

1 Introduction

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1 Introduction

Water is essential to man, animals and plants. Without water life on earth would not exist. From the very beginning of human civilization, families have settled close to water sources, along rivers, beside lakes or near natural springs. Indeed where people live, some water is normally available for drinking, domestic use, and possibly for watering animals. This does not imply that the source is convenient and of sufficient capacity nor that the water is safe and wholesome. On the contrary, in many countries people live in areas where water is scarce. Often women and children carry it over long distances, particularly during dry periods. Scarcity of water may also lead people to use sources that are contaminated by human or animal faeces, and are thus dangerous to human health.

1.1. Water supply, health and socio-economic development

A few litres of water each day are sufficient for a person's basic drinking and food preparation requirements, depending on climate and lifestyle. Larger quantities are necessary when water is used for personal hygiene, cleansing of cooking utensils, laundry and house cleaning and for meeting basic economic needs within the household, such as the raising of animals and the production of vegetables. Safe, adequate and accessible supplies of water, combined with proper sanitation, are surely basic needs and essential components of primary health care. They can greatly help improve the health of underprivileged populations in rural and urban fringe areas. They are also a source of, and condition for, socio-economic development.

Improved hygiene and health

A sufficient amount of safe drinking water is important in the control of many diseases. The World Health Organization (WHO) has estimated that as many as 80 percent of all infectious diseases in the world are associated with insufficient and unsafe water. This is particularly well established for diseases such as diarrhoeas, cholera, typhoid and paratyphoid fever, infectious hepatitis, amoebic and bacillary dysentery. Table 1.1 gives an overview, along with preventive measures. Which diseases are most prevalent and serious in a particular area depends on local factors such as climate, density of settlement, local practices and socio-economic and gender relations. The type of intervention must therefore also be locally specific and be chosen together with the local users, women and men.

Table 1.1 Transmission patterns and preventive measures for water and sanitation-related diseases

Infection	Transmission pattern	Personal hygiene	Domestic hygiene (and animal management)	Food hygiene	Water hygiene / safe water consumption	Safe human water consumption	Waste-water disposal and drainage
Various types of diarrhoeas, dysenteries, typhoid and paratyphoid	From human faeces to mouth (faecal-oral) via multiple routes of faecally contaminated water, fingers and hands, food, soil and surfaces (see Figure 1). Animal faeces (e.g. from pigs and chickens) may also contain diarrhoeal disease organisms.	●	●	●	●	●	
Roundworm (Ascariasis), Whipworm (Trichuriasis)	From faeces to mouth: Worm eggs in human faeces have to reach soil to develop into an infective stage before being ingested through raw food, dirty hands and playing with things that have been in contact with infected soil. Soil on feet and shoes can transport eggs long distances. Animals eating human faeces pass on the eggs in their own faeces.	●	●	●		●	
Hookworm	From faeces to skin (especially feet): Worm eggs in the faeces have to reach moist soil, where they hatch into larvae which enter the skin of people's feet.		●			●	
Beef and pork tapeworms	From faeces to animals to humans: Worm eggs in human faeces are ingested by a cow or pig where they develop into infective cysts in the animal's muscles. Transmission occurs when a person eats raw or insufficiently cooked meat.			●		●	
Schistosomiasis (bilharzia)	From faeces or urine to skin: Worm eggs in human faeces or urine have to reach water where they hatch and enter snails. In the snails they develop and are passed on as free swimming <i>cercariae</i> that penetrate the skin when people come into contact with infested waters. In the Asian version of the infection, animal faeces also contain eggs.	●	●			●	
Guinea worm	From skin to mouth: The worm discharges larvae from a wound in a person's leg while in water. These larvae are swallowed by tiny water fleas (cyclops), and people are infected when they drink this contaminated water.				●		
Scabies, ringworm, yaws	From skin to skin: Both through direct skin contact and through sharing of clothes, bedclothes and towels.	●	●				
Trachoma, conjunctivitis	From eyes to eyes: Both direct contact with the discharge from an infected eye and through contact with articles soiled by a discharge, such as towels, bedding, clothing, wash basins, washing water. Flies may also act as transmission agents.	●	●				



Infection	Transmission pattern	Personal hygiene	Domestic hygiene (and animal management)	Food hygiene	Water hygiene / safe water consumption	Safe human water consumption	Waste-water disposal and drainage
Louse-borne typhus, Louse-borne relapsing fever	From person to person: Through bites of body lice which travel from person to person through direct contact and through sharing clothes and bedclothes, particularly when underwear is not regularly washed.	●	●				
Malaria, yellow fever, dengue	From person to person through the bite of an infected mosquito. The mosquitoes breed in standing water.		●			●	
Bancroftian Filariasis	From person to person through numerous bites by infected mosquitoes. The mosquitoes breed in dirty water.	●		●			●

Source: Boot, M. and Cairncross, S. (eds.), 1993

Diseases caused by a lack of water are a serious health hazard. When women, men and children use very little water, either because there is little available or because it is too far away to be carried home in quantity, it becomes impossible to maintain a reasonable personal hygiene. There may simply be too little water for washing oneself properly and cleaning food, utensils and clothes. Easy access to a sufficient amount of water, reliably delivered to every household, is essential for the prevention of diarrhoeas, dysenteries, typhoid, schistosomiasis and skin and eye diseases.

Diarrhoeas in particular remain a killer disease, especially for babies and children below the age of five. Research has shown that in diarrhoea prevention more water used for hygiene and better sanitation is more important than better water quality. It has shown too that for a positive impact, at least three quarters of the families must use, and be able to continue to use, sufficient amounts of water for hygiene. They must also use hygienic methods of excreta disposal (Esrey, 1994).

These conditions have important implications for the design and planning of water improvements. Projects that aim to improve health must not only improve water services, but also sanitation and hygiene behaviour. For sanitation, it is often necessary first to raise the demand for improvements. For hygiene, information alone is insufficient. Improved hygiene behaviour comes not from just telling women and men, girls and boys how diseases are transmitted or what to do and not to do. High quality hygiene promotion programmes are needed which use participatory learning and action methods or effectively market a small number of locally desired improvements.

To improve health, improved water services need a critical mass of users. Year-round use by almost all people only happens when the users both want to use and sustain

a service effectively, and are able to do so. The implication is that the outsiders involved in a water project or programme - politicians, planners, engineers, sociologists, economists, educators - cannot impose a service. They must create a service for widespread and sustained use that meets the perceived needs of the users. This can only be done in partnership with those who will use and sustain it: the community women and men and their organisations.

Water quality, quantity and drainage

Water that is contaminated by people or animals will transmit diseases that are water borne, such as bacillary dysentery, cholera, or typhoid, or those that are water based, that is need snails or cyclops that live in the water to pass the disease on. Examples of water-based diseases are guinea worm and schistosomiasis. Control of these diseases means improving the quality of the drinking water and discouraging people from wading into the source, e.g. to bathe, collect drinking water or wash clothes. Many diarrhoeal diseases are caused more by a lack of an adequate quantity of water than by its quality.

During warm weather, biting insects are common. Most of these, notably mosquitoes, breed in pools or other open water, and sometimes even in household water containers. Tsetse flies are also active near water. An improved water service may actually increase health risks from insect breeding. The risks arise when the *water in = water out* principle is neglected (i.e. no provision is made for safe drainage of wastewater) or when the design allows breeding in the system itself. Pools of stagnant wastewater from lack of good drainage at distribution outlets are unfortunately still very common. They, and/or the stagnant water in other parts of the system have brought new or extra transmission risks of malaria, filariasis, dengue and yellow fever.

Socio-economic development

As well as better public health, improved water services are also a source of, and condition for, socio-economic development. Development comes both from the services themselves and from the processes of their planning, establishment and management. The water supply itself reduces the drudgery of women and children if it brings water closer to or into their homes, in acceptable amounts and of the desired quality, throughout the year.

Reducing time and energy for water collection may bring more equity between the sexes. In many tropical countries women work longer hours than men due to, among other things, the long distances for collecting water and fuel wood. The carrying sometimes takes as much energy as the heaviest agricultural tasks done by men, with the difference that water carrying is a daily chore. The reduced working hours and increased rest brought by an improved water supply benefit women's health and thereby the well-being of the whole family. Local access also increases the safety of

women and children who no longer need to leave their communities for their daily water requirements.

Alternatively, women have chosen to use time and energy savings of an improved water supply for their domestic, economic, social and managerial tasks. Time saved is used for household and child care, including the collection of more water for hygiene. Or it is used to free children from water carrying and give them more time for school and/or play. Unfortunately, these benefits often still go only to boy children.

Women also use time and water gains to increase their domestic productivity. They utilise them, for example, for animal raising and vegetable gardening. Cases of successful productive use, e.g. in micro-enterprises, are found in situations where time gains have been large and reliable and women have access to organisation, training, credit and markets. Men have also used water supplies productively, for example, for animal raising and brick making. Economic studies have shown that the income raised is an important contribution to family livelihood and comes at critical times when income from other sources is not available.

Less time and energy for water collection has also benefited women's many social tasks in the extended family and as neighbours, and their work in community development, including work for the local water service. There are, however, also examples of negative impacts. A new water service has, for example, reduced the opportunities for young women and girls to move outside their house. Or it has increased women's work in agriculture or animal husbandry, but this work has not benefited them and their children, as husbands spend the proceeds on other than family interests. There have also been examples of competition and conflicts over water between different user groups, when the design and management has overlooked the different demands for domestic and productive use of water between and within households.

Development benefits also come from the process of participatory planning, implementation and management. Involving women and men community members from the different sections of the community in the planning and establishment of the new service means that local resources are recognised and utilised. At the same time, the involved groups acquire new knowledge and capabilities. The water services also create new functions, jobs and, sometimes, whole enterprises. Positive experience and new skills in transparent and socially equitable community organisation, decision-making, financing and management benefit not only the water service but also other community development projects. More details on planning, implementation and management processes and their social equity aspects may be found in the next chapter.

1.2 Small community water supplies in developing countries

Communities in developing countries that want to establish and run an improved domestic water supply vary greatly. Some are small and rather isolated. They may have a scattered population, a subsistence rather than a cash economy, a limited demand for water, strong leadership but with limited administrative skills, a strong social cohesion and a long tradition of self-help and mutual support.

Others are bustling trading centres. They are situated on major roads and railways, have a cash economy, administratively experienced leaders, considerable differences between the rich and the poor, a great variation in water demands and little time and readiness for voluntary community development activities.

Some are on the outskirts of major cities or are low-income communities in inner cities. The legal status and social, cultural and economic characteristics of these more urban communities usually varies greatly. Some are former rural communities that have been engulfed by the urbanisation process, with a legal status, a high social cohesion, and a varying quality of initiative and leadership. Others are the result of urban migration and unauthorised settlement. They have been formed through an organised invasion of rural families from the same rural area under local leaders. Or they have resulted from a gradual process of settlement either by male household heads and single men who left their families in the rural areas or by whole families from the same villages or from quite different rural areas.

In the past, projects and programmes have often overlooked the different nature and history of small communities. They have applied a so-called blanket approach, using the same technology and service level, and the same maintenance, management and financing systems in every community. Implementation followed the same national design criteria and the same technical and social processes everywhere, irrespective of local social and economic conditions. It is hardly surprising that the resulting services were often unsustainable.

Nowadays, water projects and programmes increasingly recognise that different types of communities want and can sustain different solutions, not only for technologies and service levels, but also with regard to local maintenance, management and financing arrangements. The technology options may range from the improvement of the existing indigenous water sources and water transport systems to the installation of new water supplies with public facilities, group facilities and/or private facilities. Other options are a combination of traditional and improved systems for complementary use or a sequence of different systems during the rainy and wet season. One example of a climbing frame of water technology options is given in figure 1.1.

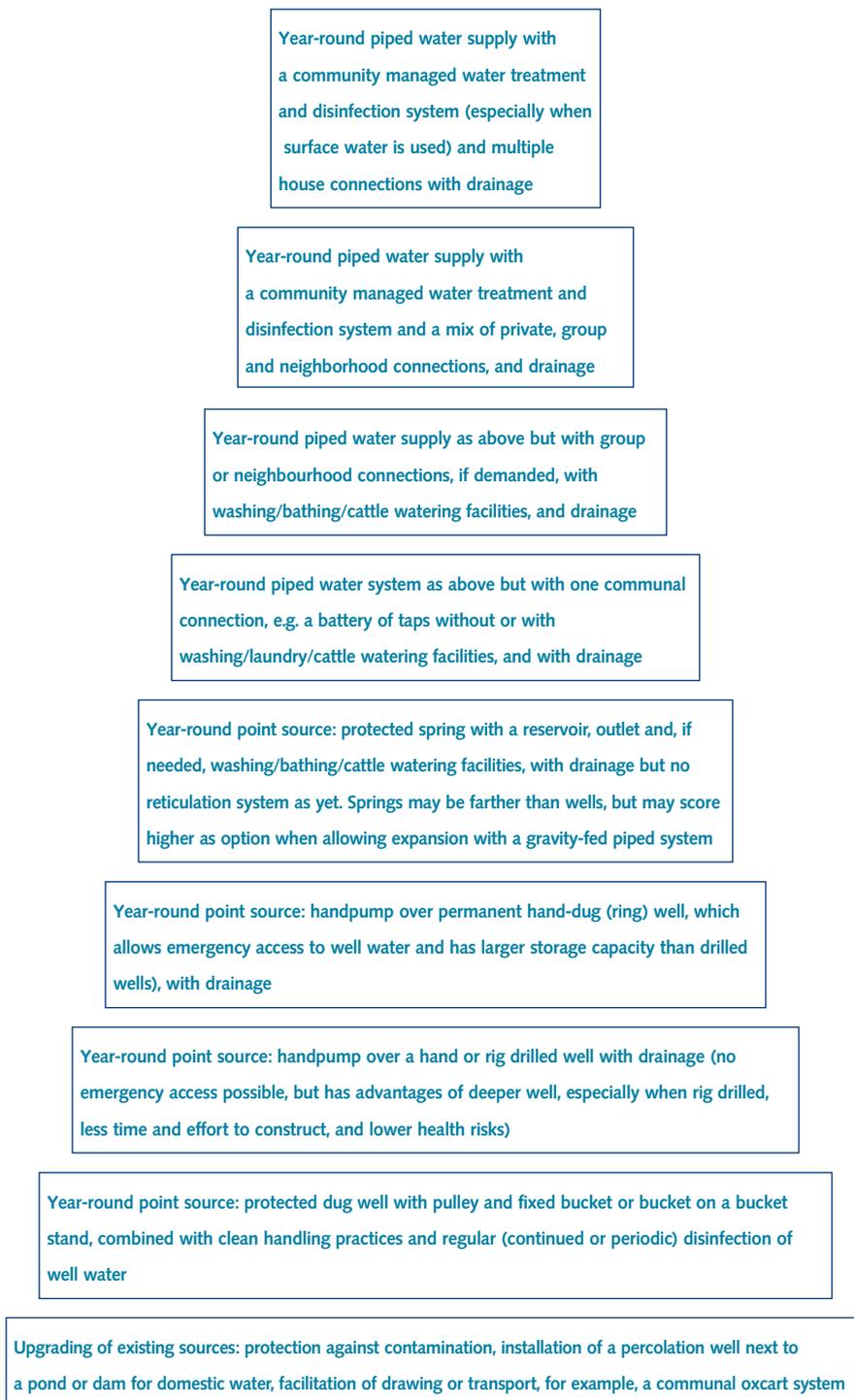


Fig. 1.1. A climbing frame of choices for improved community water supplies

Apart from helping communities to match their choice to their current needs and potentials, it is also possible to choose designs that allow moving up or down the climbing frame. A piped water supply, for example, may be designed in such a way that later, when the community has become more developed, it may be expanded with a distribution network. Alternatively, a user group may decide on dug wells rather than drilled wells with handpumps, because this makes it possible to open the well or use the manhole when for one reason or another a handpump cannot be repaired. In such a case, the users should be made well aware of the risks of contamination and be ready and able to protect the water quality through a safe alternative way of drawing (e.g. with clean buckets and ropes) and well chlorination. Where user groups decide to go for wells that are only operable/operated through handpumps, they need to take account of the scope for speedy repair and temporary sharing arrangements in the case of breakdown.

