 (1988)
Asbestos cement tank	-	R989 (1987)

Prefabricated tanks are preferable in many instances, but the transportation costs and the costs of constructing a base slab may add substantially to the final cost. By comparison a ferro-cement tank of 4100 litres capacity cost about R424 to construct in December 1987 (Frans Diener, World Vision, personal communication).

7.3.6 *Unconventional construction of reservoirs*

The use of sloping-sided excavated tanks lined with plastic sheeting is another possibility for water storage reservoirs. The problem with such reservoirs is how to roof them.

An innovative idea was developed by Intermediate Technology Development Group (ITDG) and Oxfam for use in emergency refugee camps. The sloping sides of the excavated reservoir were lined with alternative layers of polyethylene sheeting and mud. This in turn was reinforced with 'sausages' made of sections of thin walled polyethylene tubing filled with a 9 : 1 sand : cement mixture and tied at each end. The 'sausage skins' were then pricked and soaked in water before use so that they set hard. Inside the tank were constructed a number' of 'beehives'. The volume of the tank between the beehives was filled with sand to reduce evaporation. Water was abstracted through the tops of one or more of the 'beehives' (ITDG, 1969).

The construction of hemispherical ferro-cement excavated reservoirs, capable of holding 52 000 litres of water is described by UNICEF (1986). The tanks are first excavated then lined by means of a ferro-cement skin. The roof is constructed from poles and thatch. A silt trap is provided at the inlet to the tank as it was intended to harvest surface run-off in a semi-arid area of Kenya.

Rectangular water tanks of 6 000 and 8 000 litres capacity have been constructed in Brazil for rainwater storage (Sleigh *et al*, 1986). The construction utilizes pre-cast concrete corner posts together with a moveable steel mould for the walls, into which a 12 : 1 sand : cement mixture is tamped. The inside of the reservoir is rendered waterproof by the application of a 3 : 1 plaster applied on top of a pure cement, water slurry. The tanks were covered using corrugated iron sheets.

A curved inter-locking concrete block tank, which comes in two sizes (2250 litres and 4500 litres) and which requires no reinforcing, can be built with a special block mould and hand powered vibrating table available from Parry Associates, England.

Precast reinforced concrete manhole rings have also been used for rainwater and spring storage reservoirs successfully.

From the storage reservoir, water is distributed to the consumers by means of pipelines, valves, metered house connections or public standpipes. This distribution network is collectively known as the water reticulation, and will be considered in the next section.

7.4 Types of distribution networks

In designing the distribution network of pipes for a particular community, the designer must take into account the size and spatial distribution of the community. An assessment must be made of the present and future water demand of different classes of consumers. This implies a knowledge of planning proposals for the area, the expected increase in the population and the design life of the system.

Pipe reticulation layouts are basically of two types (figure 7.1):

Spinal main with branch pipes
Closed loop or grid pattern reticulation

The former is typical of reticulation network in ribbon developments and in steeply-incised topography, where water pipes will generally follow the road layout, which in turn will follow natural ridges and water sheds. The latter network is found in grid pattern township layouts, where the street network forms a grid pattern or modified grid pattern of inter connecting streets. In practice many reticulation networks are a combination of the two.

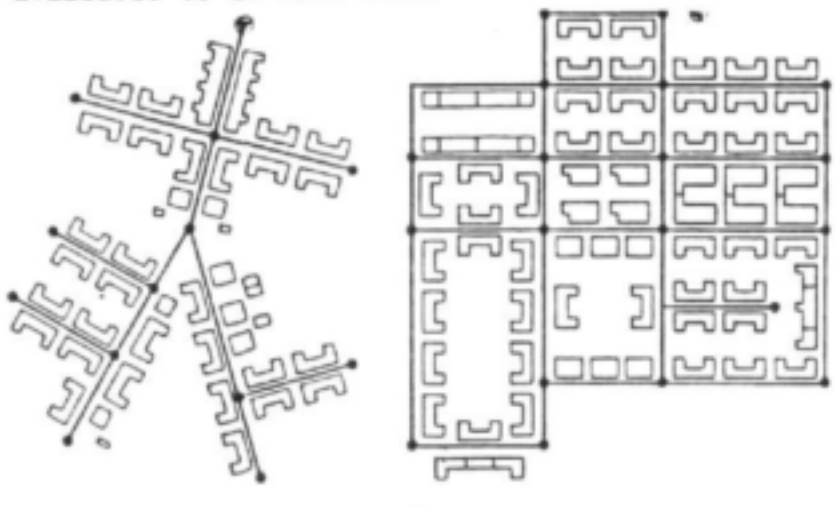
7.4.1 *Spinal main with branch pipes*

Branch networks are easier to analyse, where peak demand flows can be ascribed to each of the branches of the network. The flows are accumulative and the main spinal pipe will normally have to be increased in size progressively closer to the point of supply or the storage reservoir.

7.4.2 *Closed loop reticulation*

Loop networks, because water can enter the loop from either side, must be analysed using more sophisticated methods such as finite difference analysis or Hardy Cross analysis.

The hydraulic analysis of pipe networks is covered in most textbooks on water or hydraulic engineering. A number of computer programmes are commercially available to do this task.



A. Branched   B. Looped network

Figure 7.1 : Reticulation distribution patterns

7.5 Peak flow requirements

In Section 7.1 an example is given for estimating the present and projected future daily water demand of a typical rural community. To estimate the domestic water requirements for different water reticulation systems refer to table 7.1, where typical values are given by the International Reference Centre for Community Water Supply and Sanitation (IRC, 1981).

Alternately more detailed studies of water usage may be available from research institutions or consulting engineers. Alcock (1986) has for example examined household water consumption in a rural community in KwaZulu.

Once the average daily design flows have been determined for the projected future requirement, it is necessary to apply peak factors to take into account the daily and seasonal peak flows that can be expected in the pipe network (as referred to in Section 7.2). The average daily design flows are then multiplied by the peak factor to give the design peak flows. The location of the storage will determine what proportion of the peak flow must be taken by the pipe network. It is the design peak flows that are used in selecting the correct pipe sizes for the distribution pipe network.

7.6 Sizing pipes

In order to select the correct size and pressure class of pipes for the distribution system, it is important to know the difference in levels between the storage reservoir (or point of bulk supply) and critical points in the pipe network. This must normally be determined on the ground or by utilising an accurate typograghical map and carrying out an engineering survey. The design must ensure that there will be sufficient pressure, even under peak flow conditions, to supply water to the highest points within the layout. The pressure at these critical points is sometimes referred to as the minimum residual head and should ideally not be less than 14 m (138 kPa). In practice this may be difficult or impossible to achieve, and a lower minimum pressure may be more realistic. Botha (1985) suggests a minimum of 5 m (50 kPa). The optimum working pressure at any point in the reticulation network should be about 15 m (150 kPa) and ought not to be more than 75 m (750 kPa). Where excessive pressures are found within a pipe network it may be necessary to install a break pressure tank or pressure reducing valve. Alternatively the whole reticulation layout can be broken down into pressure zones, each supplied from its own intermediate service reservoir located at a suitable elevation.

When water flows through a pipe, there will be resistance to the flow of water due to friction with the pipe walls. The more water flows through the pipe, the more resistance there is, and the rougher the walls of the pipe, or the longer the pipe, the more energy is required. This resistance or energy is expressed in terms of meters of water, or energy pressure (kPa). This resistance is determined by the equation:

$$H_f = (b x l x q^p)/d^r$$

where H_f is the energy or resistance expressed in meters
l is the length of the pipe in meters
q is the flow rate of water in m^3/h
d is the internal diameter of the pipe im meters
b,p and r are constants representing the roughness of the pipe

For the various pipe types, b, p and r differ as follows

	b	p	r
uPVC or PE (plastic pipe)	$4,52x10^{-10}$	1,77	4.77
Asbestos	$4,18x10^{-10}$	1,79	4,79
Steel	$3,69x10^{-10}$	1,85	4,87

The hydraulic design of pipe networks requires a knowledge of hydraulics and related factors, and should ideally be performed by an engineer or technician with suitable experience.

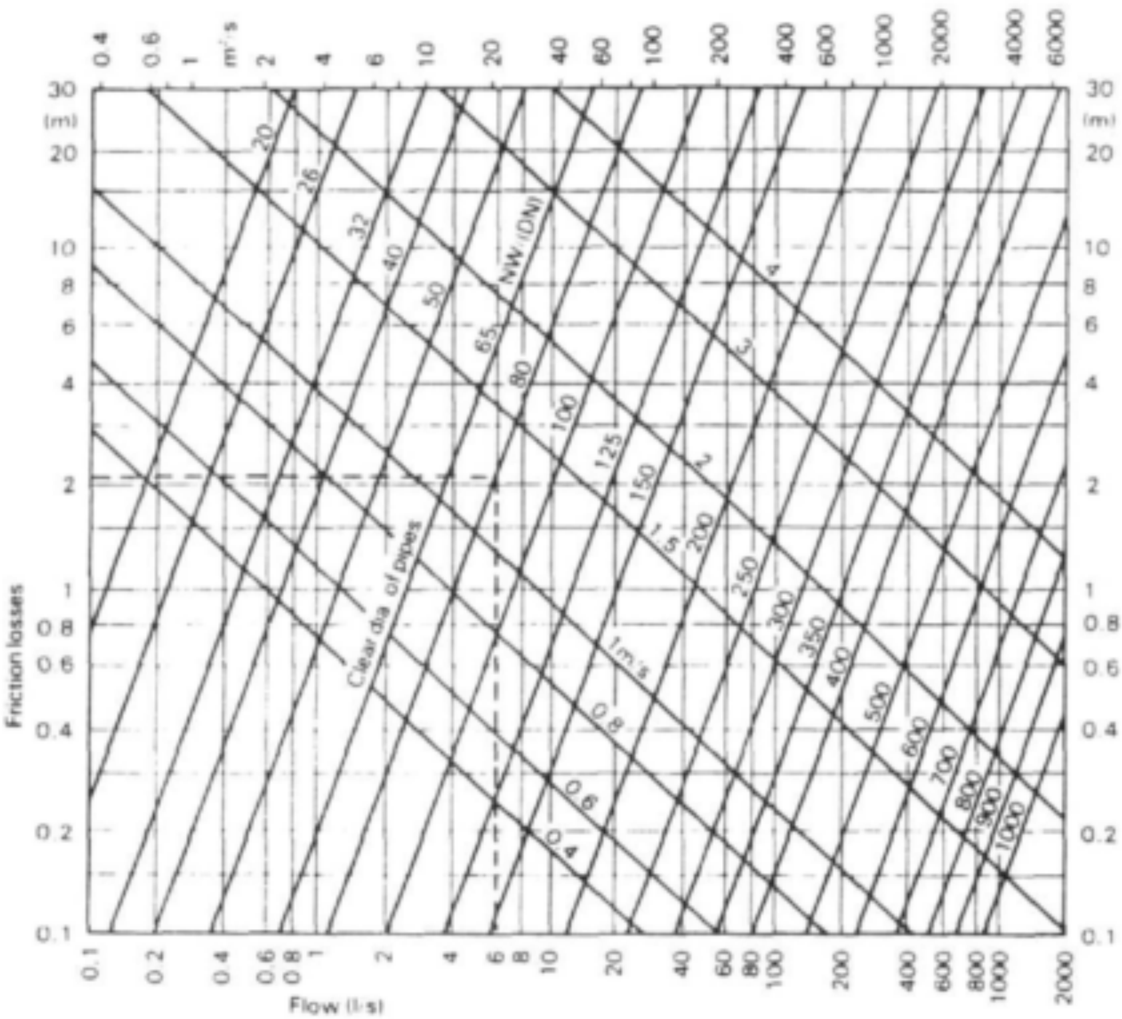
Pipes must be sized to accommodate the design flows under the prevailing head conditions. It is therefore important to know the difference in level between the intake and outlet of all pipelines. This difference is known as the static head or available head and is usually expressed in metres. From hydraulic charts or from the above hydraulic formulae it is possible to select the correct pipe size to convey the required rate of flow with the available head.

Figure 7.2 enables a rough selection of pipe sizes to be made when the pressure and flow conditions are known. Using the figure, the flow rate is selected on the bottom axis. A vertical line through this point will cross a number of diagonal lines representing different pipe sizes. For each pipe size crossing, a corresponding head loss is indicated on the verticle axis. Usually the smallest diameter pipe which gives a headloss which is still less than that which is available, is the pipe size of choice. The characteristic flows and headlosses will vary for different pipe materials, and hence the figure given should only be used as a guide. Characteristic curves can be obtained from the suppliers of pipes.

When selecting a pipe size for a pumping application, an economical design must be chosen which not only takes the cost of the pipeline into consideration, but also the pumping costs. Hence a small diameter pipe will have more resistance and require more energy for pumping, whereas with a larger diameter pipe resistance will be reduced and the energy costs for pumping will correspondingly be reduced.

7.7 Types of pipes

For many applications and particularly for smaller diameter piping (100 mm diameter or less) the use of plastic piping may be preferable because of ease of working with plastic pipe, and due to its flexibility to accommodate unevenness in trenching and ground conditions (e.g. settlement and heaving). It should be remembered though, that plastic pipes, particularly the stiffer uPVC, may puncture easily if not properly bedded in the pipe trench with fine material.



Friction losses in metres per 100m for a new pipeline of cast iron

For other types of pipe multiply the friction loss as indicated by the table by the factors given below:

New Rolled steel	0.8
New plastic	0.8
Old rusty cast iron	1.25
Pipes with encrustations	1.7

Figure 7.2 : Loss in head for the flow of water in straight pipes

The common types of plastic pipes are low density polyethylene, high density polyethylene, and unplasticized polyvinyl chloride. In addition to these there are polybutelene and polypropylene pipes on the market suitable for hot and cold water systems.

The other piping materials commonly in use in piped water reticulation systems are:

Galvanized iron
Mild steel
Asbestos cement pressure pipe
Reinforced concrete pressure pipe

Galvanized iron and mild steel piping, for use with threaded sockets, is manufactured to SABS 62/1971, and is available in sizes from 6 mm to 150 mm nominal diameter. Details of piping, jointing mechanisms and fittings can be obtained from major suppliers such as Stewarts and Lloyds or Incledon.

Asbestos cement pressure pipe is manufactured to SABS 286 and SABS 946, and is available in sizes from 75 mm to 600 mm nominal diameter.

Reinforced concrete pressure pipe is manufactured to SABS 676 (1969) and is available in sizes from 300 mm to 1500 mm nominal diameter.

For large diameter (greater than 450 mm nominal) pressure pipelines the use of continuously welded steel pipe or steel pipe with Viking-Johnson couplings is usually preferred. However, prestressed concrete pipe sections have been used successfully for major water aqueducts in South Africa in the past.

The choice of pipe material must be made by the designer, weighing up the criteria of relative purchase cost, cost of installation, handling characteristics, strength, durability, availability, corrosion resistance, maintenance and service connection requirements.

The piping normally used for small schemes (e.g. spring protection) is of either high density polyethylene (HDPE), low density polyethylene (LDPE) or unplasticized polyvinylchloride (uPVC). The former two come in 100 m rolls which makes it convenient to lay and reduces the number of joints. The uPVC comes in 6 m lengths, each length having one spigot and one socket end fitted with a self sealing rubber insert O-rings. HDPE and LDPE pipes are black, manufactured to SABS No 533 (Revised in 1982), and are marked with yellow and green ink respectively showing the type, pressure class and diameter. UPVC pipes are manufactured to SABS 966 of 1976 and are blue pipes marked in yellow ink to show the pressure class and outside diameter.

7.7.1 *Low density polyethylene pipe Type I and Agro-pipe*

LDPE is manufactured from material having a density of between 0,93 and 0,939 g/cm³ (it floats in water). It comes in two types. Type I is suitable for joining using internal nylon insert fittings, and gear clamps. A list of nominal diameters, pressure classes, inside diameters, wall thicknesses and mass per metre length is given in SABS 533.

LDPE pipes are used extensively for irrigation. Note there is also a non-SABS agricultural pipe (Agro-pipe) which, in addition to the nominal diameters in SABS 533, is manufactured in the following sizes:

10 mm nominal diameter
12 mm nominal diameter

This pipe, though usually less costly than LDPE, would generally not be suitable for domestic water supplies due to its thinner wall thicknesses and hence shortened expected useful life. This should be checked before ordering pipe.

7.7.2 *Low density polyethylene pipe Type II*

Type II is suitable for joining using external compression fittings (either uPVC, galvanized iron or brass); or externally threaded to BSP* (Classes 9 and 12 only) to take screwed brass or galvanized iron fittings. External compression fittings are more expensive than nylon inserts and gear clamps, but give a much better water tight joint (particularly under pressure). Gear clamps tend to rust when buried underground and are often broken when trying to carry out maintenance work on the joint. As spare gear clamps are usually not available this leads to all sorts of makeshift joints, many of which leak badly.

* BSP stands for British Standard Pipe thread and is the standard specification for water works fittings and plumbing fittings

7.7.3 *High density polyethylene pipe Type III*

HDPE is manufactured from material having a density of not less than 0,949 g/cm^3 (it is thus possible for HDPE to be lighter or heavier than water depending upon the density). Type III pipe is suitable for joining using internal nylon insert fittings and external clamps. It comes in sizes ranging from 15 mm to 50 mm nominal diameter. Pressure classes are restricted to Class 3 and Class 6 (corresponding to 300 kPa, and 600 kPa working pressure respectively).

7.7.4 *High density polyethylene pipe Type IV*

This pipe is manufactured to COD specifications and is suitable for joining using external compression fittings or externally threaded to BSP (Classes 6, 9, 12 and 16) to take screwed brass or galvanized iron fittings. Welded joints are also possible. As a rule this type of piping would be preferred over the previous types for domestic water supplies because of its greater rigidity, longer expected life, and use of external compression or screw-on fittings which minimise the likelihood of leaks. Sizes are given as COD and range from 20 mm to 110 mm diameter. Pressure classes range from Class 4 to Class 16 (400 kPa to 1600 kPa working pressure respectively). Details of available sizes and mass per meter are given in table 7.2.

It is worth noting that the manufacturers stress that under no circumstances should lubricant or any form of soft soap be used with a polyethylene pipe.

7.7.5 *Unplasticized polyvinyl chloride water mains pipe*

This pipe is commonly known as PVC pipe and of all the plastic pipes is probably the most suitable for domestic water supplies. It is blue in colour, light, easy to transport, robust and easy to lay. Being flexible it accommodates movements in the underground trench and can be laid on the curve, although care must be taken to ensure proper bedding. It is non-corrosive and smooth giving better flow-head loss characteristics than other non-plastic pipe materials. It is manufactured to COD specifications, making it compatible with a large variety of pipe fittings, and comes in sizes ranging from 20 mm to 400 mm diameter. Table 7.3 gives details for pressure classes ranging from Class 4 to Class 16 (400 kPa to 1600 kPa working pressure respectively).

OD		CLASS 4			CLASS 5			CLASS 6			CLASS 10			CLASS 12			CLASS 16		
Nominal	Tolerance	Min Wall Thick-ness	Nominal ID	Mass/ Metre (kg)	Min Wall Thick-ness	Nominal ID	Mass/ Metre (kg)	Min Wall Thick-ness	Nominal ID	Mass/ Metre (kg)	Min Wall Thick-ness	Nominal ID	Mass/ Metre (kg)	Min Wall Thick-ness	Nominal ID	Mass/ Metre (kg)	Min Wall Thick-ness	Nominal ID	Mass/ Metre (kg)
16	0.3	–	–	–	–	–	–	–	–	–	–	–	–	2.0	12	0.09	2.2	11	0.10
20	0.3	–	–	–	–	–	–	–	–	–	2.0	16	0,12	2.3	15	0.13	2.8	14	0.16
25	0.3	–	–	–	–	–	–	–	–	–	2.3	20	0.17	2.8	19	0.20	3.5	18	0.24
32	0.3	–	–	–	–	–	–	2.0	28	0.20	2.9	26	0.28	3.5	24	0.33	4.5	22	0.40
40	0.4	–	–	–	–	–	–	2.4	35	0.29	3.7	32	0.44	4.5	31	0.52	5.6	28	0.62
50	0.5	2.0	46	0.32	–	–	–	3.0	44	0.46	4.5	40	0.68	5.5	38	0.80	6.9	36	0.95
63	0.6	2.5	58	0.50	–	–	–	3.8	55	0.73	5.8	51	1,07	7.0	48	1.25	8.7	45	1.51
75	0.7	2.9	69	0.69	–	–	–	4.5	66	1.03	6.9	61	1,51	8.4	57	1.78	10.4	53	2.13
90	0.9	3.5	83	0.99	–	–	–	5.4	79	1,47	8.2	73	2,15	10.0	69	2.55	12.5	64	3.08
110	1.0	4.3	103	1,47	5.3	99	1,72	6.6	96	2.20	10.0	89	3,19	12.3	84	3.82	15.2	78	4.57
125	1.2	4.8	115	1.87	6.0	113	2.13	7.4	110	2.79	11.4	101	4.13	13.9	96	4,92	17.3	89	6.03
140	1.3	5.4	129	2.34	6.7	126	2.78	8.3	123	3.50	12.8	114	5.19	15.6	108	6.18	19.4	100	7.56
160	1.5	6.2	147	3.07	7.7	144	3.63	9.5	141	4.59	14.6	130	6.77	17.8	123	8.22	22.1	114	9.84
180	1.7	7.0	166	3.90	8.6	163	4,35	10.7	158	5.80	15.4	146	8.73	20.0	139	10,4	24.8	129	12.4
200	1.8	7.7	184	4.77	9.6	181	5.40	11.9	176	7.16	18.2	162	10.74	22.3	154	12.8	27.6	143	15.4
225	2.1	8.7	208	6.05	10.8	203	7.03	13.4	198	9.04	20.5	183	13.6	25.0	173	16.2	31.1	160	19.5
250	2.3	9.7	231	7.48	11.9	226	8.52	14.9	210	11.20	22.8	203	16.8	27.8	193	20.0	34.5	178	24.0
280	2.6	10.8	258	9.32	13.4	253	10.7	16.6	246	14.2	25.5	228	21.0	31.2	216	25.1	38.7	200	30.1
315	2.9	12.2	292	11.81	14.0	285	13.4	18.7	277	18.0	28.7	256	26.6	35.0	243	31.7	43.5	225	38.1
355	3.2	13.7	328	14.96	16.9	321	17.2	21.1	312	23.0	32.3	289	33.8	39.5	273	40.3	49.0	253	48.4
400	3.6	15.4	368	18.90	19.1	361	22.1	23.7	351	29.0	36.4	323	42.9	44.5	306	51.2	–	–	–
450	4.1	17.3	415	24.4	21.5	406	28.1	26.7	394	36.8	40.9	364	54.2	50.0	344	64.7	–	–	–
500	4.5	19.3	461	30.3	23.8	451	34.8	29.7	438	45.5	45.5	404	67.0	–	–	–	–	–	–
560	5.1	21.6	516	37.9	26.7	505	43.7	33.2	491	56.9	50.9	453	83.9	–	–	–	–	–	–
630	5.7	24.3	580	47.9	30.0	568	55.4	37.4	552	72.0	–	–	–	–		–	–	–	–
710	6.4	27.3	654	60.7	33.8	640	70.5	42.1	622	91.3	–	–	–	–	–	–	–	–	–
800	7.2	30.8	737	77.1	38.1	721	89.7	47.5	701	116.1	–	–	–	–	–	–	–	–	–
900	8.1	34.7	829	97.8	42.9	812	112.4	53.4	789	146.8	–	–	–	–	–	–	–	–	–
1 000	9.0	38.5	922	120.5	47.7	902	139.1	–	–	–	–	–	–	–	–	–	–	–	–

Table 7.2 : High density polyethylene type 4 (to SABS 533)

The smaller diameter pipes 20 mm to 40 mm diameter are suitable for joining using external compression fittings or solvent welded to external injection moulded uPVC fittings. They are used mainly for cold water plumbing. The larger diameters 50 mm and above come in 6 m pipe lengths, each length having an integrally moulded water tight socket complete with rubber O-ring. Bends, specials and other fittings are of a wide variety and range of materials many of which are manufactured to utilize the same mechanical rubber ring joint as the pipes. Adaptors are also available to join PVC pipes to asbestos cement (AC) pipes which are manufactured to different COD's and specifications. Bursts in PVC pipelines can be easily repaired using short lengths of pipe having double sockets, these are made by most PVC pipe manufacturers.

Flange adaptors are also available to connect pipe sizes from 50 mm to 250 mm diameter to flanges of the following specifications:

BS	- Table D
SABS 1123	- Table 10
SABS 1123	- Table 15

Saddles for house connections are manufactured from uPVC or malleable iron. 'Viking Johnson' couplings and steel bends and fittings are available for the larger diameter pipes (250 mm to 400 mm diameter).

Soft soap and lubricants are used to facilitate joining the spigots and sockets together when laying uPVC pipes.

Outside Dia. Size mm Buitedeursnee grootte mm	Klas/Class 4		Klas/Class 6		Klas/Class 9		Klas/Class 12		Klas/Class 16	
	mm	kg	mm	kg	mm	kg	mm	kg	mm	kg
20	-	--	—	—	--	—	—	—	1,5	0,81
25	—	—	—	—		—	1,5	1,03	1,9	1,29
32	—	—	--	—	1,5	1,34	1,8	1,59	2,4	2,08
40	—	--	1,5	1,70	1,8	2,02	2,3	2,53	3,0	3,25
50	1,5	2,17	1,8	2,58	2,2	3,13	2,8	3,96	3,7	5,13
63	1,5	2,76	1,9	3,46	2,7	4,87	3,6	6,40	4,7	8,23
75	1,5	3,30	2,2	4,78	3,2	6,87	4,3	9,11	5,6	11,66
90	1,8	4,74	2,7	7,04	3,9	10,05	5,1	12,96	6,7	16,75
110	2,2	7,08	3,2	10,22	4,7	14,85	6,3	18,58	8,2	25,11
125	2,5	9,16	3,7	13,41	5,4	19,36	7,1	25,12	9,3	32,37
140	2,8	11,49	4,1	16,67	6,0	24,13	7,9	31,40	10,4	40,64
160	3,2	15,03	4,7	21,92	6,9	31,78	9,1	41,34	11,9	53,26
200	3,9	23,02	5,9	34,46	8,6	49,64	11,3	64,40	14,7	82,100
250	4,9	36,23	7,3	53,62	10,8	78,27	14,2	101,82	—	—
315	6,2	57,93	9,2	85,31	13,6	124,67	17,8	161,50	—	—
355	7,0	74,10	10,6	111,14	15,4	151,98	—	—	—	—
400	7,8	93,30	11,7	138,37	17,2	172,48	---	—	—	—

Table 7.3 : PVC pipe specifications (minimum wall thickness and mass per 6 meter length)

7.8 Bedding of pipes

Bedding of the pipe is the most important part of pipelaying to ensure long life. The pipe barrel must rest evenly on the bottom of the trench for its whole length. This is achieved by levelling the trench bottom very carefully, firstly by hand with a spade, excavating holes in the base of the trench for joint sockets. If the trench is in rock or hard material, a level bedding of soft material will need to be selected from the excavated material or brought in from elsewhere. Sand or similar material is often used for this. Pipes must never be supported on blocks or bricks as this concentrates the load from the backfilled trench onto one point of the pipe leading to certain fracture at this point. Hard high spots in the trench bed leads to similar point loads on the pipe. This is a most frequent cause of failure of pipelines.

Backfilling round and under the pipe is very important in order to give support to the pipe throughout its life. This is particularly important when laying plastic and AC pipes.

7.9 Valves, water meters and other ancillaries

Control valves are necessary on main outlets and branches and the intersection of loops, in order that the flow of water can be stopped to carry out maintenance and repairs on different sections of the pipework. In addition air valves are required on high points of pipelines to release entrapped air (where there are no taps or house connections); and scour valves are required on low points to release settled sediment that can accumulate in the pipeline. Pressure reducing or pressure sustaining valves may be required to reduce or to sustain pressure at critical points in the pipeline. Pressure relief valves or surge tanks may be required to dissipate pressure waves caused by water hammer in long pipelines. Non-return valves are required on rising pumping mains to prevent the back flow of water when the pump is switched off.

Individual house connections and public standpipes should be metered and provided with isolation valves in order to collect water revenue. Bulk water meters may be installed at strategic points within the network in order to provide checks on consumption and leakage. There are valves and water meters designed for every conceivable application and size and major suppliers such as Stewarts and Lloyds or Incledon should be consulted before ordering.

7.10 Terminals - Public standpipes and private connections

7.10.1 *Public standpipes*

A public standpipe is a suitably supported water pipe, connected with a water distribution system and terminating in a tap or faucet, which is located at a public site, and from which water may be drawn for domestic use. These installations are sometimes referred to as public standposts, public hydrants, public taps, public fountains or communal water points.

Whilst the installation of a private house water connection may be the objective of most people, the installation of public standpipes may often be an economically feasible intermediate step in achieving that objective. The nature of many rural and peri-urban settlements, being scattered and not formally set - out on the ground, means that the provision of individual house connections would be excessively expensive and not feasible from an economic point of view. Nevertheless, the installation of a public standpipe water supply system can achieve many of the objectives of an improved water supply system at a lower cost than other alternative systems. A typical public standpipe is depicted in Figure 7.3. The design of public standpipe water supplies, and the economic and financial aspects, are dealt with in the World Health Organization/International Reference Centre for Community Water Supply Technical Papers 14 (1979) and 13 (1979) respectively.

Where a distribution system has a combination of public standpipes and private house connections, the system is described as a mixed system. This is often the situation which occurs when a system is being gradually upgraded to individual house connections. Cost recovery from mixed systems is generally more problematic than that from purely public standpipe system or purely private connection systems.

The provision of public clothes-washing facilities can be incorporated into the design of public standpipe systems, although careful attention to community wishes and practices is advisable before designing such facilities. Adequate drainage must be provided at all public water points. This cannot be over stressed because usually little provision is made for drainage.

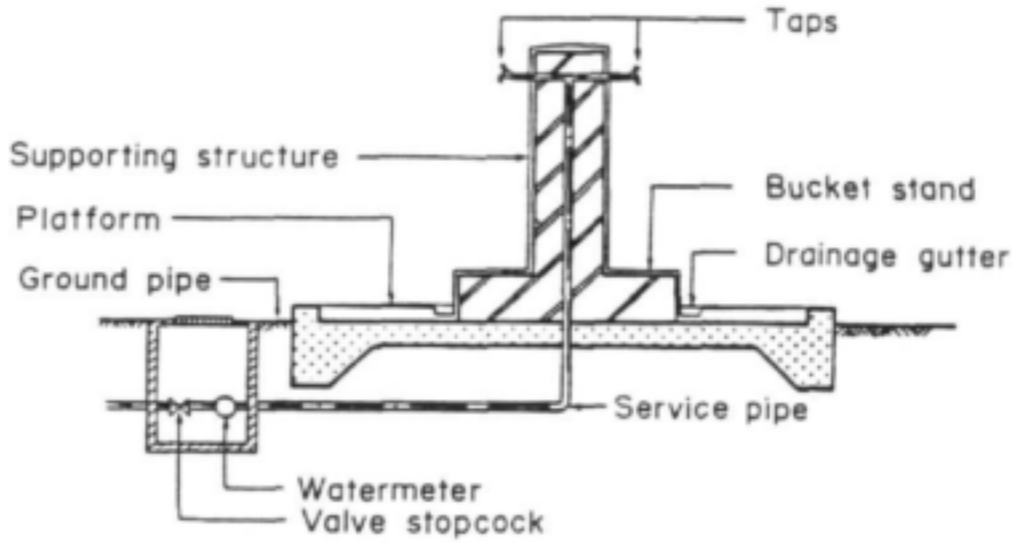


Figure 7.3 : Public standpost layout

7.10.2 *Private house connections*

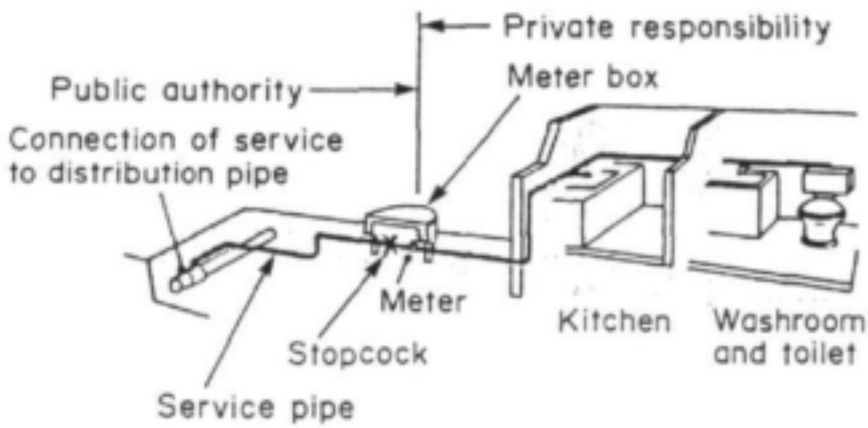


Figure 7.4 : Typical house connection layout

Where economically justified, all or some of the houses can be connected to the water distribution system by means of private house connections. In South Africa the provision of a house connection by the public water authority consists of providing a connection to the service water main, a length of small diameter pipe (20 to 25 mm diameter is normal) at the boundary of the property, and the installation of a meter and isolating valve (stop cock) in a suitable meter box either outside or just inside the property. The provision and maintenance of all underground pipes within the property boundaries and house plumbing and fixtures is the responsibility of the house owner. These costs are normally built into the cost of the house.