In the past, external projects have one-sidedly decided not to give such options to the users and sustainers of water supplies, because of the risk of contamination. This has only meant that when handpumps on drilled wells broke down and could not be repaired either immediately or at all, the wells became useless. The women had either to use another well with problems of distance, queuing, conflict and a lower water use, or turn to other, and riskier water sources.

Considering the pros and cons and the implications of each option and comparing them with the demands of the different user groups and the available resources is therefore a crucial part of decision-making. The processes for such planning and decision-making are discussed in chapter 2, along with some of the emerging options for maintenance, management, and financing of water services.

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2 Planning and management

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2 Planning and management

The last two decades have seen big changes in the approaches to planning and managing small community water supplies. When this book was first published in 1981, community participation in water projects was certainly seen as important. Water agencies had recognised the benefits of involving local people in construction, operation and maintenance of their own water systems. There was an emerging trend, too, for decisions about the design and financing of water supply improvements to be taken in partnership with the community.

That trend has continued apace. It has evolved via participatory approaches that try to ensure that development is community driven, reflects the true aspirations of all sections of society, and is both gender and poverty sensitive. At the same time, there has been a growing recognition that water supply improvements alone do not bring optimum health and development impact. Better sanitation provisions, changes in hygiene behaviour and linkages with other livelihood inputs are the complimentary activities needed to yield the full benefits.

So, the institutional framework and organisational models for planning and management of community water supplies have to be inclusive in terms of both society and sectoral interests. The role of water agency staff is very much one of motivators, facilitators and supporters of community-led programmes.

2.1 Planning for community managed water supply systems

Project versus programme

The planning and design of water supply systems in large communities is usually approached as a single project. The term *project* encompasses all the preparations for the construction of a particular scheme or water supply system. Because each community and system is different, each project is unique. There is still a need for the inclusive approach, to avoid marginalising the poor, but the ways of ensuring sustainability in large urban settings are outside the scope of this publication. For planning with a large number of small communities, a programme approach is more efficient and effective than a project approach. A *programme* is a series of integrated activities directed at the establishment and continued functioning and use of a considerable number of similar water supply services. Because the communities themselves manage the services, the technologies on offer should fit the different levels of complexity and costs that the communities concerned can manage. Choices should also not be dominated by local political interests. The challenges of a programme approach are therefore social, organisational and administrative. This has implications for the manner of implementation and the kind of expertise required.

Planning will depend on strategies adopted by the supporting agency and on basic principles that are emerging in the water supply and sanitation sector. One principle is to involve the different groups in communities right from the start of the planning process. Support agencies, community groups and users should work together as partners, and plan their activities based on mutual agreement. The latter is particularly important in contexts where communities are taking more and more responsibilities for operating, maintaining and managing their water supply schemes. Experience shows that effective management is determined to a considerable extent by non-technical issues. Therefore, the agency staff involved in planning and assessing water supply systems should come from a wide range of disciplines – social development, economics, health, environment, management and engineering.

Demand-responsive programmes

Demand-responsive programmes give each community and the various groups in that community an informed choice of services and service management systems. This means that all locally relevant groups, or *stakeholders*, get information on all relevant aspects and implications of the various water supply options. The information may include: the amount and quality of water provided; the purposes for which this water will be adequate; potential implications for health and socio-economic development; investment and recurrent costs involved; approximate walking distances; requirements and possibilities for sharing of service and costs (for example, through group connections or by forming user groups); prospects for service regularity and reliability; and differences in ease of maintenance (for example spare parts and technical skills required) and administration.

Table 2.1 Comparative advantage of community managed projects

A comparative quantitative evaluation of small community managed water services showed that

- communities with demand responsive projects (i.e. community households, both men and women, involved in planning decisions) had better sustained and used water services; communities with more (owner) rights and with capacity building for local management had better sustained services;
- household contributions to construction were only significantly associated with better sustained services if coupled with more rights and capacity building for local management;
- having a local water management organisation composed of women and men members correlated with greater access for all, especially when there were more poor people among committee members;



- when there was better representation of women and people from poor families in the water management, poor women more often perceived themselves to have influence on the water service;
- the more directly democratic and gender sensitive the planning of the technology/service levels and maintenance system (that is, with men and women from the households), the better the services had covered their cost over the last three years;
- cost recovery was also better with more community control and accountability, i.e., when local water management organisations had had some control over design and construction, applied local rules on water use/management, accounted to users for management and had a legal position.

Source: Gross et al., 2001

Comprehensive planning

The eventual local solution for an improved water supply system is the product of comprehensive planning, considering all community, technical, environmental, financial and institutional aspects.

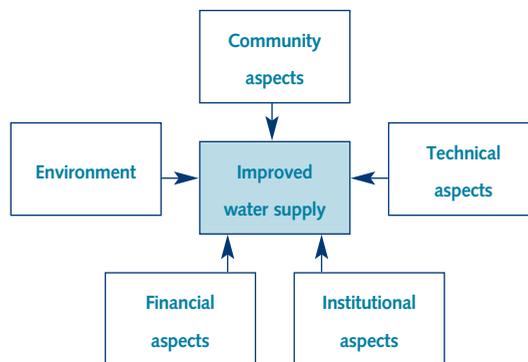


Fig. 2.1. Comprehensive planning

The stakeholders who make these choices are the groups that will use and sustain the service and thereby determine its success. At the community level, they are the local leadership, the male and female heads in each household who will be the future users and tariff payers, and the local organisation that will manage the participation process and possibly also the service after its establishment. The latter may be an elected committee or sub-committee, but other management forms are possible. It is thus very important that all these groups know what the implications of the various options are and can choose the option or combination of options that best fit their local needs, conditions and resources.

Linking water supply, hygiene and sanitation

We saw in chapter 1 that for sustainable public health benefits, 75-80% of the population have to use enough water of an adequate quality the year round and practise safe sanitation and good hygiene. It follows that small community water supply programmes need to cooperate with effective programmes for improved sanitation and hygiene behaviour.

In rural communities, households frequently have a choice between alternative water sources and women have developed criteria to choose which source to use for what purpose. This means that households will not automatically use and sustain an improved water supply. A strong demand for good quality water may initially only exist in situations of severe shortages or of heavy pollution of the water sources. In the same way, demand for sanitation in rural and peri-urban communities is often low, especially among men. Women and adolescent girls face more privacy and safety problems and so have a greater demand. However, often they can only express these needs within the family. For the true demand to be reflected in programming, there has to be gender sensitivity in the participatory planning.

Enhancing the demands for better water use, sanitation and hygiene is thus imperative to meet longer-term health benefits of environmental engineering. In the last decade considerable knowledge has been achieved on how to do this effectively. Part of this knowledge is that water engineers cannot consider local projects in isolation from the existing alternatives. To be accepted, used and sustained, the new provisions have to be and remain better than the other alternatives in terms of economic and social costs and benefits. It does not work just to tell people that this is so, or for programmes to "educate" villagers to do what outsiders consider best for them. Programme teams that support community projects must seek and value the local experiences and viewpoints, particularly from poor women and men, to understand better what local people really want and can use and sustain.

Planning and assessment

The comprehensive participatory planning illustrated in figure 2.1 involves assessment of and consultations on a wide range of different aspects. Some of the key criteria to be considered in planning are listed in table 2.2.

Table 2.2 Assessment criteria for effective planning

Assessment criteria	Key components
Community aspects	<ul style="list-style-type: none"> • Demand for water supply improvement and desired service level (men and women, including marginalised groups) • Perception of benefits related to an improvement of water supply (men and women, including marginalised groups) • Responsibility and ownership feelings • Culture, habits, beliefs related to water and hygiene • Presence of alternative water sources • Organised and elected community group to be responsible for operation and maintenance (representative of the community social structure, including men and women) • Managerial and technical capacity of the community group, and availability of tools • Possibility of grouping several communities for a single water supply scheme (in the case of small piped systems)
Technical aspects	<ul style="list-style-type: none"> • Present and future water consumption • Need to include water treatment • Technical standards and complexity of O&M procedures, with a preference for technologies that can be operated and maintained at community level • Quality, longevity and cost of equipment • Cost and availability/accessibility of spare parts, and potential for local manufacture of spare parts, as well as standardisation • Dependence on and cost of fuel, power and chemicals, if needed, with a preference for reducing this dependency
Environmental aspects	<ul style="list-style-type: none"> • Quantity and quality of water resource, including the need for water treatment, water resource management and seasonal variations • Water source protection and wastewater management
Financial aspects	<ul style="list-style-type: none"> • Cost – benefit analysis • Ability and willingness to pay • Tariff structure (covering O&M and replacement costs), with due consideration for marginalised groups • Cost-recovery procedures and financial management capacity • Alternative financial mechanisms, in case of major problems • Economies of scale



Institutional aspects

- Legal framework and national strategy
- Training availability and capacity
- Follow-up support, including monitoring
- Availability of technical assistance to the communities (NGOs, municipal and/or district level)
- Availability and capacity of local craftsmanship
- Involvement of formal and informal private sector
- Capacity of technical staff to deal with community development and knowledge of participatory processes

All communities have at least one form of water supply. From the perspectives of all these criteria it is wise to consider all forms of improvements, from improvement or upgrading of existing traditional and/or improved water supplies to the establishment of one or more supplementary provisions. Building a new system that replaces all others is not the only and necessarily best choice. On some dry plateaus with low density settlement for example, a sequence of water supplies, starting with rainwater harvesting and surface water use at the peak of the rainy season, and followed by using protected wells on the plateau and, when these dried up, handpump wells in the river valleys (sealed and with the pumps kept by the committees when they were not used) has been the agreed option.

Designing for current and future needs

To serve the needs of all groups, small community water supply designs must further be holistic, expandable and upgradable.

In a *holistic* design, the local water supply system or combination of systems meets all the basic water needs of the people. In small communities, separating water between domestic and productive uses is often unrealistic. As well as the family needs for drinking, cooking, bathing, sanitation and hygiene, both women and men often need domestic water for small-scale productive uses within the household, such as raising animals, growing vegetables and trees, processing food and making bricks. Alternative water sources are not always available for such uses. Even if there are several sources it is unlikely that there will be clear enforceable arrangements made on which source will be used for which purposes. Unless local design, management and financing arrangements encompass all the different needs, competition and conflicts over water are the result.

Designing for *expandability* is required because programmes seldom can go back to communities some years later to install new water points for new households and neighbourhoods. Social and health benefits may be lost unless service coverage levels

can be kept up and newly settled families also have access to the improved water supply. It is therefore important that, in choosing the technology and service levels, the community and its advisers also consider the system's expandability within the locally available technical and financial means.

Upgradability is an issue because improvements in basic services can be expected to lead to socio-economic growth. Communities may therefore wish to design for the future and accept the implication of some extra community investments upfront to increase the possibilities for later upgrading.

Rehabilitation of defective water supply schemes can be an alternative to investing in a new project, but that decision should not be taken lightly. The rehabilitation option has to be evaluated, as one would do with a new scheme, by taking into consideration the community's needs, preferences and capacities to sustain whatever is undertaken, as well as the support capacity of water agencies. In assessing the scope for rehabilitation, community members and support agencies should use a participatory problem analysis to review the reasons why the system needs to be rehabilitated, and carefully examine other feasible options. Rehabilitation should not simply be a matter of replacing defective equipment or repairing damaged infrastructure. The most common causes of failure are poor organisational/financial capacity and inappropriate choice of technology.

Standardisation

Several countries have chosen to standardise their choice of technology. There are positive and negative aspects that should be carefully considered before making a decision to standardise (see table 2.3). Standardisation can only last a certain number of years, as technological progress, price factors and new life styles/patterns will influence the level of service and choice of technology.

Table 2.3 For or against standardisation

For standardisation	Against standardisation
<ul style="list-style-type: none"> • Wide use of the same type of equipment encourages agents and shopkeepers to store and supply spare parts because of an important and foreseeable demand • Proliferation of brands and technology makes it difficult to organise an efficient spare parts availability system • Prices and markets can become more accessible 	<ul style="list-style-type: none"> • The chosen technology does not respond to the needs and preference of the users • The market is closed for new, innovative and cheaper technologies • Poor incentive for the involvement of the private sector • Possible conflict between country and donor policy on choice of technology



- Users become familiar with one type of technology and product
- Training of personnel can be standardised
- Competition between different brands can help prices to go down and improve efficiency

2.2 Participatory processes

How are demand-responsive programmes implemented? A number of new approaches are emerging for community planned and community managed water services, with more participatory and gender and poverty sensitive strategies, organisations and methods.

Participatory planning and design is an intensive and crucial process. It is continually necessary to consult separately with the different community groups about the different options. Because of the great technical, socio-economic and cultural variation, each programme will need to work out its own range of options and the information and tools that it will use with the groups concerned. Decisions on community financing alone, for example, cover a wide range of choices: how much should be paid to reflect local costs and age of systems; equitable payment systems; division of responsibilities and work in collection and administration; accounting and accountability; etc. There is an increasing amount of field experience in the literature about what works best under what circumstances. There are also a growing number of reference documents and advisory services that help identify the kind of choices that programmes may consider.

A programme fund to (co-)finance locally planned projects

Small communities frequently find it difficult to obtain the capital to construct improved water supplies, so local efforts seldom develop into larger programmes without some sort of outside support. The initiative for organising and financing multi-community programmes usually comes from the central or provincial government. Establishing such a programme, and creating a programme fund, facilitates the local initiation of a succession of small community water supply projects. The fund may be partly revolving, that is, it is a centrally established fund that finances new projects using repayments on earlier loans.

The communities that agree to take a loan, or qualify for a grant, or a combination of the two, contribute to the costs of planning, design and construction with their time, cash, local knowledge and expertise, materials and labour. They also manage the local planning process through their own organisation (not necessarily specific to water supply). This organisation makes the arrangements for the construction of the installations, involving community women and men, artisans and/or local contractors. It has to account for the use of the funds to the programme agency and to the women and men from the community.

Participatory assessments during the pre-planning stage

Using a demand-responsive approach implies that, before obtaining a loan or a grant, individual communities have the opportunity to assess the demands within their households and submit a pre-proposal to the programme. Candidates for funding may be all interested communities in selected programme areas, although programmes also sometimes formulate eligibility criteria.

Communities are not homogeneous entities. They consist of different groups broadly representing those that are better off, the middle classes and the worse-off/marginalised groups. In each of these groups, women and men have their own needs, knowledge and potentials. To help and get support from all, it is important to identify the various groups at the very start of the process and ensure their equitable participation. It is often necessary to meet separately with each group, because in many cultures women and poor people cannot or will not attend and speak out at large meetings. Their views have to be obtained through gatherings where they can participate in an uninhibited way. Trusted individual local men and women then convey the views to a larger general meeting and report back to a second gathering later.

The local organisation that organises the pre-planning and planning process is the heart and soul and the driving force of the whole participation process. It must be able to represent the interests of all community groups. From the beginning, it needs to have an equitable representation of both women and men from the major groups in the community. External agency staff assist this organisation to assess the existing conditions and needs for improvements and to prepare a pre-proposal for improvements. The organisation also needs to be able to submit the proposed plan for approval to all the women and men in the different socio-economic (and sometimes also different religious and ethnic) groups.

Preliminary project selection

No community exists without having some kind of water supply system. Projects can therefore not be planned in a vacuum, but must be based on the assessment of existing conditions. Assessment of existing water supplies and their technical, financial, administrative, social and environmental shortcomings is therefore a part of any rapid feasibility/baseline study. Rapid participatory appraisal methods such as the *Methodology for Participatory Assessment of Community Water Services* (Wijk, 2001) facilitate such an evaluation.

Having assessed the existing conditions and laid down the demands, needs and potentials for improvement, each community submits its resulting preliminary plans, or pre-proposal, to the programme level. At this level, the first prioritisation and selection of preliminary proposals takes place, followed by the allocation of financial and human

resources to help with detailed planning and design. In a participatory programme, such decisions are ideally also a joint process, in which women and men community representatives from the concerned communities participate. In the process, special care is needed to ensure that weaker communities are not disadvantaged. Those with less development, weak leadership, or suffering social conflict often require more assistance and/or more time to complete the process. Already advantaged communities may otherwise dominate the competition for project resources.

Detailed planning and decision-making

Once a proposal has been selected and resources have been assigned, the next stage is detailed planning and design. Programme support teams (discussed later) now use their social and technical know-how to help the communities to consider the pros and cons of different technical and organisational options and make informed choices. The choices cover a wide range of issues:

- the types of technology and the levels of service that best suit the different sections of the community;
- the number, physical design and precise location of the facilities;
- the extent to which women and men in the different community groups will participate with cash, materials and labour – including cooking and collecting water – during construction and maintenance and how the participation will be managed, monitored and controlled;
- monitoring of the quality of the work;
- local arrangements for maintenance and repair;
- the local financing system for sharing the investment and recurrent costs;
- the statutes and regulations needed for management of the water system and of water and land use to protect the quantity and quality of the water resource(s).

For the support team, most of the options and even the choices will be the same from project to project. That does not mean that they can be prejudged. The future sustainability of the new systems depends on the whole community sharing information and accepting responsibilities based on full involvement in planning decisions.

Example of a planning process

An effective planning process leads in stages from the demand of the community, to an initial service level assumption, a participatory base-line survey, an assessment of support capacity, and an analysis of results with the communities, and ultimately to a formal agreement on the water supply system chosen. The steps are:

Step 1 Demand from the communities for support in improving their water supply. This step may well have been preceded by promotion campaigns.

Step 2 Initial service level assumption, during which the support agency makes a preliminary overview of hydrological, population, technical and institutional aspects, including an inventory of theoretically potential water supply options, based on existing data and information.

Step 3 Participatory community baseline survey, during which a situation analysis is done with the communities, in order to assess the following points:

- Demand (of poor, middle class and better-off men/women)
- Consumption patterns and desired service level (men and women)
- Perception of benefits of an improved water supply system (men and women), including distance, comfort, time gains, health, income generating activities, status
- Culture, beliefs and habits of different user groups regarding water supply, sanitation and hygiene, including an assessment of experiences (good and bad) in past community projects
- Housing distribution (rich, intermediate and poor)
- Income and willingness to pay (rich, intermediate and poor)
- Present and future availability of water resource (yield and treatment needed, including seasonal variations)
- The importance of alternative and traditional water points
- Presence, capacity and representativeness of organised community groups, as indicated by men and women, and the poor
- Managerial and technical capacity within the community, by class and sex
- Managerial and technical capacity outside the community, including follow-up support
- Link with local/municipal/district authorities
- Potential technical options that match users' preference, ability to pay, management capacity, availability of spare parts and environmental conditions
- Capacity and willingness to pay for capital costs, including contributions in kind
- The need to integrate sanitation, hygiene and wastewater management

Step 4 Assessment of operations support capacity, during which an investigation is made among local, municipal and district public bodies, private formal/informal bodies or individuals, and NGOs in order to assess their present and potential future role and capacity in supporting the management, operation and maintenance of water supply schemes. The support agency will subsequently need to

- define a division of responsibilities together with the main stakeholders involved;
- promote the involvement of the private formal and informal sector not only for design and construction purposes, but also for maintenance, repairs, supply of spare parts, water quality monitoring (in conjunction with public health services), community mobilisation, etc.;
- plan for capacity building activities according to the needs that have emerged at all levels (community, public authorities, private sector).

Step 5 Analysis of results with the communities, during which the agency that implements the overall programme makes a preliminary overview of the results of the participatory baseline survey. This is then discussed with the communities, with an emphasis on all implications and long-term commitments of various technical options. Clarification should be made on the necessary organisational adjustments, as well as on the managerial and financial responsibilities, and tariff levels that are emerging. The sanitation and hygiene component is as essential as the water supply one, as it could jeopardise any effort in improving water supply schemes if not properly dealt with. Communities need enough time to consider the various options and implications, and may have to be organised in a formal group (association, cooperative, etc.). Women and the poor sometimes need help to organise separately and so gain equal influence.

Step 6 Formal agreement between the community, the local authorities and the support agency, once the community has made an informed choice on the desired water supply system, location and sites, and it has formally decided to contribute to the capital costs (in cash and/or in kind), as well as formed a community group. The agreements clearly define the responsibilities and rights of each party and spell out the sanctions and procedures in cases of violation.

Implementation

When each community has developed its own detailed plan, the same publicly accounted decision-making process at the programme level serves to decide which plans are financed through a loan, a grant or a combination of the two. Advisory services from the programme teams are made available to each community project and the project funds are transmitted in instalments to the special project bank account that each community has established.

Under this approach, procurement and construction are fully community implemented and controlled. This presumes that the capability to monitor the quality of construction and manage the contract is locally available or that training is provided. During this stage the local project committee also monitors and accounts for the fair implementation of contributions from individual householders. Care and monitoring ensure that both women and men from the different sections are trained and that women, as primary users, are not given only the low level physical work, but can exercise control over the quality of facilities.

The post-construction stage

Small community water supplies are often more difficult to keep running than to construct. The need for proper maintenance and management is generally recognised, but the actual maintenance work is frequently neglected. Building local management organisations and management capacities needs careful attention.

The local management organisation manages the water service according to locally agreed regulations and statutes that do not conflict with national water regulations. Users generally make payments that, as a rule, cover local operation and maintenance costs. Depending on the arrangements with the government, community financing may further cover primary repairs, repayment of the loan and the costs of keeping up local service coverage. Proper budgeting and financial management are crucial for a satisfactory service. Being a service for all, the local management organisation should also be accountable to all. That means reporting from the start and at clearly agreed intervals on its management, including its financial management, to the male and female household heads.

For small water supply systems, selected technicians and the management committee greatly benefit from technical and managerial experience and training during and after the construction. For larger and multi-village systems with a community-based management, the technicians and management staff are generally professionally trained and hired by the community water board. Other management options exist with roles for local authorities, such as municipalities and districts, and the private sector.

2.3 Programme organisational aspects

Small community water supplies developed under a large programme depend for their success on combining local skills and energy with proper programme support. This third section addresses some of the main organisational and staff requirements, and management models, for the kind of programmes that help communities to establish and run small, decentralised water services.

Function divisions

In large programmes in which many small community water supplies are decentrally planned, implemented and managed, success depends on the degree of completeness and clarity in the division of functions and on the standards to which those functions are carried out at each level. Table 2.4 gives the principal functions that may be carried out at the higher (national or state), middle (provincial and/or district) and lower levels. The national and local governments, with their supporting institutions, are one category of actors in the complex context of water supply; the community-based (management) organisations, NGOs and the private sector are other key actors, as, of course, are the men and women users.

If certain links in this chain are weak or missing, the overall chain is at risk. Decentralisation cannot, for example, be limited to the delegation of responsibilities to communities to manage their own water services. They also need authority and rights, such as a legal status and procedures for claiming liability in case of substandard

construction or mismanagement of funds. And they need capacity building and technical support that does not stop at the completion of a project.

Table 2.4 Functional divisions among different levels of government

Level	Functions
National/state	<ul style="list-style-type: none"> • Establishment of technical and administrative policies • Formulation of measurable objectives, strategies, guidelines and standards • Long-term programme planning • Legislation • Regulation • Management of national funds, and matching these to local contributions • Monitoring and evaluation • Accounting to Parliament, etc. • Adjustments to policy, strategies, etc. based on evaluation and public accountability • General financial control. • Interdepartmental coordination • Donor coordination • Adjustment of education curricula • Provisions for in-service training
Provincial/District	<ul style="list-style-type: none"> • Planning and execution of implementation programme(s) • Allocation of individual community projects • Resources support (financial and human) • Capacity building and training • Monitoring and evaluation of implementation and post-implementation results in relation to support approaches, with corrective action • Financial and quality control • Backstopping of established community services • Coordination for water resources management
Community	<ul style="list-style-type: none"> • Planning and implementation of local water projects • Process management • Service management (incl. O&M and financial management) • Monitoring and evaluation of services, with corrective action • Accounting for project implementation and service management to community male and female household heads • Coordination with other communities/water resources management

Management models

There is a worldwide trend to decentralise management responsibilities as well as to involve the private sector to a greater extent. This decentralisation trend has resulted in municipal councils and local governments being endowed with the responsibility of ensuring the provision of public services at local level. In some countries this has even become a constitutional prerogative. However, many local government organisations do not have sufficient capacities to assume this responsibility. Along with this trend, the great majority of national sector strategies, especially those designed for rural and peri-urban areas, include the principles of community participation and management of water supply services and gender and poverty sensitive approaches as a basic condition for sustainability. How can these trends (municipal management and community management with gender and social equity) work together without competing with one another? How can the public and private sector cooperate for greater efficiency with preservation of social justice?

In most cases municipal and local government bodies remain legally responsible, and they will delegate part of their responsibility to the communities within their borders in the case of community managed water supply systems. It is important that this is spelt out in a written contractual form, giving water committees a legal status and clearly defining the boundaries of responsibilities of each party. Unfortunately, in many countries, water committees do not have a proper legal status. There are several possibilities:

- The municipality/local government officially registers the water committee, and acknowledges the constituting act of the general assembly of the community
- The water committee operates under the legal mandate of a village development association
- The water committee is registered with the chamber of commerce as a non-profit making user association
- The water committee is registered with the chamber of commerce as an organisation with an economic interest as a cooperative, or private firm

The private sector generally intervenes as a provider of services, for a specific activity (repair, construction, etc.), or under a time-bound service contract or management contract. Remuneration corresponds to the service provided. In urban areas other contractual arrangements are made, under which the private contractor is remunerated via water fees from users.

The choice of a particular management model is influenced by issues such as: capacity of community organisations; complexity and type of technology; possibility of multi-village scheme; links with local authorities and other stakeholders; capacity of the private formal and informal sector; national policies, (on institutional arrangements at local level and decentralisation for instance).

The following management models can be applied for community managed water supply systems:

- **Tap or neighbourhood committee** responsible for operating and maintaining a specific water point
- **Water committee** responsible for all managerial, financial and technical activities of a scheme that covers a larger area than a neighbourhood, and possibly the whole community
- **Water committee contracting a private body.** The committee remains responsible for general management and control, but contracts a private body (an individual, a mechanic, a group of artisans, a firm) to operate and maintain the system
- **Multi-village water committee,** responsible for the management of water supply in several communities, where each community has also its association. This option is sought in the situation of a piped scheme serving several villages
- **Village association,** responsible for all development activities concerning the village, including overseeing water and sanitation

Sanitation and hygiene either fall under one of the above-mentioned bodies or are managed by a separate health or sanitation committee.

The committee or association may have the ownership of the assets but more important is that it is in control of the decision-making and management. For larger and more complicated piped water supply schemes, the actual management including the daily financial and technical functions is best done by a professional team consisting of a manager, accountant and technicians.

Supporting the process of change

Because of their limited size and experience in running water services, community-based management and governing boards of most small water supply systems need external support and advice in areas such as planning, implementation, management, accounting, audits, monitoring, training, special repairs, etc.

One possibility is to have special multidisciplinary government teams with expertise in all aspects of community projects. In a variation of this approach, technical and social units from different departments work together, or a government technical team cooperates with a social NGO. In such cases, teamwork is generally more difficult to achieve, but a team approach is vital for good results.

A further possibility is the involvement of multidisciplinary teams from local NGOs and consultancy groups on a contract basis. Finally, in regions in which small communities have capable local development organisations and technical expertise, it is possible to organise training programmes for these groups that include hands-on learning in and across local communities.

Forming an association of community-based water supply organisations can create a strong structure of mutual political, financial and technical support. Such associations act as bridges, facilitators and protectors of small water organisations in their relationships with the private sector and government departments.

In the spectrum of actors, an independent national and regional water regulator is required to protect the public interests. This function of regulation cannot be combined with the national and regional water departmental tasks.

Supporting communities to plan and implement their projects is quite different from planning and implementing projects oneself. In all models, the organisational arrangements will therefore be quite different from those of conventional engineering programmes. Selection criteria for staff who volunteer or are recruited to work in a support programme must include good communication skills and attitudes to work as a team, with local women and men as colleagues. The members of the team must want and be able to combine the specific knowledge, expertise and skills of local people with those of the team. Knowing and having used participatory methods and tools such as PRA (participatory rural appraisal) and SARAR (self-esteem, associative strength, resourcefulness, action planning, responsibility) are an advantage. All team members must be gender and poverty conscious, and recognise and be able to overcome or reduce inequalities between women and men and rich and poor. Teams must also have the opportunity to stay together for at least five years.

In participatory, demand-responsive and gender and poverty sensitive programmes, job descriptions of technical staff differ from those in conventional engineering programmes. Technical staff need to be aware of and take into account the socio-cultural and organisational implications of technical options. In the same way, social staff need to have a basic understanding and appreciation of the technical implications of community choices. Both types of staff often need joint training on community management aspects of the services, the use of participatory tools and techniques and the application of gender and poverty perspectives.

The *private sector* has a number of possible functions in new-style programmes. At the support level, a programme may decide to give contracts for support work with communities to technical-social teams from the private sector. It is important that these teams are chosen for their expertise and field experience and will work together for a prolonged period. A trial phase with indicators of performance (not just quantitative, but also qualitative) and the judgements of different community groups on the quality of the work further help to achieve good results. Similar procedures can help to secure good training for communities from private and semi-private sector agencies.

For technical work, the communities may decide to do their own procurement, use their own artisans and/or hire contractors. Typical help from the support programme will consist of guidance on required skills, materials and equipment, making and managing contracts, and monitoring of the quality of the work. Criteria of success are not construction rates, but the degree to which the services are sustained and used and the overall coverage levels within communities.

At the higher level, managers and other superiors need to appreciate, support and reward participatory and gender and poverty-positive work styles from social and technical field personnel. Staff performance evaluations and new contracts cannot simply be based on achieving physical and financial targets, but must take into account the quality of process work and the nature of longer-term results. Ideally they should also take into account the feedback from different community groups. It is the attitudes and behaviour of the higher management levels towards participation, gender and poverty programmes, which co-determine the overall atmosphere and results of small community water supply programmes.

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3 Small community water services in Central and Eastern European Countries – situation, challenges, needs and developments

Marek Roman

3 Small community water services in Central and Eastern European Countries – situation, challenges, needs and developments

3.1 General information

Ten countries are usually referred to as Central and Eastern European (CEE) countries: Bulgaria, Czech Republic, Estonia, Hungary, Latvia, Lithuania, Poland, Romania, Slovakia and Slovenia. They are linked by their geographical location, but also primarily by their political history after the Second World War, their current social and economical changes in the same direction and their willingness to become part of the European Union. In the years 1989-90, the CEE Countries regained their sovereignty and in the course of rebuilding a democratic political system began to change their economies to free market economies. At the same time, local governments were reinstated, and these began to take into their jurisdiction more and more matters of importance for local communities.

Regardless of common political, social and economic developments, the CEE countries differ from each other in many aspects. Differences exist in their demographic characteristics, the size of their water resources and the amount of water consumption. These characteristics are summarised in Tables 3.1, 3.2 and 3.3.

Table 3.1 Surface area, population and gross domestic product (GDP) of CEE countries

Country	Surface area km ²	Population		Population density per km ² (1998)	GDP in US \$ per capita per year
		Total million (1998)	% in rural areas		
Bulgaria	110,912	8257	31	74	1513 ¹⁾
Czech Republic	78,864	10295	32	131	4818 ²⁾
Estonia	45,200	1429	30	32	3360 ³⁾
Hungary	93,032	10,114	31	109	4569 ³⁾
Latvia	64,600	2449	30	38	2430 ³⁾
Lithuania	65,200	3702	35	57	2874 ¹⁾
Poland	312,685	38,666	38	124	4078 ²⁾
Romania	238,391	22,503	40	94	1515 ¹⁾
Slovakia	49,012	5388	33	110	3654 ¹⁾
Slovenia	20,253	1983	50	98	10,076 ¹⁾

Source: Central Statistics Office, Warsaw, 2001

1) According to the official exchange rate in 1999 (CEETAC, 2000)

2) in the year 2000

3) in the year 1997 (CEETAC, 2000)

Table 3.2 Mean annual water resources of the CEE countries

Country	Annual renewable water resources m ³ per capita per year		
	Internal	River flow from other countries	Total
Bulgaria	2410	600	3010
Czech Republic ¹⁾	600	50	650
Estonia	8050	3240	11,290
Hungary	570	10,360	10,930
Latvia	567	11,373	11,940
Lithuania	4140	2903	7043
Poland	1440	140	1580
Romania	1501	3752	5253
Slovakia	284	1246	1530
Slovenia	16,100	6600	22,700

Source: CEETAC, 2000

- 1) According to another source, the volume of water resources of the Czech Republic is higher: internal resources – 1457 m³/capita.year, river flow from other countries – 97 m³/capita.year, total – 1554 m³/capita.year (Central Statistical Office, Warsaw, 2001)

Table 3.3 Mean annual water use in CEE Countries

Country	Year of data	Total water used in m ³ per capita per year	
		Total	in towns ²⁾
Bulgaria	1997	1554	171.0
Czech Republic	1998	225	61.2
Estonia	1998	1000	43.0
Hungary	1995	657	85.4
Latvia	1997	91 ³⁾	34.4 ³⁾
Lithuania	1998	1179	33.0
Poland ¹⁾	2000	286	60.8
Romania	1996	461	92.2
Slovakia	1997	1058	380.8
Slovenia	1996	170	129.2

Source: CEETAC, 2000

- 1) According to data from the Statistical Yearbook (CEETAC, 2000)
 2) Own calculation based on data given in table 2 (CEETAC, 2000)
 3) Estimated for the largest cities.