An alternative for low cost housing or informal housing upgrading schemes would be to provide in addition to the above a yard tap, referred to as a yard connection. Adequate drainage would also have to be provided for the tap and washing areas in the yard. Refer to Figures 7.4 and 7.5 for typical layouts of a house connection and a yard connection respectively.

The question of public <u>viz a viz</u> private responsibility in the question of yard connections would have to be decided upon. The cut-off point could be at the meter, as for private connections, or could extend to include the tap (standpipe) or even the drainage arrangements such as galley and soakaway as well. The question of maintenance is affected by the decision taken in this regard.

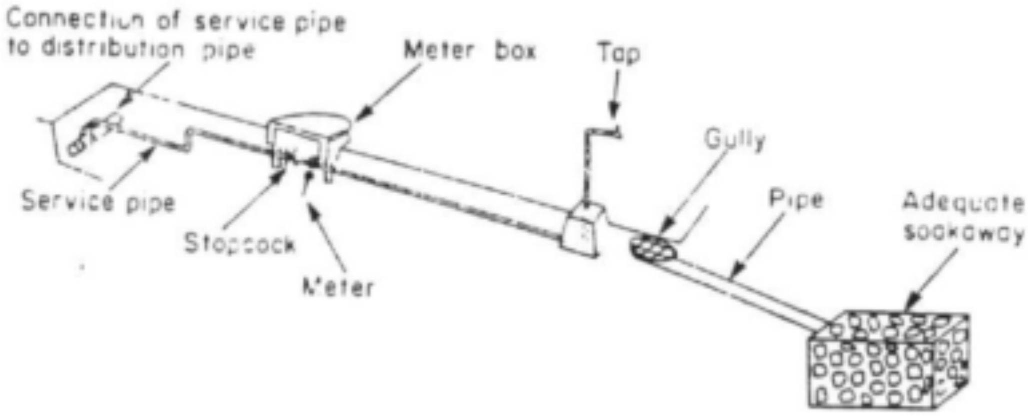


Figure 7.5 : Typical yard connection layout

7.11 Management aspects

Good management of a water storage and distribution system requires a commitment to continual monitoring of the system. Storage and distribution systems are usually "live" and grow and change with time.

All systems deteriorate with age, but the rate of deterioration can differ significantly, depending on local conditions and degree of monitoring and surveillance.

Records relating to consumer complaints, mainlaying, servicelaying, burst mains, burst service pipes, general maintenance of equipment, leakage control, residual pressures, daily demands, night line measurements, service reservoir levels, booster outputs, and water quality from the source, treatment works, and in the distribution, should be kept on a regular basis. This ensures that trends are picked up and problems can be related to other aspects of the system. Table 7.4 gives the typical problems encountered and possible causes.

DISTRIBUTION SYSTEM PROBLEM	POSSIBLE CAUSES
Burst mains and service pipes	- excessive pressure - surge problems - internal corrosion/aggressive water - external corrosion
Aggressive soil	- inferior materials - ground movement
Intermittent supply	- burst mains and service pipes - hydraulic capacity of mains and/or service pipes - tuberculated deposits and general mains debris - equipment failures (pumps, PRVs, etc.) - illegal water usage - maintenance work
Poor pressure	- hydraulic capacity (mains, service pipes, pumps, etc) - tuberculated deposits - illegal demands - maintenance work - topography
Discoloured water	- sourcewater treatment - pipeline materials/linings - corrosion debris - mains - - service pipes - hydraulic balance points - dead-ends - mixing of different source waters - excessive remedial flushing
Excessive leakage	- excessive residual pressure - internal/external corrosion - pipeline materials/linings - mainlaying techniques - leakage policy - active or passive - intrinsic characteristics - climate and soil type
Tastes and odours	- source water quality - sourcewater treatment - chemical dosing, e.g. excessive chlorine
Animals (*Asellus*, etc)	- treatment inadequacies - excessive food (mains debris) - pipeline materials and linings

Table 7.4 : Tell-tail signs and accompanying causes of problems (after Lackington 1988)

Table 7.4 indicates that before undertaking remedial works it is important to examine a variety of factors in order to identify the true cause of a problem. In the absence of such an approach, expensive remedial action can prove ineffective and abortive.

SECTION 8 - SANITATION SYSTEMS

8.0 Sanitation in general

A major motivation for development agencies to improve water supplies and sanitation is to raise the general health standards of the population they serve. The World Health Organization (WHO) estimates that as much as 80 % of all disease may be due to inadequate and poor quality water supplies and sanitation.

The safe disposal of human excreta is of the utmost importance in the control of infectious and other communicable diseases. This is something that is understood consciously or unconsciously by most people. However, the disposal of sullage or grey water from domestic washing is also an important part of sanitation. Because of the importance of the safe disposal of human excreta, the building of appropriate sanitation systems often is considered synonymous with improving sanitation. However, experience has shown that this is not necessarily the case; and that unless there is an effective primary health care education programme, the installation of improved sanitation facilities alone may not result in improved health. The importance of a holistic approach to health improvement cannot be overstressed.

Guinea worms around poorly drained public standpipes, mosquito breeding in waste water drains, and hookworms infesting the feet of people using fouled latrines, are but three instances where improved facilities can lead to worsening health conditions for the community. Primary health care education is aimed therefore, amongst other things, at educating people in the importance of washing hands after defecation, washing fresh fruit and vegetables, keeping the yard free from the excreta of small children and animals (toddlers have a habit of putting things into their mouths), encouraging and educating children to use toilets, etc. All of which are essential if the benefits of improved water supply and sanitation facilities are to result in improved health.

As with water supply, sanitation must be seen as a complex system of interrelated factors.

The successful sanitation system is therefore to be found in the successful organization of the factors affecting the health and social organization of the community.

The provision of suitable sanitation technology is but a part of this system. The criteria for assessing the suitability of sanitation technology for a particular application are:

reliability
acceptability
appropriateness
affordability

With these four criteria in mind, sanitation systems can be assessed for their suitability in rural areas.

8.1 Sanitation : technological options

There are fundamentally three main criteria for distinguishing between types of sanitation systems. These are tabulated in Table 8.1, and are as follows:

1. Need water or do not need water
2. On-site or off-site treatment and disposal
3. Aerobic or anaerobic treatment of wastes

8.1.1 *Need water or do not need water*

An important consideration when selecting a sanitation system is whether the latrine requires water for flushing and waste treatment, and if so how much water is required. If water is only obtainable from some distance from the home, and/or if the supply is very limited, the supply of water to the latrine may place a considerable burden on the users. Alternatively the latrine will not be able to function properly, and even become a health hazard. Systems which do not require water are therefore more appropriate for rural areas where water is not available readily and reliably.

Strictly speaking the distinction here is in the average moisture content of the waste materials not in the method of anal cleansing or flushing system employed. Composting is an example of a dry disposal system. It takes place with an optimum moisture content of about 60 %. Since human excreta including urine has an average moisture content of about 98 %, to achieve ideal dry composting conditions, the water content has to be reduced. This is achieved by either excluding urine and water for anal cleansing and flushing, or by seepage of excess water into the ground. Another approach is to mix in sufficient quantities of relatively dry material such as ash, grass cuttings or soil to achieve an optimum moisture content.

Wet systems by contrast would include those where the degradation, stabilization process takes place in an aqueous solution, for example, septic tanks, aqua privies and conventional water-borne sewerage works.

8.1.2 *On-site or off-site disposal systems*

The question of whether the final products of the treatment process are disposed of on or off the site depends primarily on the density and organizational ability of the rural settlements. To a lesser degree it depends on geological, topographical and other factors. In most rural communities organizational infrastructure is limited and central disposal facilities would place a large burden of responsibility on the community. However, for large institutions in rural areas (hospitals, boarding schools and colleges, etc.) it may be feasible to consider off-site treatment and disposal.

8.1.3 *Aerobic or anaerobic system*

The criterion here is on the environment and hence the bacteriological ecosystem in which the degradation and the stabilization of the waste products occurs. In an aerobic system oxygen is required (either as free oxygen or dissolved in solution) by the bacteria for the breakdown of waste products and for cell growth. In an anaerobic system the bacteria use other sources of oxygen and energy for these activities. It is generally accepted that there are three major metabolic groups of bacteria in anaerobic digestion:

1. fermentative bacteria
2. hydrogen-producing, i.e. proton reducing (called acetogenic) bacteria
3. methanogenic bacteria that convert acetate and hydrogen to gaseous products (methane and carbon dioxide).

In some aerobic systems atmospheric oxygen is sufficient to sustain the process. For example in aerobic composting, ventilation and turning of the compost is sufficient to achieve biodegradation to stable end-products. In other systems such as the activated sludge process, oxygen must be continuously introduced into the system by mechanical means. Such systems tend to be energy intensive, sophisticated and require a high degree of technology and management.

Anaerobic systems are normally to be found where sealed containers are used to collect the waste material or where the depth to surface area ratio is large. Examples are septic tanks, composting vaults and anaerobic deep lagoons.

TABLE 8.1 : TYPES OF SANITATION SYSTEMS

WATER REQUIREMENTS	DISPOSAL SYSTEM	AEROBIC PROCESS	ANAEROBIC PROCESS
none	on-site	*pit latrines *VIP latrines *composting latrines	*DCV latrine
	off-site	*bucket systems *chemical toilets	
required	on-site		*Septic tanks *Aqua privies *Anaerobic digesters
	off-site	*water borne sewerage *conservancy tank and vacuum tanker	*Above 3 systems with small-bore sewer

NOTE:

1) There is some overlap between aerobic and anaerobic systems. For example double vault composting (DVC) latrines work initially under aerobic conditions but once filled-in the final decomposition takes place under anaerobic conditions. In the case of aqua privies with small-bore sewers, the initial treatment on-site takes place under anaerobic conditions with the effluent being transported normally to oxidation ponds where the effluent is treated aerobically.

2. Terms and abbreviations are explained in the text.

8.2 Resource recovery

The potential for resource recovery from human and animal excreta is high. In Asia and particularly in China, human excreta has a positive economic value and has been used in the manufacture of compost for millennia. However, the primary objective of sanitation must always be to improve public health. Resource recovery, although technically feasible and desirable from an economic and environmental point of view, should be regarded as a secondary objective.

The chief recoverable elements from excreta are nitrogen (N), potassium (K) and phosphorous (P) in various forms. These are all essential inputs for agriculture or aquaculture and can be assigned economic values. Another potential product of anaerobic digestion is biogas, a mixture of methane and carbon dioxide. However, for the economic generation of biogas animal manure is preferred.

It is estimated that most underdeveloped countries produce seven or eight times more N, P and K in waste form than is required for agricultural purposes (Winblad and Kilama, 1980).

8.3 Types of sanitation technologies

From the three main categories of sanitation systems outlined above, a number of technological options are possible. Some of the more appropriate systems for rural areas are described below. For a more comprehensive study, the reader is referred to "Appropriate Sanitation Alternatives" (Kalbermatten *et al*, 1980b).

8.3.1 *Pit latrines (including SanPlat)*

Pit latrines are the first improved sanitatation system which have been used by communities for many centuries. However, in order to ensure that such latrines are hygienically and physically safe and aesthetically pleasant to use, a number of precautions must be taken in their construction and use.

Whilst the standard pit latrine appears in many guises in the developing areas of Southern Africa, the standard of these toilets is in about 90 % of the cases totally unsatisfactory (Friedman, 1983; Alcock, 1986). Flies have ready passage in and out of the pits, and smells and poor construction make their use less than desirable, particularly by children.

In a study of the community in the Umgeni Valley, Friedman (1983) found that only 10 % of the households had reasonably adequate toilets with an average of one such toilet per 95 people. Common defects are:

. the creation of breeding sites for flies and mosquitoes;
. smell;
. structural instability;
. unsanitary conditions;
. inadequacy of design for small children;
. lack of privacy.

The result is that although in some areas virtually every household has a pit toilet of some sort, their usage is low, particularly by small children who in many cases probably never use them at all.

There are some basic principles of pit latrine designs and use which must be adhered to ensure that the functionality of the latrine is acceptable.

These principles are:

- The pit should be a minimum of 2,5 metres deep if the diameter of the pit is 1,2 metres; or 2,1 metres deep if the pit is 1,5 metres in diameter. Allowing the pit to fill within 0,4 metres of the bottom of the slab, the above dimensions give an effective volume of 2,9 m^3.

- The pit should not penetrate the water table.

- The pit or at least the top of the pit must be lined if necessary by means of a collar to prevent collapse and the ingress of surface water.

- The floor slab must be smooth with a slight slope to drain into the pit or away from the pit when cleaning down with water.

- The toilet should have a pedestal or squat plate (depending on custom) of sufficiently small overall size to prevent small children from falling into the pit, and must be easy to clean.

- The inside of the toilet should preferably be relatively dark to prevent flies which get into the pit from being attracted by the light and flying out of the pedestal/squat plate.

- There must be an unrestricted flow of air into the toilet superstructure.

- The floor slab must be above the natural ground level to prevent ingress of surface water.

- The toilet must be attractive and well built to encourage use.

- The superstructure should preferably be constructed so that it can be moved to a new location when the pit fills, or alternatively that the materials used in the construction can be recycled.

- The toilets must afford complete privacy to the occupant.

- Small amounts of water added assist in decomposition and prolong the life of the pit (but not more than 1 litre per user per day).

- External access into the pit should be provided for in the event of pit emptying equipment being available.

SanPlat Latrine

A pit latrine slab has been designed which is low cost yet provides for a low odour or practically odour and fly free safe latrine which can be used in rural areas. This is the so called SanPlat (Sanitation Platform) latrine designed by Björn Brandberg of Sweden and now used widely in Africa. The platform or slab is cast in a slight dome shape and hence no reinforcing is required, and contains a keyhole shaped squat hole and two footrests to ensure accurate use of the latrine. A tight fitting lid seals the latrine when not in use. The basic slab is shown in figure 8.1. A smaller slab can also be used to upgrade existing latrines which have an existing floor but unsatisfactory squat hole. The SanPlat does not have a pedestal which may make it less acceptable in some areas.

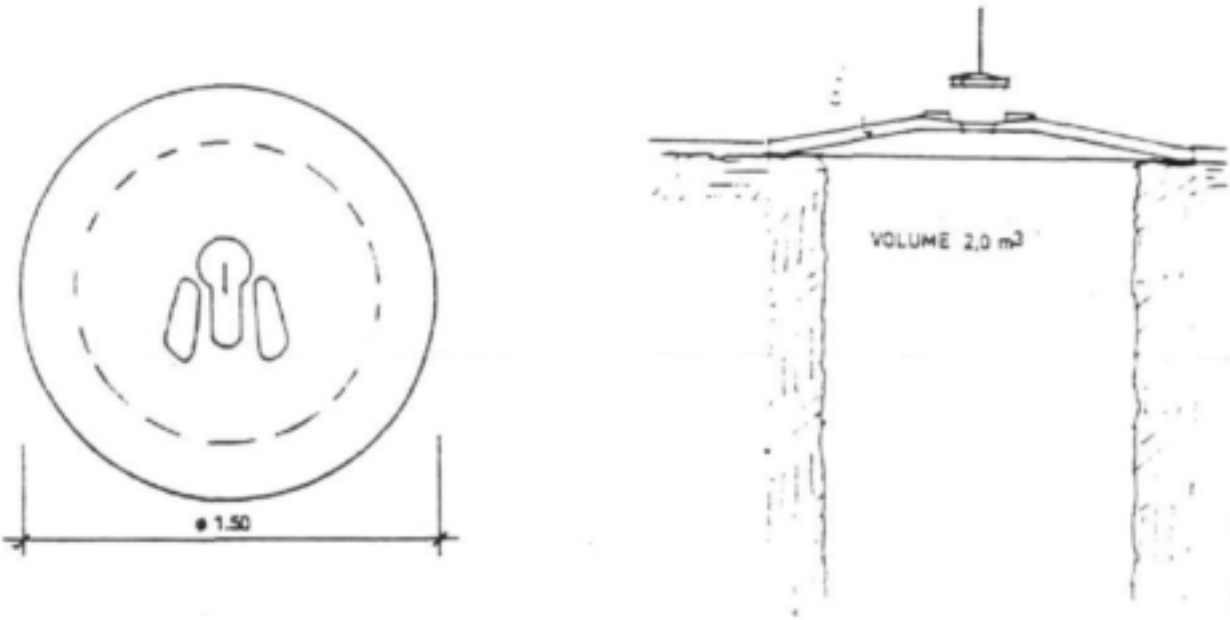


Figure 8.1 : SanPlat latrine slab

8.3.2 *Ventilated improved pit (VIP) latrines*

The VIP latrine was developed by the Blair Research Institute, Zimbabwe, and tested from 1973 to 1976. (The precursor to the VIP latrine was the Reid's Odourless Earth Closet (ROEC) developed in South Africa in the 1940's). The VIP is shown in Figure 8.2. This latrine is a standard pit latrine but includes a ventilation system to control smells and flies. The latrine structure must be well ventilated but dark inside to discourage flies from coming out of the pedestal. The toilet must be fitted with a vent pipe from the pit of a minimum diameter of 150 mm with a non-corrosive fly/mosquito-proof screen stretched evenly over the top end of the pipe. smells are drawn out of the pit through the vent pipe, and flies will try to exit the pit through the vent pipe as this is the source of light in the pit. The fly/mosquito screen traps them inside, and spiders soon spin a web to catch them.

8.3.3 *Double vault composting (DVC) latrines*

Originally this method was developed in Vietnam in the 1950's and has been introduced on a large scale in the rural areas of Vietnam since 1956. This model is built above ground, as shown in Figure 8.3. Urine is collected separately via a channel to increase the C to N ratio of the contents of the vault (urine has a very high N content). Vaults are used alternately. When two thirds full the contents of one vault are covered with soil and allowed to compost anaerobically for about two months. In South Africa the Division of Building Technology (DBT) of the CSIR has developed a similar system called either the Permanent Improved Pit (PIP) or the Ventilated Improved Double Pit (VIDP) Latrine. Although the vaults are sunk into the ground, the pit is ventilated and no provision is made for separate urinal removal. Carbon in the form of dry organic matter (e.g. grass) should therefore be added to achieve the required C to N ratio for good compost (approximately 25 : 1). Two years are recommended by the DBT for composting to achieve complete pathogen kill-off.

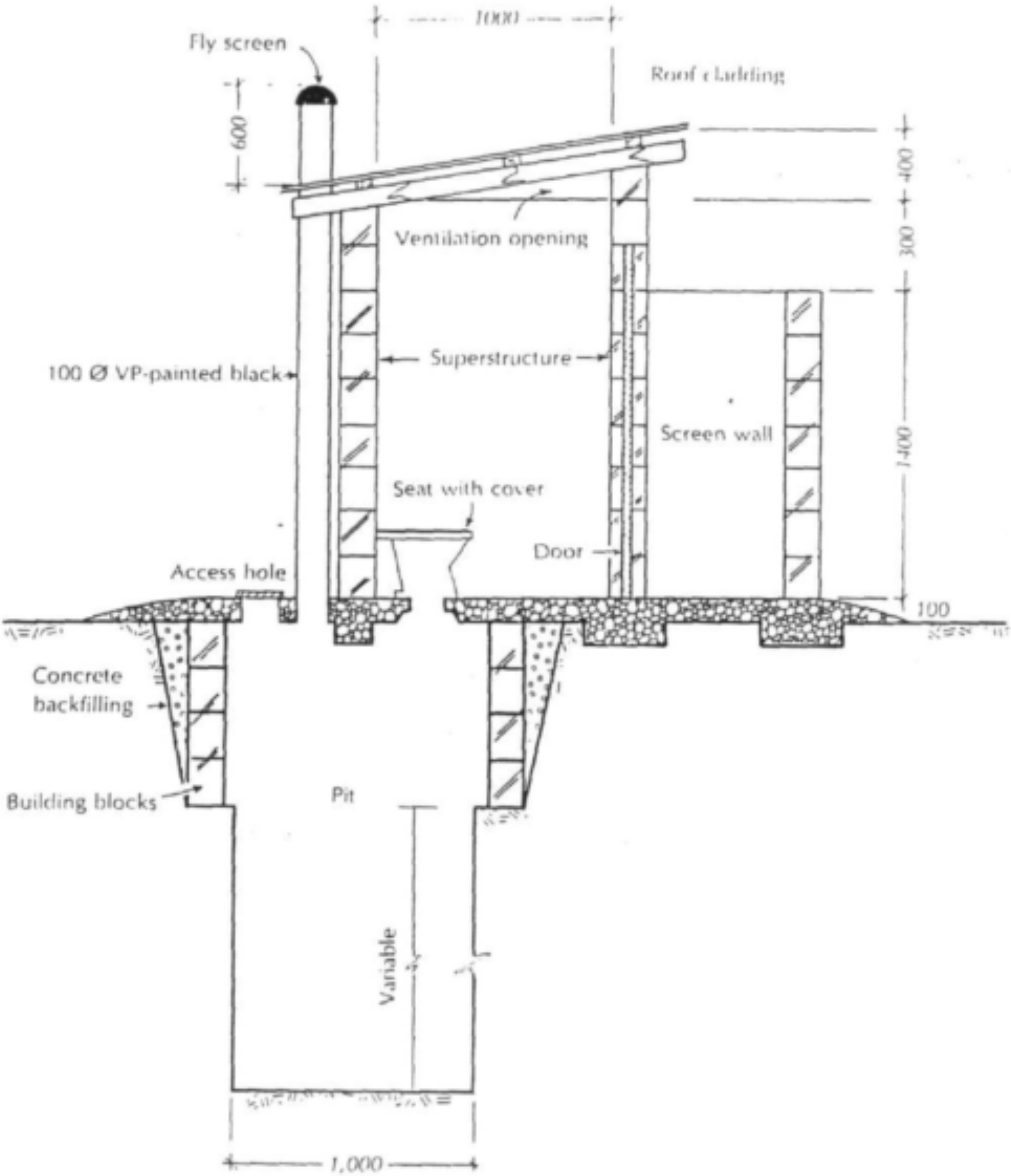


Figure 8.2 : Sectional view of ventilated improved pit latrine

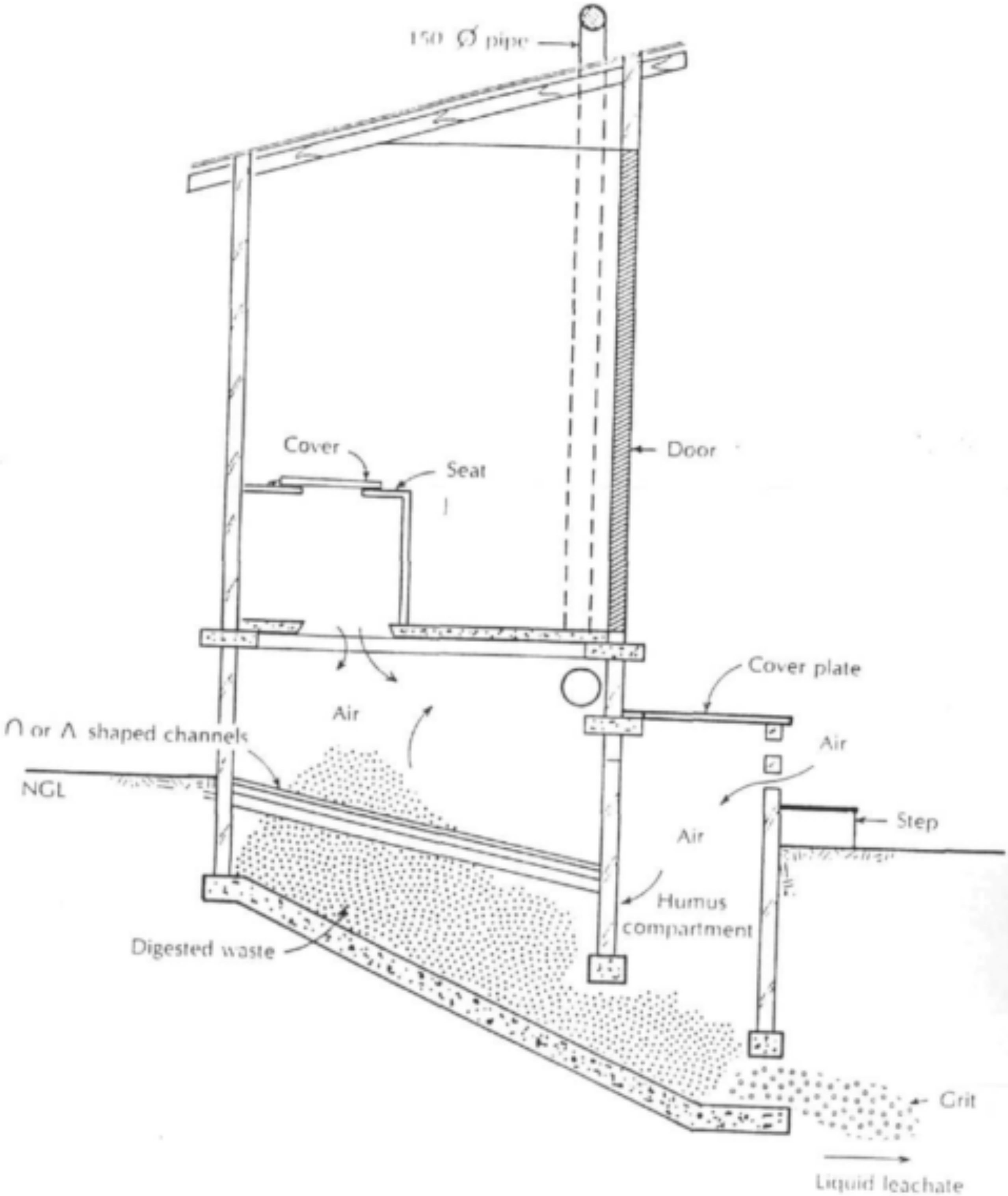


Figure 8.3 : Sectional view of composting latrine

DVC's are favoured where pits cannot be sunk 2.0 to 2.5 metres into the ground, either because of a high water table or because very hard ground prevents deep pits being dug. They are also favoured where space does not allow for the relocation of the pit latrine when it has filled. Potential for resource recovery is high, although the benefits as compared to the problems of physically emptying the pit may not be readily appreciated by the users.

In instances where the pit cannot be sunk into the ground, the holding chamber is built above ground as in the Vietnamese model. Construction details of composting and pit latrines are given by Evans (1987).

DVC latrines should not be used where:

- sufficient user care cannot be expected;
- insufficient organic waste material is available for addition to the excreta to make good compost;
- there is unwillingness to handle composted humus, and
- there is no local market or use for composted humus.

8.3.3 *Bucket and cartage system*

The system is widely employed in many African, Asian and Third World countries as well as in Australia.

The initial costs of this system are generally four or five times cheaper than a conventional water-borne system, but running costs to empty the buckets on a regular basis are high. Costs include ultimate off-site treatment of the waste, either by composting or in ponds with or without anaerobic pretreatment facilities. Rising transport costs mean such systems are likely to become even less competitive in the future.

One of the primary factors against the use of bucket systems is that fly access to the contents is not prevented, and health problems may result. In addition several taboos against the handling of excreta, as well as unsanitary conditions and health hazards to the people collecting the buckets make this system unattractive in a country like South Africa. It is also regarded as a 'second class' option by many people. The level of service provided by this sanitation system is the poor, and not really suited to rural areas.

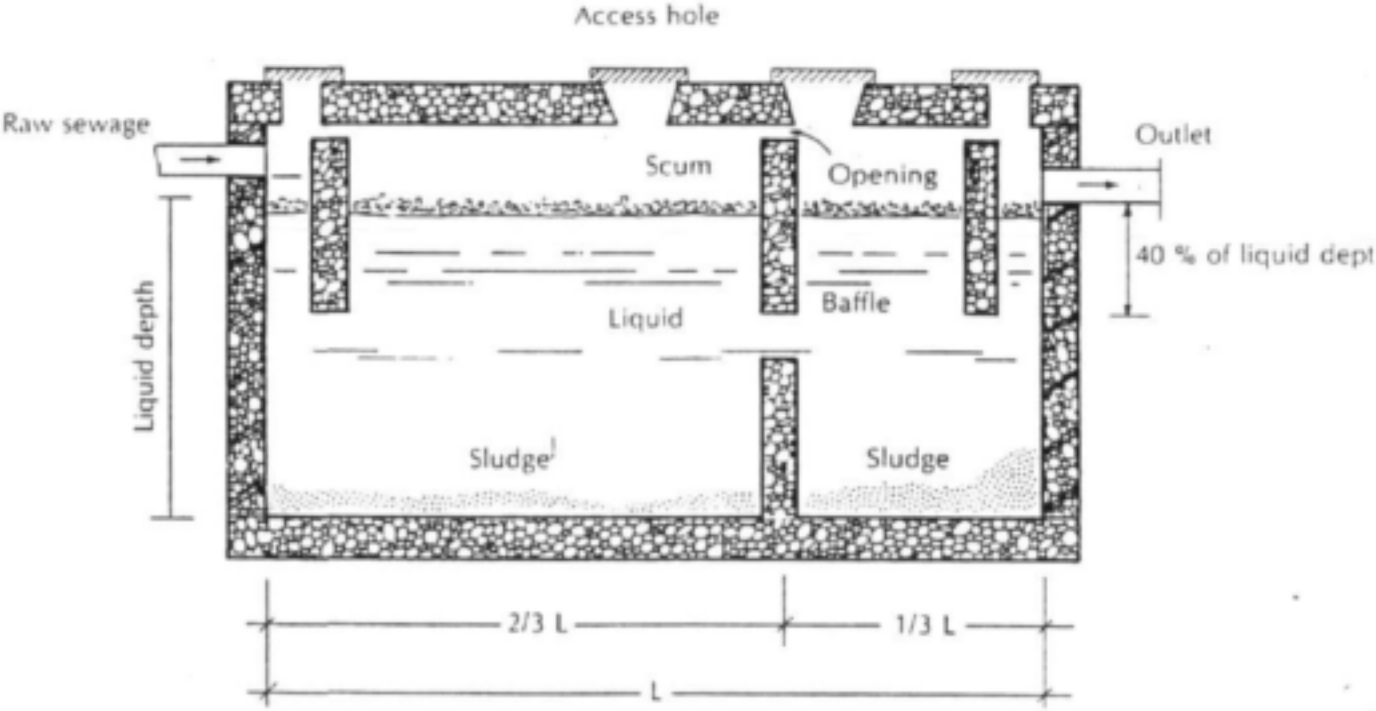
8.3.4 *Chemical toilets*

There are a number of commercially marketed chemical toilets, all of which rely on the addition of chemicals. Most of these chemicals are strong oxidizing agents, and this implies a high energy input, and consequently a high operating cost. They are unlikely, because of their reliance on purchased chemicals, to have wide appeal in most rural situations in South Africa. These toilets are consequently used only as a temporary measure in times of need. For more information refer to NBRI Information Sheet X/BOU 2-66 (Heap, 1984).