The fraction of the population deemed to be rural is similar in eight of the ten CEE countries and lies within the range of 30-38%. In two countries, however, it is higher – in Romania it is 40% and in Slovenia 50% (table 3.1). The countries with the lowest water availability per inhabitant include the Czech Republic, Poland and Slovakia (table 3.2). However, when only the internal water resources are taken into account, i.e. the resources available within the territory of the given country, Hungary and Latvia also have a limited amount of water. It has to be noted that in these countries the amount of river water that flows in from other countries forms a large share of the total amount of water resources available. To a lesser extent this is also the case for Romania and Slovakia.

The water usage per inhabitant in CEE countries varies greatly (table 3.3). The index of total water usage in m³/person/year is high in Bulgaria, Estonia, Lithuania and Slovakia. This is a result of relatively high water consumption for energy and industrial needs. The index of water use in towns in seven CEE countries lies in the range of 33-90 m³/person.year. In three countries it is considerably higher:
in Bulgaria – 171 m³/person.year; in Slovakia – 380.8 m³/person.year;
and in Slovenia – 129.2 m³/person.year.

3.2 Condition of water services in CEE countries

The availability of water and wastewater services can generally be characterised by the percentage of the population served. In the CEE countries such data are only available for the urban population, or refer to the population in general without an urban-rural breakdown. Data are rarely available for populations of small towns and rural areas. Table 3.4 summarises the data for inhabitants using municipal or community water supply and sewage disposal systems and sewage treatment plants. The data for the rural areas show that populations of small municipalities are in a worse situation than town populations as far as access to water supply and sewage disposal services is concerned. This is presented in more detail by the data for Poland shown in table 3.5. Statistical data quoted in tables 3.4 and 3.5 also clearly indicate that in small municipalities and in rural areas sewage disposal systems are significantly less developed than water supply networks.

Table 3.4 Population using water supply networks and sewerage systems in CEE countries (major parameters – according to data 1997/98)

	% of population using:								
	Municipal water supply system			Municipal sewerage system			Wastewater treatment plants		
	Country wide	Urban	Rural	Country wide	Urban	Rural	Country wide	Urban	Rural
Bulgaria	85	n	n	49	70	2	36	n	n
Czech Republic ¹⁾	87.1	n	n	74.8	n	n	63.9	n	n
Estonia	77	n	n	60	80	45	50	n	n
Hungary	94	96	88	43	63	8	30	n	n
Latvia	93	n	<50	92	n	<50	88	n	n
Lithuania	70	n	n	60	n	n	36	n	n
Poland ²⁾	84.9	91.7	76.5	55.7	83	11.5	53	79.3	10.7
Romania	55	n	n	40	n	n	28	n	n
Slovakia	81	n	n	54	84	16	36	n	n
Slovenia	66	n	n	n	74	n	43	n	n

Source: CEETAC, 2000; Central Statistical Office, Warsaw, 2000 and 2002; Foundation for the Development of Polish Agriculture, 2001; Hugh, J. and Roman, M., 2000; Roman, M., Kloss-Trębaczkiewicz, H. and Osuch-Pajdzińska, E., 2001

- 1) According to data from 2000 (Foundation for the Development of Polish Agriculture, 2001)
- 2) According to data from 2000 (Central Statistical Office, Warsaw, 2001, 2002; Hughes, J. and Roman, M., 2000; Roman, M., Kloss-Trębaczkiewicz, H., and Osuch-Pajdzińska, E., 2001)
- n No information available.

Table 3.5 Population of Polish urban and rural areas using municipal water supply and sewerage systems and municipal wastewater treatment plants (data from 2000)

Type and size of settlements	% of total population of size group using:		
	Municipal water supply system	Municipal sewerage system	Wastewater treatment plants
Urban area			
Town size group (number of inhabitants)			
200 000 and more	96.8	92.0	75.5
100 000 ÷ 199 999	95.5	89.9	90.3
50 000 ÷ 99 999	92.6	85.4	82.8
20 000 ÷ 49 999	90.0	80.4	84.6

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Type and size of settlements	% of total population of size group using:		
	Municipal water supply system	Municipal sewerage system	Wastewater treatment plants
10 000 + 19 999	84.3	72.0	76.8
5 000 + 9 999	80.4	60.6	69.3
below 5 000	72.9	44.3	60.1
Town in total	91.7	83.0	79.3
Rural area			
Villages ¹⁾	76.5	11.5	10.7
Country in total ²⁾	84.9	55.4	53.0

Source: Central Statistical Office, 2001

- 1) According to other sources (Central Statistical Office, Warsaw, 2001; Hughes, J. and Roman, M., 2000; Roman, M., Kloss-Trębaczewicz, H., and Osuch-Pajdzińska, E., 2001)
- 2) Own estimation.

After the Second World War, in the years 1945-1990, the communal water supply and sewage disposal systems in CEE countries were state owned. Institutional arrangements for managing the services also had a state-owned character. The services were either provided by state-owned companies or by central budget institutions that depended on the state administration. In general, water supply and sewage disposal in towns and rural areas were managed centrally.

After 1990 water supply and sewage disposal systems became the property of municipalities and communities. As well as gaining ownership of the technical systems, they became responsible for water supply and sewage disposal in towns and rural areas. While implementing these tasks they began to change the institutional arrangements for supplying water and disposing of sewage and wastewater in their territories. Currently there are different institutional forms such as: budgetary enterprises; limited liability companies; joint stock companies; water law companies; entrusting the operational services to private companies based on legal agreements; and others. Table 3.6 shows examples of the different institutional arrangements for water services currently in operation in Poland. The figures indicate that commercial code companies prevail in the larger cities (with populations over 50,000), while municipal budgetary enterprises and units operate predominantly in small urban municipalities (<10,000 people) and rural locations.

Table 3.6 Incorporation forms of water/wastewater service providers in Poland as at December 1999

Locality by size of population	% of all providers in a particular type of locality					
	Budgetary enterprises and units	Limited liability company	Joint stock company	Unchanged incorporation ¹⁾	Other	Total
Urban	45.6	40.2	2.0	2.3	9.9	100.0
Over 50 000	8.0	71.5	8.0	9.1	3.4	100.0
20 000 – 50 000	22.6	68.4	2.3	3.8	4.9	100.0
10 000 – 20 000	38.4	51.2	1.8	1.8	6.8	100.0
5 000 – 10 000	52.0	33.0	1.7	1.1	12.2	100.0
under 5 000	71.2	12.0	0.0	0.4	16.4	100.0
Rural localities - total	57.8	9.9	1.1	0.9	30.3 ²⁾	100.0

Source: own studies

- 1) Municipal enterprises – former state-owned enterprises operating under the State Enterprises Law of 1981.
- 2) The majority of them have entrusted operational services to others on the basis of a legal agreement or the lease of technical objects to conduct the services.

A characteristic feature of the current situation in water supply and sewage disposal services in CEE countries is the maintenance of public ownership of communal water supply and sewage disposal networks. The communities do not want to sell these assets and participation of the private sector in that field is very limited. Currently the main opening for private sector enterprise is found in the management of the operations, while the ownership of the system remains within the hands of the communities (or: remains communal property).

3.3 Challenges, needs and developments

The overall vision given by the Central and Eastern Europe Technical Advisory Committee (CEETAC) of the Global Water Partnership is that “in two to three decades there will be sufficient, safe, clean and healthy water and people living in stable societies in the CEE region” (CEETAC 2000). It is an ambitious vision, and to implement it requires complex activities within the scope of overall water resources management: protection from contamination; improvement of water supply and sanitation; improvement of protection against floods and droughts; promotion of international

cooperation in solving common aspects of water supply and sharing water resources; development and implementation of environmental protection policies; and obtaining sufficient funding.

In the field of water supply there is a need to develop the water supply networks, especially in small municipalities and rural areas. It is necessary to raise the access to domestic water supply services in those areas to the much higher levels of large towns. This is not only important from a perspective of social equity, but also to ensure an adequate level of public health. Currently, shallow wells, which are present in nearly all rural households, are the alternative water source, but the sanitary quality of their water is often unsure.

Extending the scope of water supply networks has to go hand-in-hand with improvements in the quality of water supplied to consumers. That requires further modernisation of existing water treatment plants and construction of new ones, to meet raised standards. Water supplies in small municipalities and rural areas in CEE countries mainly use water from groundwater sources. The quality of this water needs to be improved and its sources protected against pollution.

Development of improved water supply systems must be accompanied by an adequate development of sewage disposal and water treatment systems. If this does not happen, the paradoxical situation will emerge that the development of safer water supply systems in rural areas and small municipalities creates new threats to the environment and public health. Lack of funding is the biggest obstacle to improving water supply, sewage disposal and water treatment systems in CEE countries. The countries have low per capita GDPs and huge restructuring needs in many other fields as well as water services.

An important task currently being undertaken by the CEE countries is to ensure that in future the water tariffs make it possible to recover the costs of the water services, as laid down in Article 9 of the European Community water policy directive 2000/60/EC. This is an immensely difficult challenge. It will have to be implemented gradually over several years, because it will cause a significant increase in water prices for rural populations with very limited financial capabilities.

In meeting the considerable challenges they face, CEE countries do have some advantages over developing countries in the South facing similar problems. There is, for example, no shortage of qualified professionals able to design, build and maintain water supply infrastructure. The CEE countries all have university graduates with master's degrees and higher engineering degrees in water supply and wastewater engineering. Training facilities are also available for technical personnel and operators. There is an open market now for foreign technologies, particularly from Western Europe, and CEE home companies also produce appropriate modern solutions.

3.4 Relevance of this publication to CEE countries

Clearly many of the approaches described in this book are of great relevance in solving the problems of water supply services for small communities in the CEE region. It is true that, for CEE countries, the approaches and technologies are traditionally similar to the solutions used in Western Europe so that the range of technologies has to be expanded at the upper end. Knowledge for doing so exists already, as the experience of Western Europe countries is well known in CEE countries, and German and English textbooks for water supply engineering are widely available. The countries also publish their own high-standard textbooks. In Poland, for example, a new edition of a very detailed textbook (474 pages) devoted to water supply and sewage disposal in the rural areas was published in 1998.

Groundwater is the main source of water supply in small communities in CEE countries, so the information that is especially important is that on the removal of iron, manganese, ammonia and other pollutants found in underground waters. In CEE countries pressure filters are often used, with prior aeration carried out also in pressure aerators. Where possible, water supply networks in small communities are based on closed systems in which the water is obtained from boreholes and supplied by pressurised networks with equipment for the removal of iron in the distribution network or air vessels. Mostly, water tower reservoirs and air vessels are employed. The air vessels control the pumps and maintain the pressure in the water supply network in a given range.

The information on drinking water quality standards given is valuable and useful for CEE countries. In those countries, drinking water quality regulations are based on WHO guidelines. As all the countries intend to join the European Community, new regulations on drinking water are being implemented in line with the new European Union directive on drinking water (Council Directive 98/83/EC).

The data on water consumption in chapter 4.2 give examples of water consumption in various countries of Southern Asia and Australia. For comparison purposes, table 3.7 presents German data, which are very similar for water consumption to those in Central European countries.

Table 3.7 Average water consumption in households using network water in Germany
(Year 2000 data)

Purpose	Water consumption litres per capita per day
Drinking and food preparation	5.0
Bathing and personal washing	46.0
Toilet	35.0
Dish washing	8.0
Laundry	16.0
Other	8.0
Total	118.0

Source: Globus, statistische Angaben: Bundesverband der Deutschen Gas und Wasserwirtschaft (KA – Wasserwirtschaft, Abwasser, Abfall 2002 (49) No. 2 p.152).

Currently in CEE countries, households are reducing their per capita tap water consumption. This can be illustrated by the data from Poland presented in table 3.8. The decrease in water consumption by the households is due to water metering, the introduction of prices based on the actual costs of the services, the availability of better quality technology for house water installations and a reduction in water wastage.

Table 3.8 Changes in average water consumption in households in Polish towns during the period 1995-2000

Year	Water consumption litres per capita per day
1995	168.9
1996	157.5
1997	149.5
1998	141.6
1999	136.2
2000	129.8

Source: Central Statistical Office, Warsaw.

Generally speaking, the decrease in water consumption should be seen as beneficial. However, the unit water consumption (in lcd) should not be excessively reduced, as this would be in contradiction with maintaining an adequate quality of life, the full realisation of people's needs for good hygiene and a basic condition for good public health.

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4 Water quality and quantity

Dave Nozaic

4 Water quality and quantity

4.1 Introduction

The availability of a clean and safe water supply is essential for public health. Water is used for a number of needs but in subsistence economies the most important use is for domestic purposes and for small productive uses such as garden irrigation. Water for domestic use needs to be safe and free from pathogens and other harmful substances. In this chapter we will attempt to provide guidelines for the quality and quantity of water required for different levels of service.

The report Global Water Supply and Sanitation Assessment 2000, prepared jointly by the World Health Organization and UNICEF, indicates that in the year 2000 nearly 1100 million people still remained without access to improved sources of water and that about 2400 million had no access to any form of improved sanitation facilities.

4.2 Water quantity

The amount of water a person needs each day is determined by a number of factors. Depending on climate and workload, the human body needs from 3-10 litres of water per day for normal functioning. Part of this water may be derived from food.

Factors influencing the amount of water used are

- cultural habits
- socio-economic status and standard of living
- hygiene awareness
- productive uses
- the charges for water
- the quality of the water as experienced by the users¹

The water demand of the community depends on climate, standard of living, availability and method of distribution.

Table 4.1 shows the per capita usage of water for domestic purposes (via a house connection) for countries in Southern Asia.

Table 4.2 sets out typical domestic water usage data for different types of water supply systems.

1 The criteria of users and engineers are not always the same. Understanding the perspectives of the users so as to adjust designs where possible and tailor any advocacy to their concepts is more effective than simply requiring that users 'be educated' to adopt outsiders' criteria.

Table 4.1 Water usage for domestic purposes in Southern Asia

Purpose	Quantity (lcd)*
Drinking	5
Cooking	3
Sanitary purposes	18
Bathing	20
Washing utilities	15
Clothes washing	20
Total (excluding water loss and wastage)	81

*lcd: litres per capita per day
Source: Design Manual for Water Supply and Treatment, India, 1991

Table 4.2 Typical domestic water usage data for different types of water supply systems

Type of water supply	(litres/capita/day)	Range (litres/capita/day)
Typical water consumption		
Communal water point (e.g. village well, public standpipe)		
At considerable distance (> 1000 m)	7	5 – 10
At medium distance (500 – 1000 m)	12	10 – 15
Village well Walking distance < 250 m	20	15 – 25
Communal standpipe Walking distance < 250 m	30	20 – 50
Yard connection Tap placed in house-yard	40	20 – 80
House connection		
Single tap	50	30 – 60
Multiple tap	150	70 – 250

Individual house connections provide a higher level of service than a tap placed in the house yard while the yard connection generally is preferred over a communal village point such as a well or a standpipe. In the selection of the type of service, finance is usually an important factor and the choice also depends on the location and size of the community, the geographical conditions and the available water source.