8.3.5 *Septic tanks sanitation systems*

Septic tanks form part of the sewage disposal system, and can be connected to the outlet of any water flush latrines. Septic tanks are normally designed for a relatively short water retention time of one to two days, and consequently pathogen removal is relatively poor. Septic tanks normally drain to soakaways. They can either be designed to accept sullage water from bathrooms and kitchens, or more commonly to accept only effluent from toilets. Sullage water is then conveyed directly to a soakaway after passing through a suitable grease traps. The toilet itself must be a water-flush system, either conventional, low flush or

Pour Flush. The solids remain in the septic tank, and sludge must be periodically removed at normally at three to five year intervals. The main advantage of septic tank systems is that a householder may have all the advantages of water flush sanitation systems without the need for extensive municipal sewage treatment works. Or, alternatively from the point of view of local authorities, the costs are borne exclusively by the householder and not by the authority. It is essential then that sufficient water is always available for flushing purposes. Figure 8.4 indicates the features of septic tank systems.



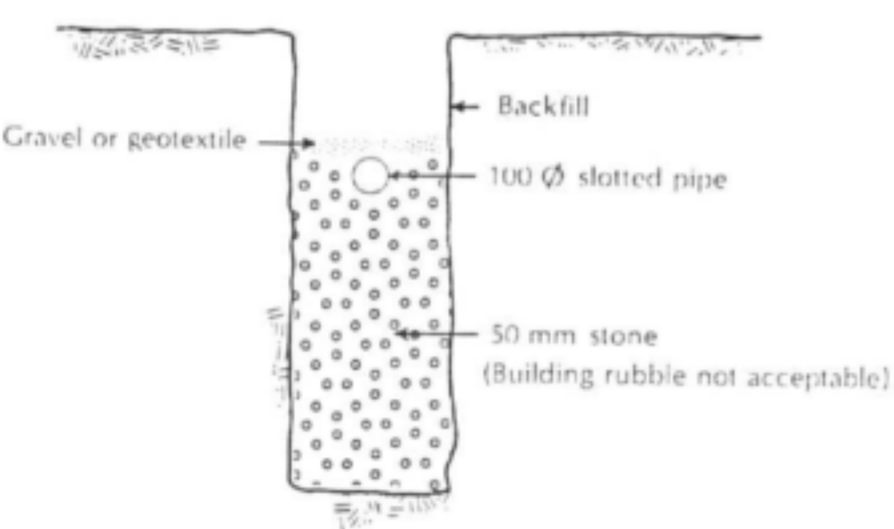


Figure 8.4 : Septic tank and soakaway

The Pour Flush toilet is basically a modification of the conventional Water Closet (WC). Small quantities of water or sullage (1 to 2 litres) are used to cleanse the toilet bowl. This is considerably less than the normal flush of about 15 litres. A properly designed Pour Flush toilet has a shallower water seal to facilitate easier flushing and is usually designed with the absence of a cistern so that water or sullage has to be manually poured into the toilet.

Septic tank systems are prone to certain technical problems which include:

. inadequate soakaway facilities, or blocking of soakaways;
. can be easily contaminated and disrupted by chemicals;
. soakaways need to be relocated periodically;
. require desludging by a central authority;
. poor pathogen removal (as with virtually all on-site systems); and
. build-up of scum in septic tank which inhibits flow and decomposition

The practice of constructing two soakaways which are then used consecutively, allowing them to drain and dry out between uses, has largely overcome the problems of blocking and relocation. Note also that the effluent of a septic tank could be fed into a reed bed system (section 8.6.3).

8.3.7 *Aqua privies*

Aqua privies are toilets located directly above or slightly offset to a watertight holding tank (figure 8.5). Because of this configuration flushing water is not required, or is required only in very small quantities to keep the tank topped up. In some systems aqua privies are connected to small-bore sewers for off-site disposal of effluent, in other systems the effluent from the aqua privy is disposed of to a soakaway on site. The function of the aqua privy is to provide settlement, stabilization and anaerobic treatment of the solid waste. This results in physical decomposition as well as a 50 % to 75 % COD (Chemical Oxygen Demand) removal. Heavy objects and non-biodegradable objects, such as stones and mealie cobs (often used for anal cleansing) settle out in the tank and hence will not result in blockages in the sewer. Because of their low water demand and the fact that a much smaller diameter sewer outlet pipe is required, cheaper sewerage systems and treatment facilities are possible.

Many aqua privies have not functioned properly in the past because of the difficulties of achieving a water tight tank. Once the water level has dropped below the water seal of the inlet pipe extremely offensive odours occur. This has lead to aqua privies being 'banned' in certain countries, for example Botswana. Aqua privies that include sullage disposal facilities will, to a large measure, overcome this difficulty. The use of plastics for manufacturing largely obviates the problem of non-water tightness.

8.3.8 *Anaerobic digesters*

This is basically a modification of the aqua privy system. Thus much that has been stated above applies to anaerobic digesters. In South Africa several commercially available systems have been developed which require minimal water for flushing. Because no water is required, the retention time of the liquid in the anaerobic digester is extended to a period of typically between 30 and 50 days, which greatly improves pathogen removal. The effluent form the anaerobic digester can be disposed of to a conventional soakaway or small-bore sewer system.

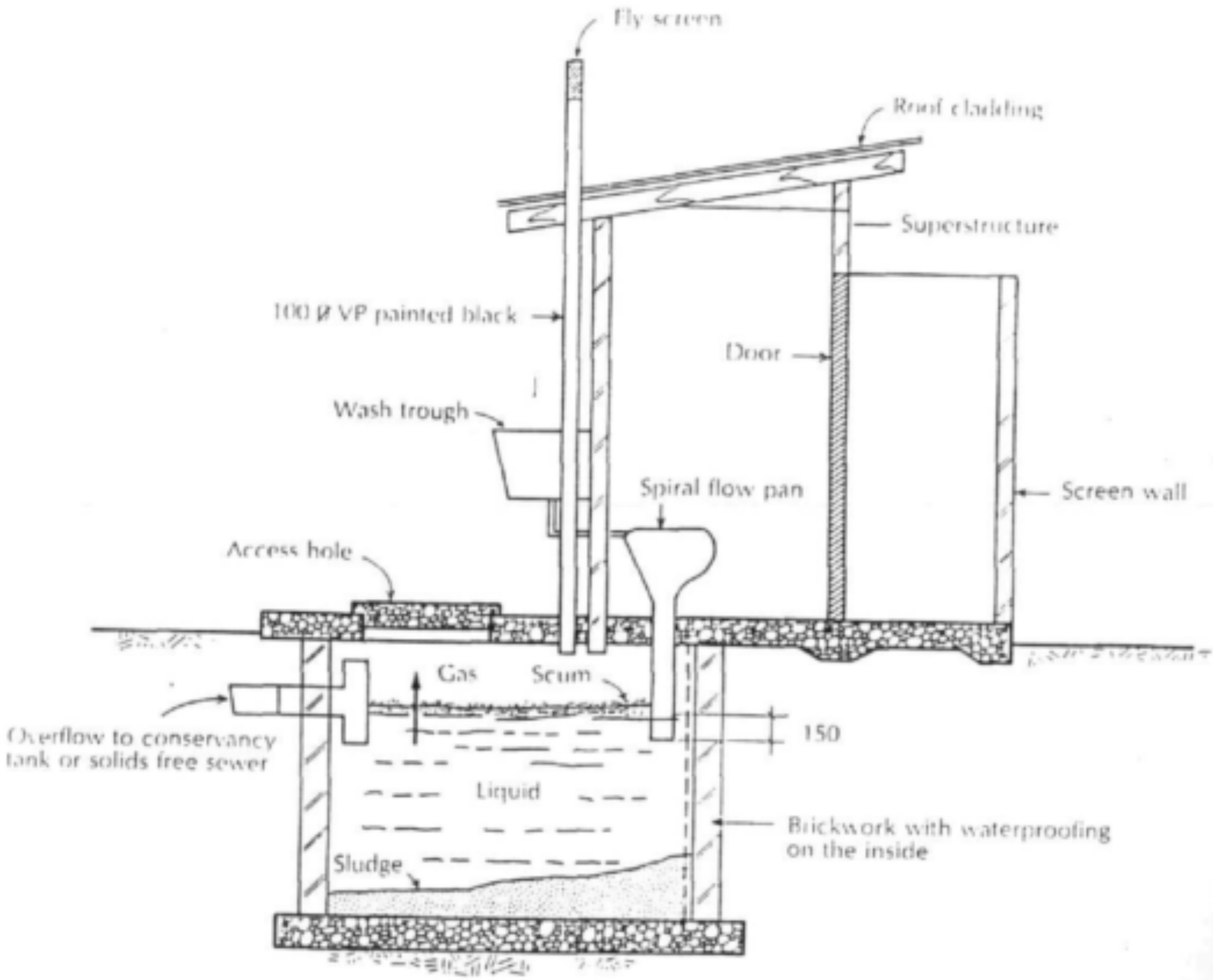


Figure 8.5 : Sectional view of aqua privy

8.3.9 *Water-borne sewerage*

This system was developed in Europe where high rainfall and unsanitary urban conditions created special problems. Water-borne sewerage systems were developed to drain contaminated surface water and later with the advent of the water closet (WC), also the disposal of sewage effluent.

The present day capital cost of water-borne sewerage, excluding operational costs, was found to be between R2 565 and R3 870 by Van Niekerk *et al* (1988). Corresponding annual household operating costs range between R345 and R779 per annum. Costs in rural areas are likely to be even higher than this due to the smaller village populations, and longer pipelengths between households. A water-borne system requires 50 to 100 litres of water per person per day to function adequately.

In addition to these high costs and excessive demands for water, there is the problem of sewer blockages which may occur, as well as the problem of the ultimate disposal of the effluent and sludge.

8.3.10 *Conservancy tank and cartage system*

This system is extensively used in the Far East as well as by several local authorities in South Africa. Normally the vault or conservancy tank is used to collect excreta plus flushing water only, with separate on-site facilities for sullage disposal. The vaults are emptied by tankers equipped with suction pumps. The tankers normally have capacities ranging from 1500 to 5000 litres. Vaults must therefore be within easy reach of tankers and a centralized disposal treatment works and vacuum tanker maintenance facilities must be provided.

Advantages over the bucket-cartage system are that less frequent emptying is necessary, and that there is less spillage of excreta. It also allows for a high degree of flexibility in matching collection facilities with demand.

8.4 Costs of sanitation technologies

In order to obtain a meaningful cost comparison between various systems it is necessary to include all the economic costs as opposed to only financial costs. Both capital and recurrent costs inherent in the system must be included. A useful figure for comparison is the Total Annual Economic Cost per household (TACH). Tables 8.2 and 8.3 give a comparison of the costs of the various sanitation technologies described above based on local and international case studies.

TABLE 8.2: SUMMARY OF CAPITAL COSTS AND TOTAL ANNUAL COST PER HOUSEHOLD (TACH) FOR SANITATION TECHNOLOGIES; 1990 COSTS (SA Rand)

TECHNOLOGY	CAPITAL COST RANGE	TACH
Low cost		
SanPlat	100-500	30-50
VIP latrine	300-1000	50-80
Composting toilet	600-1500	70-150
Medium cost		
Bucket cartage	200-750	300-500
Chemical toilet	400-1200	200-500
Septic tank	1800-2500	250-350
Aquaprivy	1500-2300	220-300
Anaerobic digesters	2000-2500	250-350
High cost		
Conservancy tank	3000-4000	550-750
Waterborne sewerage	4000-4500	700-800

Note : 1. sanitation costs include the additional cost of the superstructure;
2. waterborne sewerage, conservancy tank, and bucket systems include the cost of the bulk service (transport and treatment);
3. TACH includes capital redemption based on 0% interest over 10 years.

TABLE 8.3 : CAPITAL COSTS OF SANITATION IN KANGWANE - JANUARY 1986 (URBAN FOUNDATION)

TYPE OF SANITATION SYSTEM	TOTAL INSTALLATION COST PER SITE (R/SITE)	MONTHLY INSTALMENTS PER HOUSEHOLD (R/SITE/MONTH)
Pit latrine	262	3,26
Anaerobic digester & soakaway	800	9,95
Composting latrine	996	12,39
Aqua privy & soakaway	1 127	14,01
Septic tank	1 703	21,18
Aqua privy & sewer network	1 926	23,95
Water-borne sewerage	1 965	24,44

8.5 Sanitation upgrading

Of importance at the initial design stage of choosing an appropriate sanitation system, is the possibility of future sanitation and water supply upgrading. Should house water connections for example be planned for the future, it may be advantageous to install low flush toilets with a septic tank or digester which could later be upgraded to a conventional flush latrine. Such decisions must be based on practical considerations as well as the life cycle costing (present value economic cost) of the planned and upgraded sanitation system.

8.6 Types of disposal systems

Section 8.3 dealt with different types of sanitation systems and toilet arrangements. While covering a broad spectrum of state of the art technologies, the list is by no means intended to be comprehensive. For example electrically heated humus toilets, sand filtration toilet systems, pressurized and vacuum sewer systems, which are being investigated in California USA as part of the Rural Wastewater Disposal Alternatives Project, have not been covered.

For on-site disposal, the liquid phase of the sanitation system (whether wet or dry, anaerobic or aerobic decomposition) usually seeps into the ground either from a soakaway or directly from a pit. In the case of composting systems, the humus may be removed for gardening or agricultural purposes. The performance of soakaways and to a lesser extent pits depends to a large extent on the permeability of the subsoil and the location of the underground water table relative to the soakaway. Where adverse ground conditions occur such that seepage of the liquid wastes is likely to be inadequate, or where contamination of the groundwater is likely to pose a problem, it will become necessary to convey the effluent from the site. This may either be by means of routine collections with a vacuum tanker, or by the installation of sewer pipes. Virtually all on-site sanitation systems result in a partially treated effluent when under normal use. This results in a stabilized, settled effluent with minimal solids, or only very fine suspended matter. If the degree of pre-treatment is adequate to ensure no large particles of solids or paper, the design of the sewer system can be greatly simplified. Self-cleansing minimum scouring velocities of flow in the sewer do not need to be achieved, and blockages and hence the need for manholes at changes of direction and grade can to a large extend be eliminated. Because of the reduced need to unblock the sewer system by rodding, smaller diameter piping is possible. Hence the term "small bore sewer" (also referred to as a "solids free" sewer or "tank-sewer" system). The sewer system can be designed more along the lines of a pressurized water system. However, it is important that if sullage water is to be incorporated into the sewer system, that this either passes through the aqua privy or anaerobic digester, which will adversely affect retention times and hence the quality of the effluent, or preferably through a trap system to eliminate large particles of solids and grease. Various types of final sewage treatment systems can be used to treat the final effluent at a central location, but two systems more appropriate for rural applications are briefly outlined below.

8.6.1 *Quality of the effluent to be treated*

It must be appreciated that for any latrine where the normal volume of flushing water (10 to 15 *l* per flush) is either eliminated or substantially reduced, the concentration of COD or BOD in the effluent from the sites will in general be higher than that of conventional water-borne sewage. The addition of sullage water will have a diluting effect, and assumptions as to the expected COD values of the effluent at the treatment works will have to be made on the basis of existing information or studies from similar sites elsewhere. The "organic loading" on the treatment works will have been substantially reduced compared to a conventional water-borne system, as it is known that COD reductions of the order of 50 per cent to 75 per cent take place under anaerobic conditions within the aqua privy or anaerobic digester, depending of course upon the retention time. Provision may have to be made for the surge loading resulting from tanker emptying into the treatment works from conservancy tanks, or from digesters, septic tanks, and aqua privies.

8.6.2 *Stabilization ponds, oxidation ponds and maturation ponds*

Stabilization ponds are used primarily to reduce biochemical pollution and faecal bacterial contamination in waste waters before discharge to receiving water courses, streams and rivers. Where ponds are used to treat raw waste waters they are called oxidation ponds; where they are used to treat effluents from conventional works they are called maturation ponds.

In South Africa oxidation ponds have not found wide application. Due to the policy of water-reuse, treated effluents must satisfy stringent biochemical effluent quality standards. These standards cannot be satisfied by treatment in oxidation ponds, and therefore oxidation ponds are allowed only under a special permit issued by the Department of Water Affairs.