To obtain a first estimate of the water demand of a community it is often easier to establish the number of households from an aerial survey rather than a door-to-door census. Domestic water use can then be computed using an average size of family. Studies of existing small community water supply systems in the same area can provide useful water usage data. If possible field measurements should be taken. An alternative approach is to use a social map that community women and men prepare, in which they draw the compounds and may indicate what service level the women and men household heads have agreed upon. When sizing the water supply system (including the water treatment plant) and the type of supply, account has to be taken of other uses too. The presence in the area of a school, hospital, hostel, etc. needs to be taken into account and an adequate supply provided for these purposes as well. Table 4.3 includes figures for some typical uses in developing countries. These estimates should be used for preliminary planning and design only and should be considered as a rough guide. There is no substitute for local knowledge and this will be needed for the final design criteria for any given region or smaller administrative area.

Table 4.3 Various water requirements in developing countries

Category	Typical water use
Schools	
Day schools	15-30 l/day per pupil
Boarding schools	90-140 l/day per pupil
Hospitals (with laundry facilities)	220-300 l/day per bed
Hostels	80-120 l/day per resident
Restaurants	65-90 l/day per seat
Cinema houses, concert halls	10-15 l/day per seat
Offices	25-40 l/day per person
Railway and bus stations	15-20 l/day per user
Livestock	
Cattle	25-45 l/day per head
Horses and mules	20-35 l/day per head
Sheep	15-25 l/day per head
Pigs	10-15 l/day per head
Poultry	
Chicken	15-25 l/day per 100

(Adapted from IRC, 1981)

Table 4.4 shows water consumption patterns in South Africa in 1980 and indicates the relative amounts of water used in various sectors of the economy. This provides a guideline as to the general level of domestic consumption, relative to the total, in a partially industrialised economy. In a predominantly agricultural country, irrigation consumes a substantial proportion of the available water supply. Obviously trade-offs have to be made, where the needs of a community have to be set against the needs from organised agriculture. This could be a major consideration in many countries where agricultural use of water is a significant portion of the total. In general priority is given to domestic water supply, but productive use is important as it creates the economic basis for payment for water. In areas with a scarcity of good quality water, alternative sources of water could be used for different water uses. In some countries rainwater is preferred as drinking water. Lower quality water can be used for non-drinking purposes or productive use if no negative effects are expected.

Table 4.4 Water consumption patterns in South Africa

Demand sector	(million m ³ /a)	(%)
Direct use		
Municipal and domestic	1 516	9.3
Industrial	1 031	6.3
Mining	446	2.9
Power generation	282	1.7
Irrigation	8 504	52.2
Stock-water	262	1.6
Nature conservation	178	1.1
Indirect use		
Forestry runoff reduction	1 284	7.9
Ecological use, estuaries and lakes	2 768	17.0
Total	16 291	100.0

Based on 1980 data

It is often very difficult to estimate the future water demand of a community accurately and the engineer has to apply considerable design judgement in making the analysis. In countries with positive economic development there is a progressive increase in water consumption for economic and domestic purposes. Economic activities demand more water, and people with an increasing living standard seek a higher level of water service and more water per capita. Design criteria have to take these trends into account so as not to find the water system capacity inadequate a few years after construction. Expansion of water supply systems will generally be far less economic than construction

of larger systems from the beginning. On the other hand, over-sizing due to overestimation of population and consumption makes schemes uneconomical and leads to high unit costs for those people already connected to the supply. The right decision requires careful analysis.

If present water sources are inadequate to meet the estimated demand then new sources have to be considered that may imply a much higher water production cost. Water demand management is an important way to forestall the use of expensive new sources. It needs to be applied to all the different forms of consumption. Awareness raising and price elasticity are key elements for effective water demand management, and, as with all pricing issues, the poor need to be protected against measures which might reduce their consumption to dangerously low levels.

It is worth reiterating that local knowledge of population growth rates, affordability criteria and the type of schemes envisaged will lead to better estimates of water quantity. This is essential if errors of underestimation or overestimation are to be avoided. Also consultation with the local communities on the demands of the different user groups and arriving at a mutually agreed decision are vital for the success of a scheme. Building on experiences from neighbouring or similar areas is helpful. If water conservation is needed because of limited capacities of available sources, good communication with the consumers on demand management is crucial.

The water usage figures, given in table 4.3, include about 20% allowance for water losses and wastage. In many developing regions there are considerable losses from leaks and, sometimes, unauthorised withdrawal of water from the distribution system. Losses in these cases can be higher than 30% and sometimes even approaching 50-60% of the normal production.

As a tentative estimate, the water supply for a more or less centralised community settlement would need to have a capacity of 0.3 litres per second per thousand people when the water is mainly distributed by means of public standpipes (about 25 litres per capita per day) and about 1.5 litres per second or more per thousand people when yard and house connections predominate (approximately 130 litres per capita per day). Schemes should provide for future connections, population growth and a rise in the standard of living, which is not uncommon when the infrastructure is improved. The design would typically be based on the daily water demand estimated for the end of the specified design period (typically 10 years for water intake works and treatment plant, and 25 years for the distribution network system). The estimation would use a design population (present population multiplied by growth factor such as in table 4.5) multiplied by an estimate of initial and final per capita consumption.

Table 4.5 shows population growth factors for various yearly growth rates for periods of 10, 15, 20 and 25 years.

Table 4.5 Population growth factors

Design period (years)	Yearly growth rate			
	2%	3%	4%	5%
10	1.22	1.34	1.48	1.63
15	1.35	1.56	1.80	2.08
20	1.49	1.81	2.19	2.65
25	1.64	2.09	2.67	3.39

A community water supply system should also be able to cater for maximum hourly or peak water demands during the day. It would be normal to provide a storage reservoir of 24 or even 48 hours capacity based on average demand and to feed this at constant rate from the treatment system either by gravity or by pumps. Peak demands can then be catered for by drawing down the reservoir.

4.3 Water quality

Water for domestic use is required to be both wholesome and safe. For aesthetic reasons it is desirable that the water be clear (low turbidity) and that it be free from taste, odour and colour. For health reasons a number of chemical characteristics need to be considered. Water should have calcium and magnesium contents within an optimum range; it should not have excessive sodium; the sulphate and nitrate content needs to be limited; and various toxic metals should not be present in quantities greater than very low limits which are set in the many standards which apply to water treatment.

Of prime importance, however, is the bacteriological quality of water. Water for domestic use should be free from pathogenic organisms viz. bacteria and viruses which may cause disease. The importance of water as vehicle for the potential spread of diseases is the main concern in water quality control. It has often been stated by world health authorities that disinfection of water alone has saved more lives than the whole of the medical industry.

Many studies have been conducted on the effects of water quality on human and animal health, the acceptable limits for physical characteristics and the useful life of water distribution systems. Based on these studies, the World Health Organization (WHO) has published guidelines to help countries to set quality standards with which domestic water supplies should comply. These will often be considered as long-term

goals rather than rigid standards, as the importance of providing a suitable quantity of water of a reasonable standard for drinking and hygiene is a top priority.

The basic requirements for drinking water are that it should be free from pathogenic (disease causing) organisms; clear (low turbidity, little colour), not saline (salty); free from offensive taste or smell; free from compounds that may have had adverse effects on human health (harmful in the short or long term); free from chemicals that may cause corrosion or encrustation.

The most practical way to express quality requirements for drinking water is in tables which give the "highest desirable" and "maximum permissible" levels for each parameter. To repeat, they are guideline figures and may take time to achieve. Interim improvements that would give people enough water for their basic hygiene needs should not be inhibited by rigid adherence to "ideal" standards.

Guidelines for water quality are divided into four main aspects: bacteriological, physical, chemical, and radiological. Regular sampling and monitoring verifies that sources and delivered water meet specified criteria, but "sanitary surveys" are a vital part of ensuring that all the potential contamination risks have been taken into account. Annex 1 describes the key features of sanitary surveys and laboratory analysis in safeguarding public supplies.

Bacteriological parameters

As far as public health is concerned the most important aspect of drinking water quality is the bacteriological quality, i.e. the presence of bacteria and viruses. It is not practical to test water for all organisms that it might possibly contain. Instead the water is examined for a specific type of bacteria that originates in large numbers in human and animal excreta, and its presence is indicative of faecal contamination. Faecal or thermotolerant coliform bacteria including the *Escherichia Coli* (*E.coli*) are members of a wider group of bacteria known as coliforms, of which many types are present in soil. Thermotolerant coliforms multiply in the human gut and can be detected relatively easily by culturing. Consequently, they are the most widely used bacteriological indicator. Another, somewhat less commonly used, indicator organism is faecal streptococci. When these bacteria are found in water it indicates fairly fresh faecal contamination and the need for disinfection. Faecal bacteria are likely to be found in almost all small community water supply systems. It would be excessive to condemn all supplies that contain some contamination, especially when alternatives may be even more polluted. Rather, bacteriological testing of the water determines the level of pollution in each available source and the optimum source can then be chosen. In designing a water treatment plant one would include chlorination or other means of disinfection to remove the danger of waterborne disease.

Brief descriptions of water sampling for bacteriological analysis and the different methods for testing (membrane filtration and "most probable" number using the multiple-tube method) are given in annex 1.

According to the WHO Drinking Water Quality Guidelines, water intended for drinking should not contain any detectable E-coli or thermotolerant coliform bacteria in any 100 ml sample. However, it is recognised that faecal contamination is widespread in the majority of rural water supplies in developing countries. The sensible way forward is for the responsible national agency to set medium-term targets for the progressive improvement of water supplies.

Table 4.6 is an example of a risk classification based on thermotolerant coliforms for rural water supplies that gives the motivation and urgency to improve the quality of water meant for drinking.

Table 4.6 Example of risk classification for thermotolerant (or faecal) coliforms or E. coli for rural water supplies

Count per 100 ml	Risk category
0	In conformity with WHO guidelines
1-10	Low risk
11-100	Intermediate risk
101-1000	High risk
>1000	Very high risk

Table 4.7 shows the South African Bureau of Standards limits for these microbiological constituents. In addition to this, limits exist in some guidelines for faecal streptococci, Salmonella, Staphylococcus aureus and Clostridium perfringens. Many guidelines only limit the first three organisms in Table 4.7, however, and these are usually adequate for assessment purposes.

Table 4.7 Microbiological requirements

1	2	3	4	5
Determinants	Units	Allowable compliance contributions*		
		95% min.	4% max.	1% max.
		Upper limits		
Heterotrophic plate count	count/ml	100	1 000	10 000
Total coliform	count/100 ml	Not detected	10	100
Faecal coliform	count/100 ml	Not detected	1	10
Somatic coliphages	count/10 ml	Not detected	1	10
Enteric viruses	count/100 l	Not detected	1	10
Protozoan parasites (Giardia/Cryptosporidium)	count/100 l	Not detected	1	10

* The allowable compliance contribution shall be at least 95% to the limits indicated in column 3, with a maximum of 4% and 1% respectively, to the limits indicated in columns 4 and 5. The objective of disinfection should, nevertheless, be to attain 100% compliance to the limits indicated in column 3.

Physical and organoleptic parameters

The physical and organoleptic (aesthetic) parameters most commonly regarded as significant are turbidity, colour, odour, pH and taste.

High turbidity and/or colour impart an aesthetically displeasing appearance to water. The turbidity in surface waters results from the presence of colloidal material such as clay and silt, plankton and micro-organisms. Apart from a displeasing appearance, the particles which create turbidity also provide adsorption sites for chemicals that may be harmful or cause undesirable taste and odour. They also allow adsorption of biological organisms, which shields the organisms and interferes with disinfection.

Colour in drinking water is due to natural organics such as humic substances or dissolved inorganic compounds of iron or manganese. Highly coloured industrial waste can also impart colours in water. Apart from the aesthetic appearance, organic colour when disinfected with chlorine will produce chlorinated organics, some of which are carcinogenic. The presence of iron in the water can lead to staining of clothes when washed in such waters. Odour and taste problems in water are due mainly to the presence of organic substances. Objectionable taste or odour will cause consumers to reject a bacteriologically safe supply in favour of a less satisfactory source. It follows that water needs to be free of offensive taste and odour, usually determined by testing panels and judged according to the dilution needed to reach undetectable levels.

The pH of a particular water is a measure of its acidity/alkalinity, which in turn means its aggressivity as far as metals and concrete in pipes and treatment works are concerned. Low pH can increase corrosion, while too high a value can lead to calcium carbonate deposition and encrustation of pipe networks. Because the pH level can have an impact on the efficiency of water treatment processes such as coagulation, pH adjustment is a common practice in water treatment works.

Table 4.8 shows a comparison of recommended standards for physical parameters.

Table 4.8 Standards for physical quality

Characteristic	U.S.	Canadian	EEC	WHO	Australian guideline
Turbidity (NTU)	1-5	5*	0-4	<5	5
Colour (TCU)	15	15	20 mg Pt-Co/L	15	15
Odour (TON)	3	-	0-2 dilution numbers at 12°C	-	No objectionable odour
pH	6.5-8.5	6.5-8.5	6.5-8.5**	<8.0	6.5-8.5
Taste	-	-	2 dilution numbers at 12°C	-	-

(Adapted from Sayre, 1988; NHMRC-AWRC, 1987; and WHO, 1993)

Note: - = Data not available

* Decker and Long (1992) give the Canadian standard for turbidity as 5 NTU aesthetic objective and 1 NTU as the maximum acceptable value.

** pH value 6.2-8.5 according to EC directive 80/778 (AQUA, 1992).

Chemical

The chemical requirements of a water supply are conveniently divided into macro-determinants and micro determinants. Macro-determinants such as ammonia, calcium, chloride, fluoride, magnesium, nitrates, potassium, sodium, sulphate and zinc can be present in comparatively larger quantities. Excessive levels have a harmful effect on health, but in many cases limited quantities are necessary for the maintenance of living organisms and low concentrations are therefore desirable in the water supply. The micro determinants consist mainly of the toxic or heavy metals and are limited to very low concentrations.

In the WHO Guidelines for Drinking Water Quality, the organic constituents are divided in two categories: (i) chemicals of health significance in drinking water; and (ii) substances in drinking water that may give rise to complaints from users.

Table 4.9 lists the physical and chemical parameters set out in the South African Bureau of Standard Guidelines. Class 0 water is regarded as ideal; class 1 is acceptable and class 2 permitted when no other supply is available (but this should only be used for limited periods).

Table 4.9 Physical, organoleptic and chemical requirements

1	2	3	4	5	
Determinants	Units	Upper limit and ranges			
		Class 0 (Ideal)	Class I (Acceptable)	Class II (Max. allowable)	WHO Guideline values
Physical and organoleptic requirements					
Colour	mg/l Pt	15	20	50	15
Conductivity at 25°	mS/m	70	150	370	-
Dissolved solids	mg/l	450	1 000	2 400	1 000
Odour	TON	1	5	10	-
pH value at 25°C	pH units	6.0-9.0	5.0-9.5	4.0-10.0	<8.0
Taste	FTN	1	5	10	-
Turbidity	NTU	0.1	1	10	≤ 5
Chemical requirements:					
Macro-determinants					
Ammonia as N	mg/l	0.2	1.0	2.0	1.5
Calcium as Ca	mg/l	80	150	300	-
Chloride as Cl	mg/l	100	200	600	250
Fluoride as F	mg/l	0.7	1.0	1.5	1.5
Magnesium as Mg	mg/l	30	70	100	-
Nitrite and nitrate as N	mg/l	6.0	10.0	20.0	≤ 1
Potassium as K	mg/l	25	50	100	-
Sodium as Na	mg/l	100	200	400	200
Sulphate as SO ₄	mg/l	200	400	600	250
Zinc as Zn	mg/l	3.0	5.0	10.0	3
Chemical requirements:					
Micro-determinants					
Aluminium as Al	µg/l	150	300	500	200
Antimony as Sb	µg/l	5	10	50	5
Arsenic as As	µg/l	10	50	200	10
Cadmium as Cd	µg/l	3	5	20	3
Chromium as Cr	µg/l	50	100	500	50

>

1	2	3	4	5	
Determinants	Units	Upper limit and ranges			
		Class 0 (Ideal)	Class I (Acceptable)	Class II (Max. allowable)	WHO Guideline values
Cobalt as Co	µg/l	250	500	1 000	-
Copper as Cu	µg/l	500	1 000	2 000	2 000
Cyanide (free) as CN	µg/l	70	70	70	70
Cyanide (recoverable) as CN	µg/l	70	200	300	-
Iron as Fe	µg/l	10	200	2 000	-
Lead as Pb	µg/l	10	50	100	10
Manganese as Mn	µg/l	50	100	1 000	100
Mercury as Hg	µg/l	1	2	5	1
Nickel as Ni	µg/l	50	150	350	20
Selenium as Se	µg/l	10	20	50	10
Vanadium as V	µg/l	100	200	500	-

Note: The limits for iron are based on aesthetic aspects.