In contrast, maturation ponds are general throughout South Africa as a polishing stage for sewage treatment effluents even for the largest city, Johannesburg. Many rivers in South Africa are seasonal and may have no flow during the dry season other than the sewage effluents discharged into them. Maturation pond treatment appears to modify the nature of the pollution so that fungal and filamentous growths, and bacterial contamination are minimised in the receiving water final discharge water. If appropriately designed, maturation pond treatment also dispenses with the need for chlorination of effluents before discharge.

In facultative ponds (i.e. where both aerobic and anaerobic zones exist), oxygen is mainly supplied by algal photosynthesis. An equilibrium sludge mass is established after 2 to 20 years of operation. This means that the rate of sludge addition is the same as the rate of sludge degradation by aerobic and anaerobic processes, and hence no sludge accumulation occurs.

It has become more or less standard practice when designing pond systems in Southern Africa to provide some form of anaerobic pretreatment to stabilize the raw sewage. This prevents segregation of solids and liquids and reduces offensive odours and the formation of floating sludge mats. It also reduces COD and BOD. When treating effluent from on-site aqua privies, septic tanks or anaerobic digesters, this pretreatment process is already substantially complete.

Depending upon the actual strength of the effluent, dilution with surface water (e.g. storm water runoff) may be considered beneficial to the pond treatment process.

Pond systems can produce effluent of high standard, in some cases better than from conventional works. Due to low operational and maintenance requirements pond systems are often ideal for underdeveloped communities. Training of supervisors (maintenance personnel) must be considered at the design stage. The major maintenance requirement is to keep down plant growth on the banks of the pond to prevent mosquito breeding; and to fill holes and scouring caused by animals and wave action.

8.6.3 *Wetland systems*

Suitably designed and operated wetlands have considerable potential for low-cost, efficient and self maintaining wastewater treatment systems. The ability of plants (primarily marsh plants like reeds, rushes, water lilies and even certain trees) to remove nutrients, bacteria and other chemicals from wastewater depends on the existence of bacterial colonies on the roots and stems of these plants as well as in the earth or gravel media in which they grow.

The wetlands must be properly designed to ensure that the soil or growing media does not block, and that sufficient contact is allowed for adequate nutrient and bacteria removal. In general terms, between 5 and 10 square meters of vegetation per person are required. Depth of beds is optimally between 0,6 and 1,0 m.

Wetlands, however, are not suitable for the treatment of raw, unsettled sewage. They should be viewed more as a polishing step to primary treatment processes like septic tank treatment or trickling filters.

Aspects of wetland systems have been pioneered in South Africa, and they are being increasingly used in small towns and other applications.

The cultivation of woodlots, in particular Eucalyptus gum trees, which are relatively big consumers of water, is one avenue that should be considered for the safe disposal of sewerage effluent. First because there is little danger of food contamination via the faecal-oral route, and second because the provision of wood meets the domestic energy requirements of rural and peri-urban communities. The use of maturation ponds and high rate algae ponds which receive low strength waste water for fish farming has received a good deal of attention and research in Asia, Israel and South Africa. Such ponds are being successfully used for fish production in many areas. The flesh of the fish remains clean, the only risk of contamination coming from the gut and from handling. A chlorine bath dip after cleaning ensures a safe food product.

The different types of disposal systems described briefly above are not intended to be a comprehensive list of disposal options but to point out that a more integrated approach to waste water disposal can lead to resource recovery as an added benefit.

8.7 Choice of technology

In rural areas at present simple unimproved pit latrines and a few ventilated improved pit latrines (VIPs) make up by far the majority of systems in use. VIP and SanPlat technologies are suitable for virtually all applications except where ground conditions are difficult for any type of on-site system. Even in these situations though, these technologies can still be used to a certain extent. VIP and SanPlat technologies are economical and result in a major improvement in on-site sanitation for rural areas.

It has been stated (Jackson 1991) that there are no inherent merits in the proprietary water dependent on-site sanitation systems when compared with a VIP latrine. Householders would only be willing to pay more if they could obtain a reliable in-house toilet such as a low volume flush toilet with a fail-safe water seal.

8.7.1 *End user preferences*

The following elements should be investigated before the detailed design stage:

Squat plates or pedestals

Before specifying the type of toilet pedestal and seat, it is essential to determine the preference of the target group to the type of toilet preferred. For example, most Asian communities prefer the squat plate arrangement, while amongst the Zulu community there is a strong preference for pedestals. It is not sufficient to assume that precedents already set in a particular area necessarily reflect preferences of the users. The materials used in the manufacture of the pedestal or squat plates must be chosen for aesthetic reasons combined with ease of keeping clean and robustness. Rough, porous surfaces are unacceptable.

Shape of toilet bowl

In the specification of toilet pedestals for pit latrines there are two conflicting trains of thought. The one is that the inside of the toilet should taper outwards from the toilet seat at the top to the opening in the slabs at the bottom. The reason being to minimize fouling of the inside of the toilet bowl. The other opinion is that the toilet bowl should taper inwards from the seat at the top to the opening at the bottom to obviate the psychological fear that small children have of falling into the pit. In the sanitation programme in Botswana (Van Nostrant, J and Wilson, J G, 1983) a prefabricated glass fibre reinforced plastic (GRP) pedestal insert was manufactured with a seat opening of 310 mm tapering to a 200 mm diameter opening which projected through the cover slab. This was well received by pit latrine owners. It should be accepted that as it is a primary objective of any sanitation system to encourage everyone to use toilets, catering to the needs or preferences of any subsection of the community, whether it is young children or the aged, is important. Toilet bowls should be designed and manufactured in such a way as to be aesthetically pleasing, easy to keep clean, prevent splashing or blocking and be acceptable to all classes of users.

Toilet seat and covers

It is desirable to have seats and covers installed on all toilet pedestals. When covers are installed on VIP latrines, it is important to ensure that spacers separate the seat cover from the toilet seat to allow adequate ventilation of the latrine. A minimum gap of 15 mm around the circumference of the opening should be maintained. For SanPlat systems on the other hand, a tight fitting lid is required. Toilet seats with smaller openings should be available for installation by parents of small children to encourage them to use toilets. The toilet seats and covers should be designed to be replaceable and manufactured from aesthetically pleasing, smooth and robust materials. Child seats should be optional extras purchased by the user.

Cleansing materials

Because it is considered a taboo subject, the type of materials used for anal cleansing have often not been discussed or even considered when designing sanitation systems. This is a grave error and has lead to expensive malfunctions and failures of sewerage systems.

It is generally accepted by people who have had experience of septic tank systems that certain materials e.g. cigarette butts, disinfectants, newspaper and sanitary towels, should not be thrown into the system. However, in many townships and rural areas throughout Africa, maize cobs, leaves, newspaper and stones are common materials used for anal cleansing. These can and do cause blockages in the sewers. The aqua privy system with a small bore sewer was pioneered by Prof G Marais (1978) in Zambia as a response to this problem. In India the normal method of anal cleansing is to use a jug of water and the left hand. This practice has lead to the development of the Pour Flush toilet in Asia. Pour Flush toilets would not necessarily be appropriate or even workable in an African situation where bulky or heavy materials are used for anal cleansing.

The use of strong disinfectants, non-biodegradeable cleaning materials, and the disposal of common household wastes in the latrines should be discouraged through an effective educational programme.

Superstructure

Before the designer specifies the type of superstructure it will be necessary to ascertain whether prefabricated superstructures are available as proprietary items and at what price. Superstructures could be made in a wide variety of ways and use of indigenous skills and materials should be investigated and encouraged. The size of the toilet may be of importance to the householder for a number of reasons.

Doors

If ventilated latrines are to be fitted with doors and locks, then the door must allow adequate through ventilation so as not to materially effect the principal of the design. The orientation of the door of an outside latrine with respect to the house should be decided by the home owner.

Addition of materials and chemicals to toilets

Living and eating habits have an effect on the sanitation system. For example large quantities of pot scrapings of maize meal when added to the sewers can substantially increase the COD of the raw sewage at the treatment works and contribute to blocking of the sewers. Grease adversely effects scum formation in septic tanks. In some areas, due to the privacy afforded by the toilet superstructure, the toilets are used as bathrooms. The addition of large quantities of sullage water to pit latrines or DVC latrines will nullify their intended method of operation. The addition of household refuse to pit latrines adversely effects their useful life. A wide range of chemicals, emulsifiers and disinfectants are prescribed for addition to toilets of all descriptions. Until the chemical and bacteriological effects of these additives are understood, their use should be avoided. Most microbiological systems are sensitive to these chemicals.

Posters for the correct use of toilets

Regardless of the sanitation system to be employed, users education is essential. The erecting of permanent, easily understood, pictorial posters in newly installed toilets has been found to be one of the most effective methods of gaining user cooperation. This should be backed up by a programme of public health education.

SECTION 9 : MANAGEMENT OF WATER SUPPLY AND SANITATION PROJECTS

9.0 Overall objectives

The provision of a reliable, safe and accessible supply of domestic water for a rural community who may be poor, are without a conventional statutory authority and have relatively informal arrangements with regard to property boundaries and public space, poses a different dimension to project management for the engineer or planner. Alternative technical options have to be considered and compared not only for practical feasibility and for evaluation on a common economic basis, but also for long term sustainability within the structures possible for that community. The aspects of sustainability related to operation and maintenance, local management and financial control have been identified during the course of the International Drinking Water Supply and Sanitation Decade to be the greatest challenge to developers (Cairncross *et al*, 1980 and World Bank, 1980). Lessons learnt during the past decade in this respect have been as follows:

- Reliable operation and maintenance must be undertaken as a collective responsibility for the system to function satisfactorily. Central to reliable operation and maintenance is the question of sufficient revenue from water users to cover recurrent costs.

- Finance is usually limited, and therefore the lowest cost solution compatible with continuous reliable operation must be found. Self-help projects are advocated as ways of extending limited funds. Part of the project finance may be in the form of a loan, but then the system must be designed in such a way that loan repayments can be met by the community.

- Control of water distribution is necessary if water wastage is to be avoided and consistent supplies are to be assured. Control is also necessary to prevent conflicts between water users from arising.

A further issue is the question of expanding, upgrading, or carrying out major repairs to the water supply system in the future. These needs requires generation of finances to create a reserve fund within the system. This is important because outside loans or grants are unlikely to become available at the right time in the future to effect the needed or desired changes. Understanding of the importance of such a reserve fund is usually difficult for a community who often require cash just to secure the basic needs. Accountability is a factor which plays a decisive role both in terms of financial planning and in terms of how payment is extracted equitably from the beneficiaries of a water supply scheme.

For a water supply system to be implemented and to continue to function on a reliable basis, an administrative structure has to be established. Water and Sanitation Committees at the community level must be recognized as an essential prerequisite for successful projects of this nature. In the Southern African context, it has been found that District Development Committees should be established comprising *ex-officio* government personnel as well as representatives from the tribal authorities and

elected representatives of the communities to co-ordinate activities between the communities and the Government at the magisterial district level. It is important for non-government organisations not to neglect the need to include government departments in their development efforts, particularly as these government departments have the responsibility for the long term growth and well being of the communities. Equally it is important for government department officials to recognise the valuable role that non-government organisations can play in the development of rural communities. Governments must establish a coordinated approach to development backed by sound policies and institutional frameworks to support such policies.

Water supply projects may be the initiation of or may build on a much wider process of social development. This process, which often has considerable bearing on the outcome or the success of a particular water supply project, is perhaps the most challenging and rewarding aspect of each project. Engineers and planners who are used to working in a more conventional single discipline approach to water supply, must recognise the challenge of this approach, and take steps to actively nurture the development process through involvement in such projects.

In conclusion it can be stated, with the benefit of hind sight that the "top-down" approach and implementation of projects designed by 'outside experts' has lead to an unacceptably high proportion of improved water supply projects failing. These failures can be attributed amongst other things to inappropriateness of the technologies employed. For example there are cases where aid programmes are tied to procurement programmes for equipment from the donor country. The equipment is often not maintainable by the communities for whom it is intended and logistic problems arise in the supply of imported spare parts. Another equally important cause of failure is the lack of indigenous institution building and manpower training as part of the water supply development programme. The World Bank (1976) have identified the most important problems in the water supply and sanitation sector. They have found a lack of government water and sanitation policies in developing countries, with consequent undefined and overlapping responsibility between numerous agencies.

There is generally institutional weakness at all levels as well as a lack of trained manpower. Low income levels of communities together with a failure to collect adequate charges from water users leads to the economic collapse of many water supply projects. Public health education is invariably lacking. Failure in water supply systems are frequently due to poor operation and maintenance procedures or lack of spare parts. There are also difficulties in communication between communities and support agencies.

Clearly from the above technological problems are less important than the social and organizational constraints. Institutional weakness is singled out as the most important problem. Consequently any objectives of the water supply project must address these constraints. Broadly speaking community-related social objectives may be achieved by having the community participate in planning, constructing, operating and maintaining the water supply system. Particular attention should be given to institution building within the community aimed at increasing public awareness, self reliance and efficient management. This should be supported by sound government policy, and initiatives by Government to actively support and coordinate development activities.

9.1 Community participation

9.1.1 *Definition and general discussion*

The term community participation simply means the involvement of people in projects that are aimed at improving their own lifestyles. Due to the diversity of project objectives and activities, this term has been understood and interpreted at different angles. However, a working definition seem to emerge among some international development organizations. According to this definition (White, 1981:2), participation has three dimensions as follows:

- involvement of all those affected in decision making about what should be done and how;

- mass contribution to the development effort, that is, to the implementation of the decisions;

- and sharing in the benefits of the programmes.

Taken together, these three dimensions define community participation as an active involvement of the local population in defining their problems and making decisions concerning the projects, their implementation and evaluation thereof. Community participation is further defined as an organized involvement of a community in a development effort with all major population groups being represented as opposed to a person-to-person relationship (IRC, 1988:1).

It is further stated (White, 1981:3) that the involvement of the population in physical work can hardly be considered community participation unless there is at least some degree of sharing of decisions with the community. Therefore, if an outside agency remains in total control of the process and merely calls upon beneficiaries to give their labour directly, one cannot speak of community participation though there is an element of self-help labour. Unfortunately, the latter situation occurs in some South African national states. For example, the government merely employs labour from the community to dig trenches for pipelines and then claim popular participation when in actual fact the community has had no input in the decision-making process. Proponents of community participation condemn this practice because it is undialogical and dehumanizing. Genuine involvement of a community at all stages of development is, therefore, essential for the success and sustainability of projects.

Political commitment on the side of government authorities as well as an establishment of a closer relationship with the intended beneficiaries are prerequisites for successful community development projects. In the development process there are three participants identified by Berger (Coetzee, 1986:122), : the policy makers, the scientists or theorists and the 'ordinary' masses. The first two participants have their own different objectives and policies, and they may tend to empower as well as ignore the needs of the masses. This is due to the conceptional gap among all parties which needs to be bridged. In planning for water supply projects, the policy makers and the scientists (project leaders) should take cognizance of the fact that the 'ordinary' masses know best the situation and complexities of their everyday life situation.

Therefore, a dialogue should be encouraged between the project planners and user community in order to negotiate a strategy to implement a water supply project. The sociologists or other social scientists are ideal catalysts in this regard. Romm and Hölscher (Coetzee, 1986:1217) state that a sociologist must account for and come to terms with other subjects' meaningful vision of the world. Ideally this is the form of a dialogue in which a sociologist as a subject and a layman as a subject, negotiate an intersubjective reality. The sociologist should be able to encourage all parties to open up to the views of others, rather than ignoring or eliminating them. A possible way of achieving this in practice is to conduct a scenario planning exercise at an early stage of the project in which the relevant individuals from the community, Government, tribal authority and development agency work through a planning programme. Possible problems and consensus decisions on how to approach them should be dealt with.

Elmendorf and Buckles (1980:1) state that a dialogue approach requires a significant amount of time-consuming consultation with the community. Therefore, the optimum use of scarce technical resources will require that water supply and sanitation agencies co-ordinate their activities with other national entities such as health clinics, which already have trained or trainable promoters at the village level.