Organic constituents

Tables 4.10 and 4.11 give limits for various organic substances that are undesirable in the water. Some are classified as known carcinogens, others are not classifiable on the basis of inadequate evidence.

Table 4.10 Classified categories of carcinogens

Category	A	B1	B2	C	D
Volatile organic compounds	Benzene vinyl chloride		<ul style="list-style-type: none"> • Trichloroethylene • Pentachlorophenol • Chloroform • Bromodichloro- methane • Bromoform • Carbon tetrachloride • 2,4,6- Trichlorophenol • Acrylamide • 1,2- Dichloroethane 	<ul style="list-style-type: none"> • 1-Dichloroethane • 1,1,2-Tetrachloro- ethylene • 1,1-Dichloro- ethylene • Dibromochloro- methane • P-Dichlorobenzene 	<ul style="list-style-type: none"> • Toluene • Xylene • Mono- chlorobenzene • Ethylbenzene • Hexachloro- cyclopentadiene • 1,1,1- Trichloroethane • 1,2- Dichlorobenzene

Category	A	B1	B2	C	D
Pesticides			<ul style="list-style-type: none"> • Heptachlor epoxide • Hexachloroene • Toxaphene • Aldrin • Malathion 	<ul style="list-style-type: none"> • Endrin • Methoxychlor • Ethoxychlor 	

Source: Chung, 1993

Note: A. Human carcinogen, based on sufficient evidence from epidemiological studies.

B. Probable human carcinogen, based on at least limited evidence or carcinogenicity to humans (B1), or usually a combination of sufficient evidence in animals and inadequate data in humans (B2).

C. Possible human carcinogen, based on limited or equivocal evidence of carcinogenicity in animals in the absence of human data.

D. Not classifiable, based on inadequate evidence of carcinogenicity from animal data.

Table 4.11 Groups of organic compounds of potential health significance that could be present in water sources, and treatment and distribution systems

Chlorinated alkanes	Chloramines
Chlorinated ethenes	Chlorophenols
Aromatic hydrocarbons	Trihalomethanes
Polynuclear aromatic hydrocarbons (PAH)	Chlorinated acetic acids
Chlorinated benzenes	Halogenated acetonitriles
Pesticides	

(Adapted from WHO, 1993)

Most of the organic compounds are from industrial sources and quite often from the petroleum industry. In rural areas such compounds are unlikely to be present but in cases where there are chemical or petrochemical industries in the vicinity it would be safe to err on the side of caution and perhaps to analyse for volatile organics.

Radiological constituents

It is uncommon for radioactivity to be a cause for concern but this aspect is considered in a number of standards. The World Health Organization (WHO) limit the following:

Gross alpha activity	0.1	Bq/l
Gross beta activity	1.0	Bq/l

If radioactivity is suspected the water should be tested.

Sampling

In assessing an existing or potential water supply, every effort should be made to take suitable samples of the water and to have them analysed as fully as possible. Distance and lack of facilities can make bacteriological analyses difficult to carry out, but modern technology test kits are now available which can determine many of the common microbiological indicators in the field. With portable refrigeration and incubation available reasonably reliable counts can usually be obtained.

Analytical methods are available from Standard Methods for the Examination of Water and Wastewater published by APHA, AWWA and the WEF, but many of the field test kits come with their own methods. See also annex 1.

For chemical analyses it is not as important that the sample be fresh, although care should be taken to preserve samples as far as possible and these can then be analysed in a suitable laboratory for macro- and micro-determinants. At least one full analysis should be carried out if at all possible prior to the drinking water source selection. Furthermore, during the course of the project additional analyses of certain key determinants should be undertaken. Obviously the more knowledge that is available regarding the supply, the better.

Annex 1 discusses the role of sanitary surveys and laboratory analysis in maintaining quality standards.

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Web sites

Water quality: http://www.who.int/water_sanitation_health/index.html

Discussion groups

GARNET (Global Applied Research Network) Water Quality Monitoring, see <http://info.lut.ac.uk/departments/cv/wedc/garnet/tncwq.html>

5 Integrated water resources management

Patrick Moriarty

5 Integrated water resources management

5.1 The concept of IWRM

To have a water supply system you first need a sustainable water source of sufficient quantity and quality. The inexorable increase in global population and use of water for economic purposes puts ever greater pressure on the world's water resources. Integrated water resources management (IWRM) recognises that there will always be more potential demand for water than there is supply, and that to deal with this there is a need for negotiation and compromise. While there is a desire to seek so-called "win-win" scenarios, these are often difficult to put into practice in real terms. The reality is that hard decisions need to be made, and that in allocating water among competing users there will be winners and losers. The aim must therefore be to develop institutions that have the respect and legitimacy to enforce their decisions even when these are unpopular. The core message for the drinking water and sanitation sector, voiced in Vision 21 (WSSCC, 2000), is that domestic water must come first. No other activity has equal importance, and there can be no "trading" of domestic water entitlements. Water use should be seen as a pyramid, with domestic use representing the smallest, but most important amount at the pyramid's apex.

IWRM is perhaps one of the most misunderstood and yet at the same time most important concepts in the water sector. It is misunderstood because most people within the important sub-sectors (irrigation, industry, water and sanitation) think of it as "someone else's business". Yet IWRM was developed as a philosophical structure to bring together the different sectors. It is important because in the many parts of the world where water quantity and quality are threatened by over-use and poor use it is an approach that offers a way out of the "zero-sum game" of competition between sectors.

The question that programme and project managers involved in community water supply should ask themselves is not "is IWRM relevant to me?" but rather "how can I apply the philosophy of IWRM to my project?". This chapter sets out to answer this question, through a brief introduction to the concepts and history of the philosophy of IWRM, and then by outlining a set of practical questions that can be posed at the project level to identify the necessary actions to approach best IWRM practice.

5.2 The complications that make IWRM necessary

Water is not a resource in the way that, say, land and minerals are. Both of these represent finite quantities of a given resource located at some fixed place in space and time. Water, by contrast, is a flux, constantly recycled through the hydrological cycle and biological processes in humans, animals and plants. Add man-made recycling and reuse,

and the balancing of “supply” and “demand” becomes a far from simple issue. A litre of water flowing from the source of the River Thames in England may be “consumed” as many as nine times before it finally flows out into the North Sea to rejoin the hydrological cycle of evaporation and precipitation.

Water’s unique properties as a finite but renewable resource lie at the heart of many of the problems associated with its management. Essentially, it is the rate at which water is used in a particular place in comparison with the rate at which it is replenished that determines whether there is a scarcity or a surplus. In global terms and on a cumulative basis, there is no shortage of fresh water. The world’s freshwater crisis is one of water resource distribution in space and time. The 400 cubic kilometres of water that falls on New Zealand each year (117 million litres for each inhabitant) cannot easily be used to supplement Egypt’s one cubic kilometre per year (20 litres per inhabitant). Nor is the floodwater of the Ganges that inundates huge areas of Bangladesh in the monsoon season readily harnessable to meet that nation’s irrigation and drinking water demands in the dry season.

Man’s activities, too, prevent us from making optimum use of the water resources that are available. Water use in one location can affect the ability of others to use it in another location hundreds or thousands of kilometres away. Most of the world’s international rivers have transboundary issues around high withdrawals or excessive pollution by upstream riparian states that in turn reduce the quantity or quality of water available to downstream countries. Though it may be less visible, the same problem applies to groundwater resources where “selfish” pumping rates by one aquifer user can dry up the wells of another, or injudicious fertiliser use can launch a nitrates time bomb to contaminate supplies for the next generation.

These large-scale examples of the misuse of water resources generally make headlines and there has been increasing political and scientific attention given to improved IWRM as a means of conflict resolution. This international “peacemaking” has deep resonance for the development of water resources at community level, for a key truth about water resources development and management is that in a large majority of cases it is an essentially local issue. While much of the literature on resource management concentrates on basin level and transboundary issues, the reality is that worldwide most day-to-day conflict over water resources occurs at the level of small rivers or localised aquifers (for an excellent overview of water resource problems at a variety of scales experienced in South Asia see Moench et al. (2002).

At community level, integrated management of local water resources is intuitive. Water has to be found for domestic purposes, for horticulture and for small-scale agriculture. So individual householders recognise the need to conserve and protect available

supplies. What they may not realise so readily is the responsibility also to avoid degrading the river or groundwater so as to protect the resource for downstream communities. That is a matter for both education and regulation. It remains true that it is the enlightened self-interest of informed water users that provides the best basis for catchment protection, water conservation and demand management.

That is why the modern concept of IWRM puts so much emphasis on bottom-up development and management of water, and on management at the most appropriate level. Yes, it is essential to have a proper framework in place for management at the river basin level, but this must rest upon a solid foundation of local good practice.

5.3 A brief history of IWRM

Based on the emerging concept of a water crisis, IWRM was developed as a philosophical framework to look at the way in which water resources are developed and managed, and importantly, to keep in sight the vital links between water resources and land management. As such, IWRM is a relatively new concept. It was first discussed globally at the UN Water Conference at Mar del Plata, Argentina, in 1977, but not really put on the international agenda in any concrete form until the early 1990s. A number of significant meetings were held during this period including the 1990 New Delhi Global Consultation, the 1991 Nordic Freshwater Initiative in Copenhagen, the 1992 Dublin International Conference on Water and Environment (ICWE), the 1992 UN Conference on Environment and Development (The Earth Summit) in Rio de Janeiro (UNDP, 1992), the 1994 Ministerial Conference in Noordwijk (VROM, 1994) and the 1994 OECD/DAC meeting in Paris (OECD, 1994). These meetings challenged existing sector-oriented management practices for water resources as being unsustainable from an economic and environmental perspective, and set out a number of principles and recommendations for integrated water resources management. In an attempt to provide guidelines for the implementation of Chapter 18 of Agenda 21 (the action programme of the Rio de Janeiro Conference), the Noordwijk Ministerial Conference summarised key issues in IWRM on which international agreement has been reached, and gave an overview of the main IWRM principles for the DWSS sector. The meeting concluded that "access to adequate water and sanitation is a basic need, and the long-term objective in the DWSS sector therefore continues to be 'safe drinking water supply and sanitation for all'".

The interest in water resources continues to grow and it is now a key issue on the international agenda. The UN General Assembly Special Session (UNGASS) in 1997 called for urgent action in the field of fresh water. Furthermore the UN Commission on Sustainable Development in April 1998 made recommendations on comprehensive strategic approaches to freshwater management. The 2002 Johannesburg World Summit on Sustainable Development Implementation Plan includes the welcome

commitment to halve the number of people without access to safe water and sanitation services by 2015. It also commits countries to “Develop integrated water resources management and water efficiency plans by 2005”. (United Nations 2002). The challenge remains to transfer this high level political commitment into local action.

The objective of integrated water resources development and management is to ensure optimal and sustainable use of water resources for economic and social development, while protecting and improving the ecological value of the environment.

At the heart of IWRM lie the four so-called Dublin principles, first set out in the 1992 Dublin meeting, and then ratified through inclusion in Chapter 18 of Agenda 21 at Rio. The four principles are reproduced in their complete form in box 5.1. Today, a wide range of international and national agencies support these principles.

Box 5.1 The Dublin Principles for Integrated Water resources management

Principle No. 1:

Fresh water is a finite and vulnerable resource, essential to sustain life, development and the environment

Since water sustains life, effective management of water resources demands a holistic approach, linking social and economic development with protection of natural ecosystems. Effective management links land and water uses across the whole of a catchment area or ground water aquifer.

Principle No. 2:

Water development and management should be based on a participatory approach, involving users, planners and policy-makers at all levels

The participatory approach involves raising awareness of the importance of water among policy-makers and the general public. It means that decisions are taken at the lowest appropriate level, with full public consultation and involvement of users in the planning and implementation of water projects.

Principle No. 3:

Women play a central part in the provision, management and safeguarding of water

This pivotal role of women as providers and users of water and guardians of the living environment has seldom been reflected in institutional arrangements for the development and management of water resources. Acceptance and implementation of this principle requires positive policies to address women's specific needs and to equip and empower women to participate at all levels in water resources programmes, including decision-making and implementation, in ways defined by them.



Principle No. 4:

Water has an economic value in all its competing uses and should be recognised as an economic good

Within this principle, it is vital to recognise first the basic right of all human beings to have access to clean water and sanitation at an affordable price. Past failure to recognise the economic value of water has led to wasteful and environmentally damaging uses of the resource. Managing water as an economic good is an important way of achieving efficient and equitable use, and of encouraging conservation and protection of water resources.

(WMO, 1992)

5.4 IWRM – a framework for multiple systems, actors, sectors and scales

It is clear from its inclusion in the Agenda 21 and the Johannesburg Implementation Plan that IWRM is part of a wider effort to manage the world's resources in more sustainable manner. Because of this, IWRM differs from earlier engineering approaches to water management in that it attempts to work within a framework of the whole hydrological cycle, to see how water interacts with other elements of the ecology and how it is linked to land management. As such, IWRM includes all the different phases of the water flux, which are often referred to under different colours: blue water for surface and ground water resources; green water for the soil moisture used by plants; white water for atmospheric moisture; and brown and grey water for the effluents coming from sanitation and urban run-off.

IWRM implies looking at the whole "system", social, political, and physical (hydrological); at integrating the needs of all users upstream, downstream; and at balancing these needs in a way that satisfies the needs of the greatest number possible in a sustainable way while maintaining a healthy environment.

The IWRM philosophy aims at involving all actors: individuals, companies, organisations, and governments; in all sectors: irrigation, domestic water supply, industry; at all scales: local, national, and transitional. This is not to say that it implies that all stakeholders should be involved in all decisions, rather that each problem is solved at the level and by the stakeholders most appropriate and able to do so. Neither does IWRM suggest that all water issues should involve unwieldy bodies on which representatives from each sector sit together to oversee each others' work. The question of scale, and the differentiation of roles and responsibilities according to scale are crucial to a proper understanding of IWRM.

While IWRM is now well-accepted at an international level, and while an increasing number of countries are adopting or adapting new water laws to address its basic principles, most concrete activity has been decidedly top-down. What is more, there has been little real movement towards better integration of IWRM philosophy into the day-to-day management of projects within water sub-sectors. Although IWRM touches upon many issues that are crucial for the adequate planning, implementation and management of drinking water supply projects, there is little evidence of IWRM principles being applied within the DWSS sector (Visscher et al., 1999). The next section therefore discusses practical approaches to the application of the IWRM philosophy in the drinking water and sanitation sector, and in community water supply in particular.

5.5 IWRM in community water supply

Before looking at how to implement IWRM for community-based water and sanitation projects, it is worth briefly considering in more detail the reasons for doing so. Developing community schemes is complicated enough, so why make it more so by adding an element of resource management? Many professionals within water sub-sectors think that IWRM has nothing to do with them. They think it is for high level "resource-managers", that it concerns only transnational or river basin level issues, or, most dangerously, that because of the small amounts of water involved (at least relative to the total water balance and water use from other sectors) that there is no need for community managed water projects to bother about IWRM.

However, there are compelling reasons for involvement, largely rooted in the growing water crisis and in existing and potential conflict over water. In this light, the approach to IWRM taken in Vision 21 is clear. The water supply and sanitation sector as a whole has a vital interest in involvement in IWRM to safeguard the right of people to access domestic water in the face of growing competition from other uses and sectors. While clear and compelling, this approach is also somewhat self-centred and does not really address the wider philosophy of IWRM in its more positive sense. It ignores the fact that the other uses of water often form vital parts of the livelihoods of the people whose rights are being protected. A broader interpretation is possible based on the holistic philosophy of IWRM and the interest in protecting the rights of the poor. This interpretation seeks to ensure the right of access of communities to a fair share of water resources for all their livelihood needs, domestic and productive, and seeks to make the use of this share as efficient, effective and sustainable as possible.

Domestic water use is often a relatively minor part of the total water demand, seldom exceeding 10% (although importantly this can often represent a far higher percentage of available water). In comparison, irrigation's share of worldwide freshwater resources is often placed at 70%. The simple, but wrong, conclusion drawn from this is that that

ensuring domestic water availability is not much of a problem. This is a serious and dangerous fallacy. The crucial point is that the 10% requirement for domestic water is the amount that must be safeguarded at all times and in the worst conditions.

Box 5.2 Acute water crisis in Kurnool

KURNOOL: Acute drinking water scarcity in this town has reached dangerous proportions with a man dying on Thursday evening in a stampede that began when a water tanker arrived.

The incident occurred in Joharapuram area on Thursday evening falling under the Kurnool Municipal Corporation Limits when a crowd of about 100 persons tried to be the first to get to the tanker. In the stampede that followed Mahboob Basha fell down and was crushed to death by others who trampled over the fallen man to collect water.

The situation in Kurnool is turning from bad to worse with more than 3 lakh people here receiving just 10 lakh gallons a day against the normal supply of 40 lakh gallons a day. The shortage has become very severe in the last six days due to the falling water levels in the Tungabhadra, the only drinking water source for the town that is built on the right bank of the river. This is the first time in 14 years that we are facing such an acute water shortage, a KMC official said adding that poor rains in the district as well as in the catchment areas of the river has led to it drying up.

It was only when dry river bed was noticed that the KMC began making efforts to draw water from other sources. District collector G Sai Prasad began monitoring the situation after asking municipal engineer Rammohan Reddy to go on leave holding him responsible for not taking steps to pre-empt the crisis.

Though the collector later ordered the irrigation department to divert about 60 cusecs of water from the Gajuladinne project to the Tungabhadra river, only about 10 cusecs reached the town as farmers en route diverted the water to their fields. It is with this limited water that the KMC is supplying 10 lakh gallons a day to the town. This is the second time that water was diverted to the town from Gajuladinne project, the first time being in 1988.

Meanwhile, Kurnool MLA TG Venkatesh is supplying drinking water to the most affected areas through 20 water tankers with the KMC chipping in with three tankers. However, these are not adequate to meet the demand with the result that private operators are making a killing supplying water at high cost.

Times of India – Friday, 13th July, 2001

It is therefore essential to have some concept of a drinking water reserve: that is an amount of water held in storage (artificial or natural) for domestic use, which can be relied on completely, and which is not allowed to be used for non-domestic purposes. The concept of a reserve is present in South African water law, where it is stipulated that a reserve for domestic and environmental needs should be calculated at a catchment level (RSA 1998). Though this is conceptually simple, in reality both estimating and then safeguarding a reserve is extremely complex. Nevertheless, introducing the concept to both communities and project personnel is essential. In addition, monitoring frameworks that can help them to estimate when the limits of the reserve are being approached are a crucial part of implementing community level IWRM. The second implication of the reserve concept is that once it has been established, non-domestic uses of water may need to be seriously reduced, and the mechanisms to allow this to happen developed and put in place. Much of this will be beyond the scope of the individual WSS project, but it is important to realise that where a reserve is not safeguarded, the sustainability of a system cannot be taken for granted.

A crucial point about IWRM is that it entails different activities at different scales. IWRM at a national level is mainly about developing policy and legislation, as well as encouraging better cooperation between national stakeholders. Catchment or river basin IWRM is about making allocation decisions, preferably with full involvement of catchment stakeholders. IWRM at local levels, while also often involving allocation decisions, is more about the optimal use of resources at the local level, and about ensuring that local activities do not adversely affect the quality or quantity of water available to downstream users.

From the point of view of drinking water supply projects or programmes, the ideal situation is found in countries where IWRM is fully mainstreamed in national strategy and an integrated set of activities can take place at all three scales. There will be supportive national legislation, devolved (preferably catchment level) decision-making, and active demand management at the local levels. There will be a monitoring framework that ensures that the information needed to make decisions at each level is available and there will be mechanisms to ensure that stakeholders' voices, including those of poor communities, are heard in decision-making about water resource allocation.

Where such a framework exists, the IWRM roles of the community water supply project will be relatively clear, as will the rights and responsibilities of the community level actors. The main work of the project agency in such a case will be to ensure that it complies with existing legislation, and that the community receives training and awareness-raising about their rights and responsibilities. These could include issues such as source registration and protection, quality monitoring, issuing of an abstraction permit, etc. It will also be important to check to what extent the framework recognises

the rights to water of non-formal users: poor people, and women who use wetlands, catchment areas and groundwater for water and other products vital to their livelihoods.

However, such situations are unfortunately still rare. Few countries have effective water resources management legislation, and fewer still the means to implement it. What can the water and sanitation supply project do in such situations to observe good IWRM practice? To start with, it must be recognised that it is not necessary to have such an ideal framework in place for a community water supply project to begin to implement IWRM. Ironically it is often in cases where there is no overall IWRM framework that such work is most essential. The next section deals with the application of the IWRM philosophy at a community level in the absence of clear national policy or implementation guidelines. It sets out a framework of key principles, adapted from the Dublin principles, which can act as a guide to ensuring that local level IWRM is being implemented.

5.6 A practical approach to implementing IWRM within water supply and sanitation projects and programmes

The six key principles set out in box 5.3 below and discussed in more detail in the remains of this chapter can, where properly internalised within a project or programme monitoring structure, ensure that IWRM principles are met. The list is specifically focused on the water and sanitation sector, and at the project and system level. It aims to help staff of WSS organisations to identify how they can best incorporate the relevant IWRM principles into their own projects and systems, rather than worrying about more abstract policy or regulatory matters that are outside their sphere of influence.

Box 5.3 Six IWRM principles for the drinking water supply sector

1. Catchment management and source protection are essential to ensuring sustainability of supply
2. Water use efficiency and demand management must be addressed to minimise the need for new source development
3. Multiple uses of water by women and men in differing social strata should be acknowledged and encouraged
4. All stakeholders should be involved in decision making, but particular emphasis should be put on the active participation of users
5. Gender and equity issues must be addressed throughout the project cycle
6. Water provision should be priced so as to discourage wasteful use, while ensuring the right to access of a necessary minimum for all for domestic and small scale productive uses by the whole family

(Adapted from Moriarty et al., 2000)

Principles one and two deal with resource availability, three deals with how to maximise the impact of water in people's livelihoods, four and five deal with the 'social' aspects of implementing IWRM, and six deals with the economic principles.

The starting point for any water and sanitation project must be the availability of a suitable resource: that is a sufficient quantity of water available at all times and in all conditions, and of suitable quality. This is covered by the application of principles one and two. What these principles mean in practical terms of project implementation is that, before embarking on a drinking water supply scheme, a thorough assessment must be carried out of the current and likely future domestic water needs of the community (including for small-scale productive uses), the realistic possibilities of minimising these through demand management, and the long-term sustainable availability of water from all sources to which the community has access. Only where a match between estimates of total supply and demand is made can a sustainable water service be embarked upon. Where the figures do not match up, alternative sources will need to be investigated and identified. Two linked approaches to assessing available water resources are given in box 5.4.

Box 5.4 Water budgets and participatory resource inventories

Two of the practical tools that will help most in assuring the sustainability of a water resource are a water budget and a water resource inventory.

A **water budget** should try to estimate all inflows and outflows to the source or catchment, both natural and man-made, and ensure that these balance. A water budget should be done by a professional hydrologist. However, a water resource audit carried out by a multi-disciplinary team can contribute considerably to the accuracy and usefulness of a water budget.

A **water resource audit**, carried out in collaboration with the community, can help in preparing the water budget by identifying all current sources and uses of water within the area, and by identifying trends in use and thus possible future demands. To be of any use, all water use within a community must be assessed, even if the project is interested purely in domestic supply. A resource inventory can also serve as a useful starting point to working with communities on wider IWRM issues (for an example of the practical implementation of a water resource audit see KAWAD, 2001).

A crucial issue in carrying out the water budget and resource inventory is whether to work at the source, or catchment scale: whether to deal with one or multiple sources. Only in the rare instance where the source being used is demonstrably not affected by other water uses from different sources can it be considered in isolation. Wherever



there are known or suspected linkages between different sources, all uses and all sources must be included. It must be emphasised that to carry out an effective water budget, expert hydrological knowledge must be sought in addition to local information gathered through resource audits. Ideally the two should be fully integrated with a hydrologist taking part in the resource audits.

Where the main source is shared on a large scale, as is the case where domestic water is drawn from a river, it may be difficult to get any but the broad outlines of a water budget in the absence of a larger scale IWRM framework. However, even here, local knowledge and past records can still provide indicators as to potential problems such as seasonal or occasional scarcity. In many cases, though, the resource being used will be essentially local, and here the main criteria will be climate, physical conditions and different uses.

A warning

It is essential that a proper hydrological assessment, based where possible on long-term data, is used to back up direct evidence from the field such as resource audits and direct measures such as pumping tests. In particular traditional WSS approaches such as pumping tests of new boreholes provide a "snapshot", which can be highly misleading, as they give no idea of the likely behaviour of the water resource before or after the test is made.

Principle one – Catchment management and source protection are essential to ensuring sustainability of supply

The starting point from an IWRM point of view must be to assure the sustainability of the source(s) used. The following questions cover the core aspects, and should be addressed in the resource inventory and water budget.

- Are sources adequate for both current and projected domestic use (including small-scale productive uses)?
- Are they reliable throughout the year and over the years? Many sources suffer from seasonal or periodic failure.
- Is ownership of the source controlled by the community? Are there competing claims to it by other users or uses?

If, given the relevant data, the answer to any of the questions is no, then action will need to be taken to ensure that an adequate supply can be assured. These actions can be either demand reduction (see next principle) or supply augmentation. Activities that can augment supply include watershed management and groundwater recharge (see next chapter).

Involvement of the community in watershed management activities, particularly where these are small scale, can be highly beneficial. In many Latin American countries a key part of any community management scheme is the buying of the source and its surrounding catchment area. Often this is accompanied by catchment improvement activities such as planting trees and regulating or banning farming activities within the catchment area.

Ensuring the sustainability of the source may be something for a community to do on its own, or may alternatively be something that is too large scale for much meaningful action. There is also an intermediate level, which occurs particularly with multi-community schemes, where a common source is shared and where it is necessary to work in collaboration with other communities. Here a framework for deciding on allocation issues may need to be developed between communities.

A crucial adjunct to source protection is effective monitoring, to know how much water is available and when, for example, some uses must be stopped to ensure that the reserve isn't eaten into. Setting up a resource-monitoring framework with communities is a core activity.

Principle two – Water use efficiency and demand management must be addressed to minimise the need for new source development

In addition to the physical activities to augment quantity and preserve quality mentioned above, there may be a need for local level management between competing resources, and effective demand management of all water uses. This can take the form of simple rationing, or banning of certain activities, but is more likely to be successful where efforts are made to involve communities (and important sub-groups within them) in the monitoring and regulation of their own water use.

Demand management in traditional water and sanitation thinking tends to be linked mainly to improved efficiencies in transmission systems. Poorly maintained systems can have losses of 50% or higher, and dealing with these clearly provides an important area for creating 'extra' supply. However, given the reality of competing uses at a local level, it is also necessary, in collaboration with the community, to identify all the uses to which water is put, and all the potential actions that can limit excess water consumption within these various uses. While simple rationing may be sufficient, aspects such as improved irrigation efficiency may be more productive in terms of overall impact of water on people's livelihoods. Such activities fall well beyond the scope of individual community water projects and call for close collaboration with other specialists and institutions.

An important element of demand management within this holistic framework is water reuse, and multiple uses of water. So, for instance, wastewater can be used for irrigation, or small dams can be used for irrigation, fish production and domestic water supply.

Both principles two and three have important implications in terms of strengthening community capacity and developing new institutions (or strengthening existing ones). How this can be done is dealt with briefly under principle four.

Principle three – Multiple uses of water should be acknowledged and encouraged

Another aspect of dealing with IWRM in water supply projects is the need to reassess the role of the community water supply project itself. Traditionally the role has been to provide water for drinking and associated domestic needs – washing, food preparation, etc. However, it is increasingly recognised that this approach itself is guilty of failing to be truly participatory, by imposing an alien representation of water on communities.

Communities seldom differentiate between water for domestic and non-domestic uses. Water, particularly in areas where it is scarce, represents a vital productive resource. It is needed to carry out many activities essential to livelihoods other than drinking. A recent FAO publication estimates that, even in urban areas, 40% of land is dedicated to food production, much of it irrigated using “domestic water” (FAO, 2000). Market gardening, domestic livestock, laundry services and micro-industry are all potential productive uses that will typically compete for some of the resource (Lovell, 2000). Addressing this issue means designing supplies with multiple uses in mind, and also identifying potential alternative sources (rainwater harvesting, wastewater reuse, etc.).

A livelihoods-based approach to developing water supplies is more likely to achieve a measurable impact on people's well-being and is also more in tune with IWRM principles. Such an approach to water supply takes as its starting point an analysis of the different ways in which water is used within people's livelihoods. Once these have been understood a water supply system is designed that meets as many of these needs as possible. The system may actually be a mixture of systems and sources all managed within a single framework. It may also be a single system with multiple uses for the water it provides. In some ways this latter case is the most challenging to the drinking water and sanitation sector, as in the past projects have often actively sought to discourage people from using their community water supply systems for non-domestic purposes.

It is important to note that a livelihoods-based approach does not mean that the WSS sector should become involved directly in clearly non-domestic issues such as irrigation. What it does mean is that it should try to address all household level activities, including productive ones.

The livelihoods-based approach not only greatly increases the impact of water on people's well-being but also, by directly linking water supply to economic activity, greatly increases willingness and ability to pay for water and maintain water supply systems. (James et al., 2001; Waughray, 1998)

Principle four – All stakeholders should be involved in decision-making, but particular emphasis should be put on the active participation of users

Principles four and five lie at the heart of the social dimension of IWRM. They cover the issues of both how water is controlled and in whose interests. Principle four starts from the Dublin principle of subsidiarity in decision-making. The focus in this case, however, is on decision making at the community level, and hence on the involvement of the key stakeholder groups at this level, the community, or more specifically the users of the WSS service. The statement emphasises an involvement in, rather than control over decision-making. This reflects the fact that IWRM decision-making is complex and involves multiple stakeholders at multiple scales. To suggest that one group of stakeholders should 'control' the process is both unrealistic and undesirable.

Where a fully developed national IWRM framework exists, the main activity for a water supply project will be in ensuring representation of the water committee on water management bodies at local and regional/catchment levels. One of the main activities in such a system will be the negotiation of a water right, to ensure adequate supply for the planned system. This means in particular ensuring that the voices of women and the poorest are heard and that they have the opportunity to make known their requirement for an equitable share of available water resources. Internationally, the paradigm of catchment management is rapidly gaining acceptance as the model for devolved management of water resources, and where such a system exists (such as in South Africa, Zimbabwe and a growing number of other states) it is clear that domestic water supply initiatives should seek ways to be actively involved. However, as already explained, their principle focus and main area for activity must continue to be in ensuring fair allocation at the local level.

Where there is no national framework, the problem is more complex, as the project may need to develop new institutions to deal with the management of water. This will be a complicated process with the need to involve all user groups within a community. The process may involve setting up water resources management committees, or integrating domestic water supply concerns into existing bodies, such as the watershed management committees of India. Those on the committee will need support and training in monitoring and decision-making about issues such as allocation of water rights, demand management, and source protection, as well as in using monitoring to ensure that a domestic reserve is maintained and protected.

Care must also be taken to ensure that the decision-making role of the community is tailored to what can realistically be undertaken at their level. There is no point in making a village water committee responsible for river basin level allocation decisions. Where a shared surface water source is being used, such as a river, a village level water committee will be responsible for allocation and management of the water right within the village, and should also be encouraged to join with other village committees to ensure representation in catchment-level bodies.

Principle five – Gender and equity issues must be addressed throughout the project cycle

If principle four represents the *how*, then principle five deals with the *why*, or more specifically the *for whom*, of better linking between community water supply and IWRM. The principle represents the change in emphasis and broadening of interest from the original Dublin declaration, with its focus on the role of women, to a wider gender focus that seeks to identify and cater for the different needs of men and women, and in particular of poor men and women. It emphasises that burdens and benefits are shared equally between men and women, necessitating particular efforts to enable women to claim their right in decision-making. Men and women use water for different activities – for example women are almost universally responsible for domestic water, but a much more mixed picture exists in irrigation. For example it is often also women who do vegetable gardening, but once irrigation becomes involved and the cash-raising potential of marketing vegetables becomes greater, men can become more involved. In general women and men have different as well as common interests in water and therefore both need to be included in water-based decision-making.