Some organizations may not be able to afford the services of a social scientist, and therefore, may train a facilitator or promoter at village level to provide a link between the planners and community. Facilitators may be selected from professional elites such as teachers, nurses and agricultural officers. These facilitators must at least be trained in social science techniques in order to provide planners with information about community attitudes, perceptions, preferences and doubts (Elmendorf and Buckles, 1980:1). The facilitator must also be able to understand and communicate the technical and economic aspects of available alternative technologies to permit communities to make appropriate choices.

It should be stated, however, that the task of convincing a community to change their habits is difficult. Another consideration is the fact that the organizational capabilities of communities vary, thus in some communities participation could be achieved much easier than in others.

9.1.2 *Who should participate?*

The answer is simply that all people in the community irrespective of their political affiliation or social status should participate. Therefore, a reaching out strategy needs to be developed in order to motivate and encourage even the poorest of the poor to participate. It is important that the views of disadvantaged groups should be taken into account in order to avoid domination of elites in water organizations.

Unfortunately, participation in most projects tend to come from the economically advantaged and educated groups rather than the poor. Worst of all is the fact that in most African cultures, women are usually deprived of the opportunity to voice their opinions in public. Thus, despite the fact that fetching water is basically the women's task, decisions regarding the choice and siting of water schemes are done by men. In some cases, the low-income groups may be reluctant to participate for fear of financial commitment. Therefore, facilitators should guard against these tendencies. Although it may be difficult to rapidly change age old customs of human behaviour, efforts should be made that all groups in the community be involved. This could be done through consultations with opinion leaders, women's groups, burial/savings clubs, church groups and so on.

9.1.3 *Procedure for community participation*

It should be borne in mind that there is no one model for community participation which is applicable to all situations in the developing world. Even in the same country, participation may take different forms because the socio-economic situations of the people also differ. Therefore, aspects highlighted in here could be modified to suit particular circumstances. Most of the information presented here was derived from worldwide literature, as well as from local experience gained and evaluation studies conducted in the field of water supply and sanitation projects in South Africa.

Phase I : Problem identification and selection of communities to be first served

In this phase, planners need to identify communities with problems related to water supply and sanitation in their area of operation. Therefore, the phase relies primarily on existing information and requires limited field work. In view of the fact that not all communities can be served simultaneously, there is a need for prioritization. Criteria that might be applied in prioritizing communities include:

- Communities who take some initiative to upgrade their own services;
- Communities where water sources are polluted and unprotected such as springs, ponds and rivers where animals drink and wade;
- Communities where facilities have broken down, or are functioning poorly;
- Poor/disadvantaged communities who live below the poverty level (worse socio-economic problems);
- Communities where water is scarce with an availability of less than 20 litres per person, per day;
- Communities with history of repeated and/or frequent incidence of water and excreta related diseases;
- Communities where sanitary facilities are limited (Chandler, 1986:2).

Village profile should also include more information on indicators of community organization, previous experience in participation projects, local leadership and factions within the community. In addition (IRC, 1988:18) suggest that the following critical questions be considered in this pre-planning stage:

- Is there a legal framework which permits community participation?
- What has been the background of community participation in the country and particularly in the region of the project?
- What is the likely level of 'social readiness' for changes envisaged and for the desired level of community support?
- What governmental and non-governmental organizations are concerned with water supply and sanitation, community involvement and the involvement of women?
- Who can assist in preliminary designs of community participation?
- What is the variation in the country or region in terms of cultural traditions, languages, felt need for improved water supply and sanitation?

- **Will technological solutions influence levels of acceptance and community participation?**
- What is the political climate which supports or constrains community participation?
- How can existing social or developmental structures be best used in new projects?

This information would be used to assess the level of readiness of a community to participate in water supply and sanitation project. In this way, planners will be in a position to rank communities into high and low-priority groups, with the former selected for initial action.

Phase II : Participatory planning

Having drawn up a priority list of communities to be served first, the next step which must be treated with circumspect is the establishment of contact with the community. A more humane approach is imperative in order to foster long term rapport with the community. This phase involves consideration of a number of key issues, each discussed individually as follows:

■ Negotiation with local authority and focus group's/opinion leaders.

At the outset, local authority and other community leaders must be contacted to assess the need and priority given to water supply and sanitation in relation to other priority needs. The planners should explain the rationale for choosing that particular community, objectives of the project as well as the possible spin-offs the project will bring about. A dialogue should be encouraged at all stages of negotiation. If the local authority is in favour of the project, it becomes imperative that planners conduct further discussions with focus groups in the community. It is important that planners collaborate with all levels of community to ensure that all parties are fully aware of the project. Focus groups of not more than six people may include:

- Burial societies/savings groups
- Women's leagues
- Professional and business elites
- Youth groups
- Political groups

Throughout the discussions, planners must ensure that an open dialogue is encouraged, thus, encourage the introverts or reticent members of the group to feel no constraints against expressing their views. Unless more specific information is needed, the discussions must be kept as informal as possible. Informal discussions are valuable in eliciting a variety of information as well as creating a relaxed atmosphere conducive enough for group members to supply information. A checklist of information that needs to be gathered is essential. However, questions could be memorized in order to maintain the smooth flow of discussions. Advantages of group interviews are twofold thus:

- Community members who for whatever reason are unable to express their views publicly are able to do so in small group discussions. For example, in most African cultures, it is still a taboo for women to speak in front of men, therefore, group interviews gives them a chance to voice their opinions. Moreover, women are more knowledgeable about water use in the family unit.

- **In group interviews there is usually a self-correcting mechanism. For example, if one person over or underestimates the number of perennial springs in the village, his peers will give a more realistic view of the situation.**

If the proposed project is acceptable to all community representatives, the next step is to arrange for a formal meeting.

■ Community meeting

A formal meeting is vital to ensure that all members of the community know about the project. At this stage, the project is formally introduced in order to gather additional information and ideas on the proposed project. The planners should once again explain the rationale of undertaking the project in a particular settlement. Possible spin-offs of the project should be made explicit. Once again, efforts should be made to encourage a dialogue. Furthermore, planners should try to establish the indigenous knowledge available in community. It is further essential that planners also suggest possible water supply alternatives for the area.

During the ensuing dialogue, both the planners and community members must formulate alternatives based upon suggestions made. The emphasis should, however, be on selecting alternatives that reflect all of the diverse ideas of community members. No effort is made at this point to eliminate or criticize alternatives or to show why they may not be feasible (Chandler, 1986:4).

The planners should at this stage also negotiate to undertake a socio-technical feasibility study to ensure local details within the community. An agreement to report back the results of the study in another meeting should also be made.

■ Feasibility study

Many development agencies dismiss the necessity of undertaking both the feasibility and project evaluation studies because they are neither time-efficient nor cost-effective. Indeed, in the past social scientists allowed themselves endless months, if not years to generate detailed information that became irrelevant, inaccurate, late and unusable for project planning. However, in the past few years sociological and anthropological researchers involved in development work have significantly refined their approaches and techniques (Cernea, 1985:399). For example, Chambers' short-cut methods of gathering social data such as group interviews (discussed previously), direct observation, key informants, informal interviews, learning from the rural people etc. could be utilized to ensure that the information is timely, accurate and usable.

It is essential to undertake a survey in order to assess aspects that may influence the potential success of the projects. Although most of the social data could be gathered from group discussions, there is some personal information which could only be gathered at household level. Such data include water use and practices, income level, demographic and epidemiological factors to support available statistics. However, a combination of group discussions and individual interview to gather this data could be used.

A representative sample of a few households could be interviewed to ensure that the following details are established (Whyte, 1986:18-19).

- morbidity and mortality rate, especially among infants
- practices in relation to water, sanitation and health
- number of children born per women
- household as an economic unit
- migratory patterns (especially of men)
- source and level of income
- seasonal variation of income
- attitudes to paying for water
- familiarity with credit facilities
- possibility of paying in ways other than in cash, for example assistance in maintenance
- levels of education for different sex and age groups
- attitudes and willingness to work together as a group.

Technical considerations may include:

- available/potential water sources
- distance
- per capita usage
- assess technical feasibility of various alternatives
- work out cost/benefit of alternative solutions
- local technical knowledge, skills and capabilities.

The local people need to be involved in the survey, particularly on social aspects because they are more knowledgeable about habits and resources available in the village. This involvement can come about through the selection of facilitators, particularly from the professional elites such as teachers, nurses, health workers, agricultural extension officers, social workers and health inspectors. In most cases, these facilitators are experienced in working with the community, share its culture and know who the leaders are. However, minimum training on social science techniques of gathering data, observation of water use habits, listening and interview is imperative. Besides the knowledge of people and resources, another advantage of involving local facilitators is to eliminate suspicion especially if outsiders undertake surveys on their own. Of importance is the fact that facilitators could be utilized throughout the project cycle. After completion of the survey, the results should be analysed and interpreted.

■ Second community meeting

This is the stage at which the results of feasibility study as well as the analysis of technical alternatives suggested by the community are reported back for discussion. The planners should be in a position to clearly explain the implications of software/hardware mix to the community and should encourage a dialogue. The community should be guided into choosing technologies that are soundly acceptable and affordable. Due to their cost-effectiveness, the labour-intensive rather than the capital-intensive method of implementation should be encouraged.

In addition, the technology should be simple, fulfil the user's needs and expectations and be easily maintained by the beneficiaries.

Having chosen an appropriate technology for the village, a water committee should be elected. Efforts should be made that this committee is as representative as possible. The role and duties of the committee should be clearly defined in order to limit misunderstandings. A checklist of aspects that need consideration at this stage include:

- timing for construction
- choice of level of service and siting of schemes
- allocation of tasks
- provision of paid or free labour
- supervision of labour
- provision of local materials
- provision of food or drinks for workers
- applying penalties/allocating rewards
- management of community funds
- keeping records of labour as well as financial contributions
- provide housing for technical team
- provide storage for tools
- eligible people to be trained in operation and maintenance of schemes
- choice of site (Whyte, 1986:43)

In fact, this will be the duty of the water committee who, after planning, would report back to the community. Having discussed all pertinent issues, the meeting may adjourn with an understanding that the community will be kept informed about new developments.

A water committee should be provided with proper guidance in:

- book keeping
- financial management/keeping accounts
- organizing community meetings
- recording minutes of meetings
- paying operators
- dealing with user's complaints
- organizing collection of payments etc.

A training session is imperative in order to eliminate uncertainties regarding the role of a water committee. Both parties must be prepared to learn from each other about the most appropriate method of running the project. Efforts should be made to get all people involved to participate.

Phase III : Implementation

The labour-intensive approach requires the most efficient planning. Therefore, the role of all parties should be clearly defined prior to implementation. It is important that the community have a major input into scheduling implementation of the project according to seasonal activities such as cultivating the land, planting and harvesting. The community should be encouraged to decide on the most appropriate time to start with the construction.

The planner's role is to design and supervise construction, while the water committee should have inputs in organizing cash collection, labour to construct and local materials where applicable. If the materials have to be imported, the local government departments could assist with transportation. The on-the-job training of personnel for operation and maintenance should begin as soon as possible (Chandler, 1986:5). In most cases, some committee members assist in the construction process itself.

Committee members may also keep record of households that contributed labour so they can enforce sanctions for non-participants. Throughout the project cycle, health education should be emphasized. Householders should be encouraged to build pit latrines in order to prevent recontamination of the water sources. At completion of this phase, a ceremony may be organized to officially inaugurate and hand over the facility to the community. Handing over the facility to the community is not the end of the road. There is a need for establishment of links with a programme support network to ensure back-up assistance, where necessary. For example, links could be establish with relevant institutions such as clinics, agricultural offices and so on. Chandler (1986:6) states that the role of the programme support network is to provide back-up to communities and their institutions on request. Such support will require the provision of additional funds and manpower. Back-up support may be provided in conjunction with other programmes, such as hygiene education, which may be supported through primary health care units. The programme support network may also include a unit to evaluate projects.

Phase IV : Project operation and maintenance

Village Level Operation and Maintenance (VLOM) is the most efficient method of ensuring self-sustainable projects. Therefore it is imperative that local people be trained on simple maintenance procedures of the scheme. Criteria for selecting eligible operators include:

- level of education
- knowledge of official language(s)
- knowledge of local language(s)
- previous related experience or skills
- age
- sex (depending on complexity of the scheme, otherwise unisex is recommended)
- fairly sure to stay in the area (old age pensioners could be good candidates)
- good local standing
- local artisan

The duties of the local operator of public facilities may include:

- undertaking routine maintenance
- doing simple repairs
- reporting immediately in case of breakdown
- reporting periodically to water committee
- showing facilities to official visitors
- helping in health education
- advising people on correct use of facilities
- handling disputes, controlling queues etc. at standposts (Whyte, 1986:45)

The water committee on the other hand is responsible for supervision and remuneration of operators while the community is responsible for payment/contribution of an agreed upon nominal fee for maintenance of the scheme.

Phase V : Evaluation

Evaluation of water supply and sanitation projects has recently gained momentum throughout the Third World countries. For decades, international donor agencies had invested a lot of capital to improve the quality of life in the developing world but did not necessarily receive any feedback. The need to undertake evaluations was prompted by repeated project failures throughout the developing countries. Evaluation is an integral part of community water supply and sanitation projects. In every development effort there are a set of objectives to be accomplished. Therefore, project evaluation aims at assessing whether the intended benefits have been achieved or not. According to Swanepoel (1989:71) there are often misconceptions among community workers who see evaluation as a luxury to be undertaken only if there is time and if the community worker has research experience. He further states that this is far from the truth and states that evaluation is an absolute necessity and it is not necessary to have research experience in order to undertake evaluation. But why should projects be evaluated?

The rationale for water supply and sanitation project evaluation include:

- an assessment of the appropriateness of the technology used as well as the performance of water supply and sanitation schemes
- a comparison of people's hygiene practices after completion of the project with habits observed prior to implementation
- an assessment of the attitudes of people towards their water supply and sanitation schemes
- the determination of the impact of community participation and involvement in development projects
- the provision of feedback to planners regarding their original planning assumptions in order to modify project designs to avoid previous mistakes and repeat successes in the future.

Evaluation can be done in two phases thus:

- While the project is in progress throughout its lifespan. This can be best described as a monitoring phase. At this stage, planners are able to see if objectives are being met, problems encountered and changes that are effected.
- At completion of the project. At this stage, the overall impacts of the project is being evaluated to establish if it has accomplished its objectives.

Once again, the community should be involved in evaluation of the project. As with the base-line survey, informal group discussions, interviews etc. will result in valuable data and encourage community participation.

9.1.4 *Conclusion of community participation aspects*

Ever since the inception of the International Drinking Water Supply and Sanitation Decade (1981-1990), community participation in development projects has become a catchword among development agencies worldwide. It is a recognized fact nowadays that projects that take human factors into consideration are more likely to be successful than those that do not. In the past, planners ignored social expertise in water supply and sanitation projects, as a result many project failures were recorded. Therefore, it is important for planners to take people's aspirations, needs and perceptions into consideration when planning for community water supply and sanitation projects.

9.2 Planning and construction

9.2.1 *Planning*

In planning the water supply, cognisance must be taken from the start, of the operation and maintenance of the system. If it is recognized that these responsibilities cannot be assumed by any existing institution and will have to rest with the communities themselves. Consultation with the Water and Sanitation Committees must be sought in establishing the location of public standpipes and other facilities. Siting of standpipes is an important decision to any community. It is desirable to leave the decisions to a consensus of opinion, or in the event of a dispute, to mediation by a technically competent person. The location of pipeline routes similarly must be discussed with members of the Committees in the field. The development agency has the responsibility to site pipelines to avoid causing disputes or disagreements and must restrict the number and length of pipelines to fit the budget.

9.2.2 *Construction*

Construction of such water supply systems can and should be undertaken as far as possible by the local community. Such involvement has definite social benefits. A sense of ownership of the water supply is instilled in the community thereby enhancing the security of the system. Temporary employment is created and money flows into the community. Skills are learned and ideally local maintenance personnel can be trained. In some areas due to the nature of the topography, mechanical trenching may be impractical. Knowledge of the location of buried piping is retained by labourers. This can be invaluable in the future locating of the buried pipe.

In the case of pipelines, semi-rigid polyvinyl chloride (PVC) small diameter piping has proved to be ideal material for underground water pipes. Being flexible it conforms to the undulating, non-linear pipe routes which are usually located along existing roads and often traverses rocky topography. Thermal expansion and contraction are automatically accommodated in this type of construction. The level of skill required to correctly lay and backfill this type of pipe can be quickly learned by relatively unskilled personnel.

Maintenance of PVC piping is considered easier than with asbestos-cement or steel piping. High density polyethylene (HDPE) piping has not proved as reliable as PVC piping, and with the passage of time the frequency of burst occurring with HDPE may increase, while the PVC piping will withstand the normal operating conditions without trouble. Above ground piping that is subject to mechanical damage and ultra-violet radiation should be galvanised iron or mild steel. The metering of all public standpipes and connections, as well as bulk meters, is an essential feature for proper control, as is the provision of lockable taps and valves. The installation and threading of steel pipes and fitting must be done by a trained plumber.

The use of labour intensive construction methods, has for millennia been recognised as a viable and desirable way to mobilize communities and populations to assume responsibility for their own development and to construct in the process works of sometimes monumental proportions.

A labour intensive approach is desirable for the following three reasons:

i. To relieve unemployment in the area;

ii. To generate a sense of shared community responsibility towards the water supply infrastructure;

iii. To reduce costs of the construction.

The organization and day to day supervision of labour intensive construction projects places an additional burden of responsibility on the engineer and his agents; and some formula to reimburse him for the additional duties it imposes, will have to be worked out in advance of the construction phase.

9.3 Operation and maintenance

The effective operation of the water supply depends upon the competence of the Water and Sanitation Committee elected from the local community. Water development projects can be considered an ideal vehicle for achieving community development objectives. The need for secretaries, treasurers, meter readers, maintenance plumbers and standpipe attendants creates a number of permanent jobs. Jobs have been created for a capital investment of less than R300 per job (Rivett-Carnac, 1984).

Local maintenance of the water supply is a worthwhile goal for economic, social and logistic reasons. Maintenance costs on public standpipe systems have been calculated at between 1 percent and 3 percent of the capital cost *per annum* (Rivett-Carnac, 1984). The need to train local maintenance personnel has been clearly demonstrated.

9.4 Financial considerations

The principle of collecting sufficient revenue to cover the recurrent costs of community water supply and household sanitation systems has been more or less universally adopted. Recurrent costs include operational costs, maintenance costs and loan repayments if applicable. The principle of recurrent cost recovery is critical to project sustainability of community water supply projects. Where the capital cost of individual installations is recovered, increased coverage and hence impact of the programme is assured.

> 'Important cost recovery distinctions can therefore be made between household sanitation and community water supply. For the latter, operational expenditures usually represent a much larger share of total project-life costs. Cost coverage relates primarily to operational activities to meet operation, maintenance and replacement expenses. For low-cost sanitation schemes, cost coverage relates primarily to investment activities to meet costs of construction materials, labour, etc. For water supply projects the main goal is sustainability, while for low-cost sanitation schemes, it is expanded, improved latrine coverage.' (Int. Reference Centre Newsletter No. 177, 1988)

The problem of cost recovery - and here the emphasis is not on the financial loan recovery of commercial banks which include their own expenses and profit margins, but on recurrent cost or 'expense coverage' - has been the focus of attention of the World Health Organization, the International Reference Centre and the World Bank for the past decade.

9.4.1 *Cost recovery from public standpipe supplies*

The coupon system (Rivett-Carnac, 1984) is an attempt to overcome the interrelated problems of collecting sufficient revenue to cover recurrent costs; collecting revenue on an equitable basis from consumers according to the principle of those who use more water pay more; reduction of wastage of water; and obviating the problem of the theft of water money. The coupon system is not without its problems. However, it has been operating satisfactorily for over eight years in certain areas near Durban. It is unanimously regarded by the community as a fair system.

The sale of water from water kiosks, which are leased to private entrepreneurs, has been employed by the Department of Development Aid in Inanda near Durban and is described by Alcock (1987).

9.4.2 *Relative water costs*

Domestic water supplies are in essence a system set up to distribute a commodity, namely water. A parallel with manufacturing, distribution and marketing of other commodities can be drawn. In common with other commodities which have unit production, wholesale and retail costs, water too has a unit cost structure called the tariff structure. Water falling from the heavens is by definition free; water supplied from a tap is not. The difference in cost is as a result of the processes that the water has been through in order to reach the tap. These processes could include:

impoundment
abstraction
pumping
storage
treatment
distribution
marketing

Obviously each of the processes has cost implications of its own. Implicit in any calculation of unit costs or tariffs is a knowledge of the variables that directly effect the unit cost. Some of these variables can be measured, some predicted with reasonable accuracy and others merely guessed at. Examples of these are: quantity of water supplied, estimated cost of construction and the rate of inflation respectively.

A comparative analysis of the cost/supply ratio's of various water supply technologies conducted in Natal/KwaZulu during the period 1980 to 1983 yielded the following results shown in Table 9.1 (Rivett-Carnac, 1984).

Of interest in the above figures is the large range of costs from the lower to upper end of the scale (protected springs and mobile tankers) both of which would be considered of low amenity and accessibility value. In contrast municipal water supplies which have the greatest amenity, occupy the middle ground in terms of cost.

An extremely important reason for determining accurate tariff structures, is for equitable cost recovery (selling the water) to achieve economic operation of the water supply system. Some process costs may affect the cost of the water supply more than others. Examples of these in developing areas have been found to be chemicals, energy costs and operators salaries. Obviously any economies that can be found to reduce these costs are of relatively great importance in reducing the final selling tariff of the water. Consequently a good deal of research and development is taking place Worldwide to reduce energy costs by seeking alternative sources of energy; to reduce the need for costly chemicals by using simpler treatment methods; and to make systems simpler to operate and maintain so that expensive and scarce skilled manpower can be spared.

TABLE 9.1 : RELATIVE UNIT COSTS OF VARIOUS WATER SUPPLY TECHNOLOGIES (Rivett-Carnac, 1984)

SOURCE	RANGE OF UNIT COSTS (relative to municipal tariffs)
Protected springs	0,098 to 0,213
Wells (6 m deep)	0,114 to 0,454
Boreholes (30 m deep with handpump)	0,187 to 0,748
Municipal tariff	1,0
Public standpipes (reticulated from municipal mains and operated economically)	3,34
Rainwater collection and storage	3,58 to 7,78
Mobile tanker supply	9,17

Notes

1) Unit costs are relative to municipal water tariff applicable at this time (assumed to be unity in the table). The public standpipe supply was buying water at the municipal tariff and redistributing it from public standpipes controlled by a paid attendant. All maintenance and some capital repayment were covered from the sale of water.

2) Unit costs are for equal volumes of water and have been arrived at from an analysis of the installation costs, the number of users, the per capita consumption and the design life of the system, in the case of springs, wells, boreholes and rainwater systems.

3) Unit costs in the case of mobile tanker supply were based on installation costs plus operating costs divided by the volume of water supplied to Inanda during the emergency water supply during 1980.

Capital costs for installations are subject to continuous inflation and can only become greater with the passage of time. Water supply systems that were built in the past supply cheaper water than those built today, which in turn are cheaper than those which will be built tomorrow. Delays in the provision of water supplies result in higher capital costs and hence higher selling prices of water in the future.

9.5 Training

The desirability of improving community institutions and fostering organizational capability can be regarded by certain governments with suspicion. For this reason it is best to confine organizational and training efforts to strictly sectoral activities. In order for development efforts to succeed in the water supply and sanitation sector, some institutional training is essential. Research to date has indicated the need for training in the following fields listed in Table 9.2.

TABLE 9.2 : COMMUNITY TRAINING PRIORITIES FOR THE WATER AND SANITATION SECTOR (Rivett-Carnac, 1984)

Constituting and running of water and sanitation committees
Bookkeeping, accounting and recording of minutes
Training of plumbers for constructing and maintaining unsophisticated water distribution systems
Training of borehole pump maintenance personnel
Training of latrine and water storage tank builders
Training of community health educators
Training of primary school teachers in the fundamentals of domestic water supply, management, public health awareness and personal hygiene

The single most important problem in water supply programmes has been identified by the World Bank as 'institutional weakness at all levels'. The World Bank has as a consequence shifted the emphasis of its loans policy to support project-related training. Once it is satisfied that the organization, management and staffing on the recipient body is adequate to construct, operate and maintain the water supply facility, it is prepared to finance the loan. Training of waterworks personnel is at present virtually confined to in-service training on municipal works. Public health education, which should be directed at as wide a public as possible, particularly primary school teachers and community health workers, is at present con fined to an 'elite' or lower echelon civil servants. A great deal more training of skilled and semi-skilled personnel is needed in this sector.

Communities must be encouraged to achieve higher levels of sanitation and hygiene. Educational programmes should be run in parallel with water development programmes and emphasis must be placed on primary school education, if the full benefits of improved water supplies and sanitation are to be realised.

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APPENDIX B

Flow meter for measuring the discharge of small pumps

Flow Meter for Measuring the Discharge of Small Pumps

*by Y.M. Sternberg, M. Silver and S.V. Allison**

A recurrent problem faced by extension workers and volunteers involved in small-scale irrigation projects in developing countries is the difficulty of measuring the discharge of small pumps. Happily for them, a simple and low-cost measuring device has recently been developed and is described here by its inventors.

Conventional methods of flow measurement

Flow measurements are an essential part of good on-farm water management, for unless the discharge of a pump is known, it is totally impossible to plan irrigation schedules intelligently. There are at least three conventional ways in which to measure the discharge from a small pump. The most common is probably the *drum and stopwatch* method, in which the flow from the pump is directed into an open-topped oil drum and the time required to fill the drum is recorded. The second alternative is to place a *notched* weir or *specially-shaped flume* in the supply ditch and to measure either the head upstream of the weir or the head differential across the flume. The third way is to use *commercial* or *scientific flow meters:* the latter employ the principles of ultrasonic waves or heat sources. Simpler ones are known as venturi meters and are based on the venturi system.

Each of these methods has its limitations. The main disadvantage of the drum and water approach is that the pump discharge has to be about 3.5 ft above the ground; there may not be clearance below the pump discharge to do this, or if a flexible pipe discharge is being used, the additional head imposed may lead to an artificially low indication of discharge from the pump. Also, a drum is an extremely cumbersome object to carry around. The notched weir and flume methods require the construction and/or installation of the device, an accurate measurement of the head or head differential, and the availability of a calibration table. Commercial water meters, typically designed for use in high pressure pipe systems, such as the water distribution network found in urban areas, generally impose severe head-loss penalties. Most scientific flow meters do not have this limitation but tend to be rather delicate and are expensive. None of these devices has the degree of portability desired by extension workers, who may wish to check discharges from many quite widely-scattered pumps in the course of a day. The device described here, known as the hole-in-the-bucket flow meter is capable of measuring flow in the range of 5-30 gallons per minute (gpm). It can be used to measure any flow which is discharged into the atmosphere, including pumps, provided the outflow pipes can be directed into the bucket. It obviously cannot be used in a closed pipe network system.

Proposed flow meter

The device, shown in Fig. 1, consists of a two-gallon plastic bucket with eight holes of ¼" diameter drilled in the bottom and corks to plug the holes. If a plastic bucket is not available, another type of container could be used provided it is properly calibrated. A supporting frame can be made of wood or metal — its sole function is to support the bucket. In lieu of a supporting frame, the bucket could be held but this is not recommended as it would be difficult to hold the bucket stationary for even short periods of time. At a certain distance from the bottom, the bucket is marked with a horizontal line to indicate the depth at which the flow should be stabilized, that is, when inflow equals outflow: this is determined from the calibration test which is described later. In operation, water enters the bucket from the top and flows out through the holes. The water level in the bucket is brought as close to the horizontal line as possible by plugging and unplugging holes as needed until

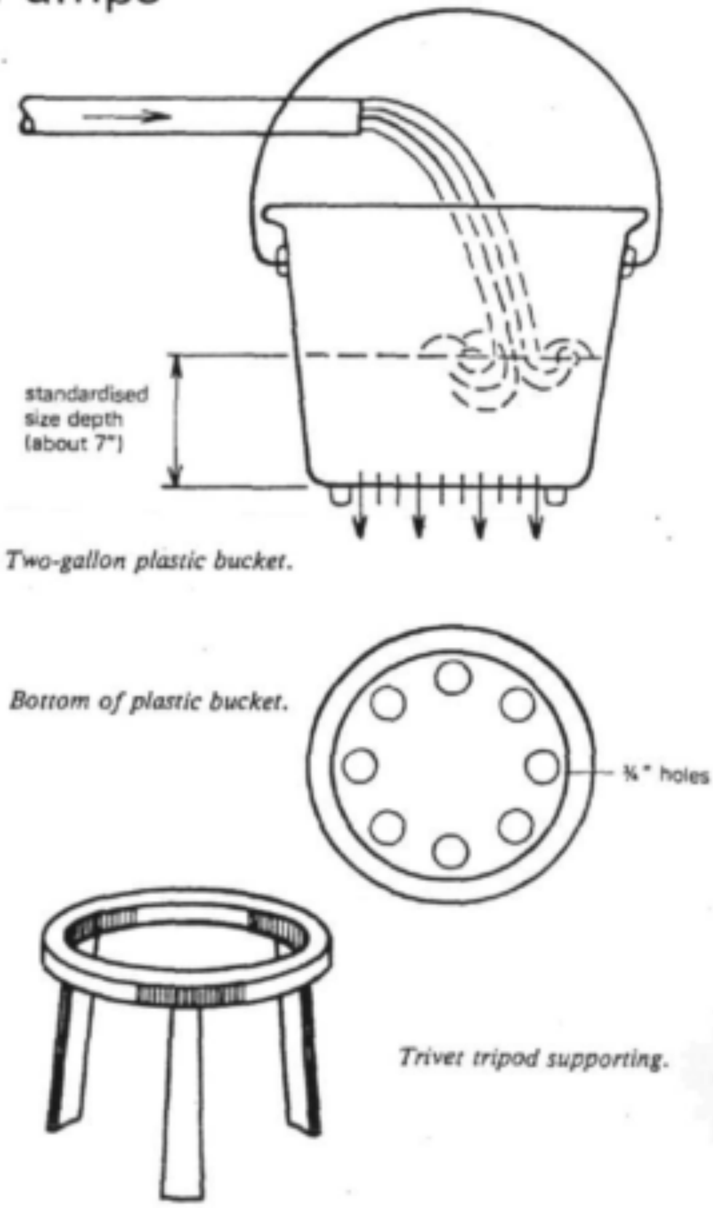

Fig. 1 Bucket and supporting frame.

† The eighth hole was continuously closed because the maximum inflow available was 30 gpm.

Table 1. Calibration chart (based on stabilized depth of 7")

No. of open holes	*Flow (US gpm)*	*l/s*
1	5	0.38
2	10	0.76
3	15	1.14
4	20	1.52
6	25	1.90
7	30	2.28

equilibrium is reached. The number of open holes corresponds to a given discharge, whose value is determined from calibration tests.

Calibration procedure

Discharges corresponding to a water depth of 2-10" above the bottom of the bucket were determined for one to seven ¼" holes: the flow was first measured for a single hole with a 2" depth. Discharge from an inlet pipe, equipped with an accurate flow meter, was adjusted until the water in the bucket stabilized at the desired 2" depth. This procedure was repeated three times and the average discharge recorded. The entire process was then repeated to determine the discharges which occur when the water is stabilized at 4", 6", 8" and 10" depth from the bottom of the bucket. In this manner the bucket was calibrated for one through seven† ¼" holes.

Results

It is evident that the same discharge can be obtained from a wide range of depths and open holes. For example, a discharge of 5 US gpm can be obtained from a flow through one hole with a depth of 6.7" or by flow through two holes at a depth of 2". Statistical analysis of the data indicates that for the 5-30 gpm range the smallest error in discharge (about 6%) occurs at a depth of 7". It turns out, very conveniently, that a ¼" hole will discharge about 5 US gpm under a head of about 7". The recommended calibration chart is given in Table 1.

Conclusion

The hole-in-the-bucket meter is sufficiently accurate for most practical purposes. There is no point in measuring a pump discharge with better accuracy when the efficiency of water distribution varies from farmer-to-farmer, field-to-field and day-to-day by a far greater margin than the accuracy of the meter. Thus, the hole-in-the-bucket flow meter is an inexpensive device, easy to manufacture, portable and sufficiently accurate for field work. ●

0305-0920/82/0404-0159/$1.50/0

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