The issue of socio-economic equity in allocation of water resources is of central concern to practitioners in community water projects. Whether the poor use water exclusively for drinking, or for a range of small-scale productive activities, they tend to lack the political strength to make their voice heard in official water allocation bodies. Where such bodies exist, advocating for a sufficient and fair “water right” within such bodies can be an important aspect of a community water supply programme.

Principle six – Water provision should be priced so as to discourage wasteful use, while ensuring the right to access of a necessary minimum for all

The sixth and last principle deals with the economic aspects of IWRM. It is concerned with the pricing of water to ensure waste minimisation and a most effective use of limited resources. It should not be confused with the current emphasis in community water supply on cost recovery. Clearly money paid for water can be used for cost recovery and for operation and maintenance, but the underlying principles are very different. In addition, given the relatively price inelastic nature of demand for domestic water, and the widely held belief that access to domestic water is a right, there are good

reasons for not trying to price domestic water at a level that assures anything other than basic system maintenance.

While a number of existing economic tools for domestic water services, such as stepped tariffs, may seem initially promising also for IWRM purposes, they should be used with care. Stepped tariffs for instance have the problem that where supply is met in part by water vending, the high tariffs charged to vendors will be passed on to their (largely poor) customers. A second, more general argument against trying to use user charges to minimise waste is that on their own they do nothing to encourage service providers to improve system efficiency.

Given these concerns, and the previously mentioned fact that domestic use is typically small, it is questionable whether in the domestic water sector pricing should be used as an economic measure to limit domestic water consumption. Where the sector can play an important role is in ensuring that other, non-domestic uses such as water used for irrigation are charged at a realistic rate. Equally, where private sector providers are involved, charging of high bulk purchase prices for water rights, while capping volumetric charges to individual households, may provide a powerful incentive to maintain systems adequately. The same thinking can be applied to charging a community on two levels, firstly as a community for its water right, and secondly as individuals for actual water used. However, this is a complex issue that will not be gone into in more detail here.

5.7 Conclusions

While an effort has been made to suggest some practical implications of applying the six principles within a community water supply programme or project, the reality of both community water supply and IWRM is that blueprints cannot work. Each community (or at least district or region) must be dealt with in a manner tailored to its own social, economic, and physical reality.

The principles can be used as the basis for an assessment of an individual project's success in achieving best IWRM practice. The principles presented here were developed from a project carried out by IRC and partners in six countries in Africa, Asia, and Latin America (Visscher et al., 1999). In that experience, a slightly different set of principles provided a framework to which were added leading questions and key indicators relevant to country and local specificities. The questions and indicators were established and agreed upon by all stakeholders in a participatory manner as part of the IWRM learning process.

It was the complete list of questions and indicators that then became the monitoring or benchmarking tool, through which the projects started to take account of IWRM principles. The European Commissions "Strategic Approach" which adopts a similar approach, provides fully worked out lists of questions and indicators for all stages of project and programme development for, among others, domestic and municipal water supplies (EC 1998).

A final note on the development of an IWRM approach within community water supply projects is on the need to work more effectively with other sectors. At the scale of the community water supply project, the other sector most likely to be involved is the irrigation sector, although increasingly IWRM also involves industry and mining. Breaking down sectoral barriers isn't easy, particularly at the local level, where line management can often make people slow to make decisions on their own initiative. Knowing what other water development activities are planned for an area is the minimum that is necessary in assuring the sustainability of the resource. Active collaboration with the *water for food* sector can lead to greatly improved impact on livelihoods.

Water resources management, whether governed by a well-developed national framework or implemented internally within a project, is an essential part of any community water supply scheme. Without ensuring the sustainability of the resource, it is meaningless to talk about the sustainability of the scheme. By starting at the level of the individual project, and using the framework of six key issues set out in this chapter, an approach to IWRM is achieved that is practical and implementable.

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6 Artificial recharge

*By Sinha Ray with a contribution on
sand dams by Milu Muyanga*

6 Artificial recharge

6.1 Introduction

One of the growing concerns facing scientists and engineers in development and management of groundwater resources, is to manage this depleting resource efficiently. The key to a successful groundwater management policy is a thorough understanding of groundwater recharge and discharge processes. Under suitable conditions it is possible to supplement the natural recharge of an aquifer and so add to its safe yield. This is called artificial recharge. Strictly speaking artificial recharge is the process by which the groundwater reservoir is augmented at a rate exceeding natural replenishment. Any man-made scheme or facility with the objective of adding water to an aquifer may be considered to be an artificial recharge system.

Sustainability of sources has become one of the major issues of the rural drinking water supply sector. In this endeavour, the role of the government sector is being shifted from actual implementing authority to that of a facilitator. Since rainwater harvesting and artificial recharge can play a major role in achieving sustainability of drinking water sources, local communities need to be encouraged to take them up on a large scale. Chapter 7 refers to various kinds of rainwater harvesting structures through the ages which have proved to be very useful to society in many different parts of the world. Inputs are needed from governments and NGOs to establish the conditions for communities to take the necessary action.

In the context of small-scale artificial recharge projects, community participation is essential in at least the following areas:

At the planning stage:

During this time, the basic parameters are explained to male and female community representatives, so that they understand the options available and can weigh the advantages and disadvantages of each option. They must also decide between individual household facilities and community facilities. For further details see chapters 1 and 2.

At the implementation stage:

Community women and men can take charge of the material transportation to the site, and to the extent possible be involved in construction training, actual execution and quality control. This will ensure the use of knowledge of both groups, enhancement of local skills with equal chances for both in case of a gender equity approach, and a shared sense of ownership.

Operation and maintenance:

Routine operation and maintenance must be planned for and carried out by men and women in the communities themselves. Weighing and comparing the different tasks helps to achieve equity and to decide whether and where compensation may be needed. An equitable balance in work and benefits is one of the conditions for sustainable systems.

Sharing of “new” water resources:

This is crucial, since it is an area that has the potential for serious conflict. Understanding and agreeing on norms for abstraction and discipline by each and every member of the community of users is absolutely essential. This is possible only if the community is involved in developing the local rules and control mechanisms and accounting for their application.

Evaluation and modification of design:

When all different groups in the community (or in large communities their representatives) have been involved in the above stages, this last step should be smooth and spontaneous. Once the systems are in place, each group should be given the opportunity to reflect critically and improve upon the design so that learning and development continue. Involving both women and men from the various socio-economic sections in the evaluation and making sure (e.g. by using gender and class sensitive participatory tools and techniques) that all can share their views equally taps a wide range of knowledge and experiences.

6.2 Methods of artificial recharge

There are many reasons why water is deliberately placed into storage in groundwater reservoirs. A large number of artificial recharge schemes are designed to conserve water for future use. Other projects recharge water for objectives like control of saltwater intrusion, filtration of water, control of subsidence, disposal of wastes and secondary recovery of crude oil from oil fields.

Artificial recharge methods can be classified in two broad groups: (a) direct methods and (b) indirect methods.

Direct methods are subdivided into surface spreading techniques and sub-surface techniques. The most widely practised methods employ different techniques for increasing the contact area and residence time of surface water in the soil, so that a maximum amount of water can infiltrate and augment the groundwater storage. In *surface spreading* techniques, the various methods available are flooding, ditch and furrow surface irrigation, stream modification and finally, the most accepted one and suitable for small community water supplies, run-off conservation structures or rainwater harvesting.

In *subsurface techniques* injection wells and gravity head recharge wells are more common.

Indirect methods of artificial recharge adopt the technique of induced recharge by means of pumping wells, collector wells and infiltration galleries, aquifer modifications and groundwater conservation structures. They require highly skilled manpower and other resources. The different methods of artificial recharge are presented in tabular form in figure 6.1.

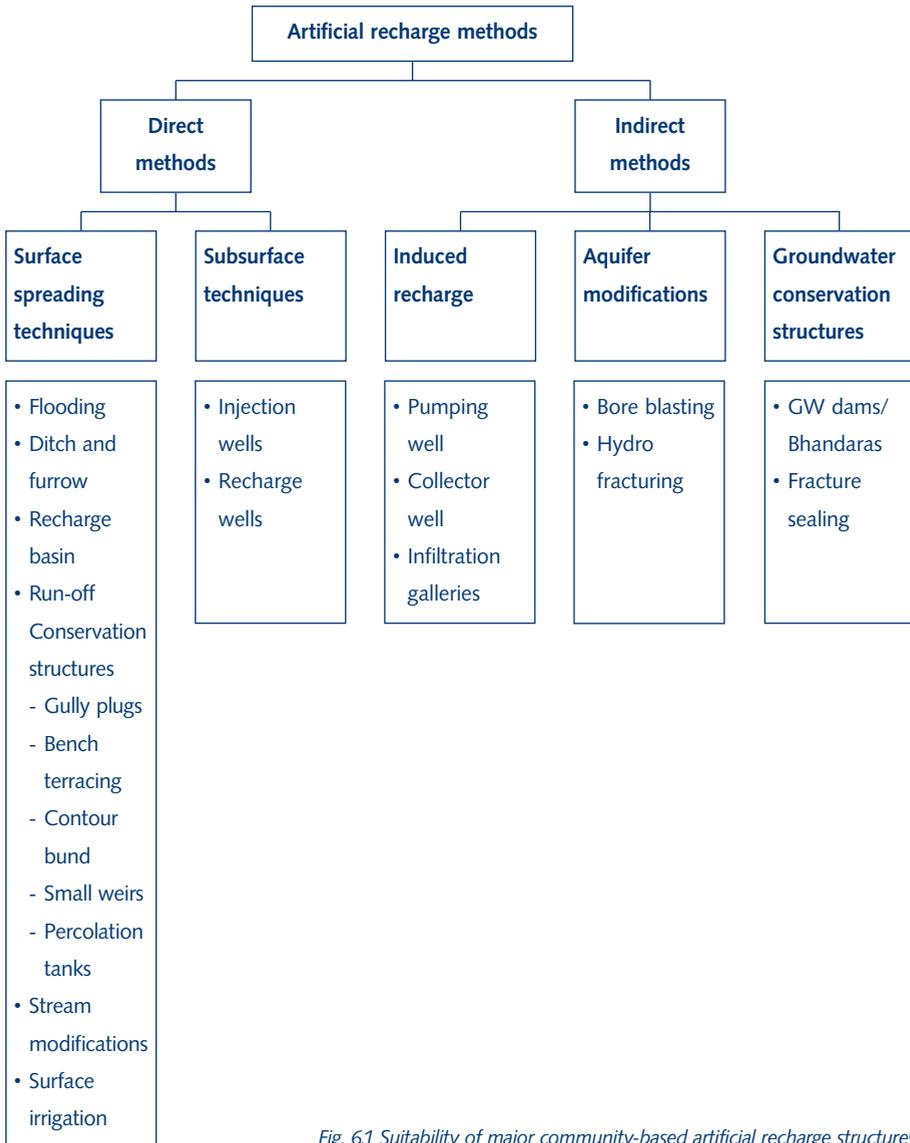


Fig. 6.1 Suitability of major community-based artificial recharge structures

In principle, the choice of an artificial recharge method is controlled by factors such as hydrogeology, socio-economic aspects, sources of water, etc. However, based on operation and maintenance needs and management aspects in the Indian context, the major methods can be grouped as in Table 6.1 below.

Table 6.1 Overview of different artificial recharge methods and their indicative capital cost, construction features and O&M aspects (based on Indian experiences)

Artificial Recharge Methods	Indicative cost in India (US \$ Oct. 2001 level)	Complexity in construction		Operation and maintenance	Remarks
		Technology	Labour involvement		
Percolation tanks	200 per '000 m ³	Complex technology requires technical expertise	Through local contractor	Community based	Zone of influence downstream extends up to 1 km
Recharge shaft (depth 10-15m ø 2-3 m)	1250-1750	Simple technology with technical guidance	Local labour	Community based	Requires yearly desilting
Recharge pit (2mx2mx3m)	100	Simple technology	Local labour	Community based	--do--
Check dams	200 per '000 m ³	Complex technology requires technical expertise	Through local contractor	Community based with little training	
Recharge trench	100-200	Simple technology		Community based	
Recharge through handpump	100-175	Simple technology with technical guidance	Through local contractor	Community based with little training	
Recharge through dug well		Simple technology with technical guidance	Through local contractor	Community based	

6.3 Direct methods

Surface spreading techniques

The following considerations become important before undertaking artificial recharge through surface spreading techniques:

- The aquifer to be recharged should be unconfined and sufficiently thick to provide storage space.
- The surface soil should be sufficiently permeable to maintain a high infiltration rate.
- The vadose zone should be permeable and free from clay lenses that may cause perched water conditions.
- Groundwater levels in the phreatic aquifer should be deep enough to accommodate the water table rise, avoiding possible water logging conditions.
- The aquifer material should have moderate hydraulic conductivity so that the recharged water is retained for a sufficiently long period in the aquifer and can be used at the time of need. Very high permeability results in the loss of recharged water due to subsurface outflow, whereas very low permeability will limit the desired recharge rate.
- Topography plays an important role in controlling the recharge rate. Areas with gently sloping land without gullies or ridges are most suited for surface water spreading techniques.

Five different surface spreading techniques are described below.

Flooding

Flooding techniques are very useful in selected areas where the hydrogeology favours recharging the unconfined aquifer by spreading surplus surface water from canals or streams over large areas for a sufficient length of time to recharge the groundwater body. Figure 6.2 shows the method by which the surplus canal/stream water is diverted through a delivery canal and released as sheet flows over the permeable soil of the area. To ensure proper contact time and water spread, embankments are made on two sides of the area. They guide the unused surface water to a return canal which feeds the excess water back to the original canal downstream.

This technique helps in reducing evaporation losses from the surface water system. The water conserved in the groundwater storage can be pumped for augmenting canal supplies during summer or to provide irrigation water to adjacent areas. It is the least costly of all water spreading methods and maintenance costs are also low.

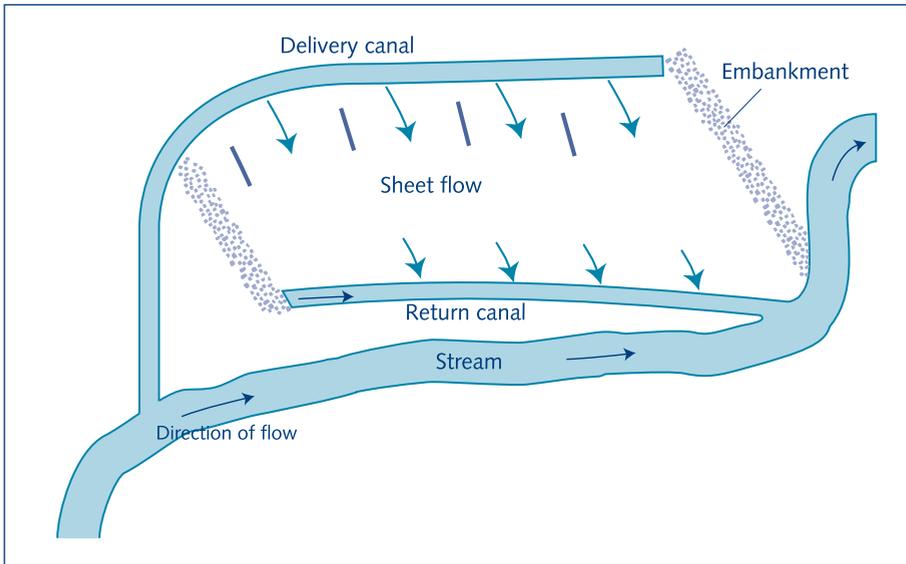


Fig. 6.2. Flooding technique

Ditch and furrow method

In areas with irregular topography, shallow, flat-bottomed and closely spaced ditches or furrows provide maximum water contact area for recharge water from the source stream or canal. This technique requires less soil preparation than recharge basins and is less sensitive to silting. Figure 6.3 shows a typical plan for a series of ditches originating from a supply ditch and trending down the topographic slope towards the stream. Generally three patterns of ditch and furrow systems are adopted (Fig. 6.5, 6.6 and 6.7).

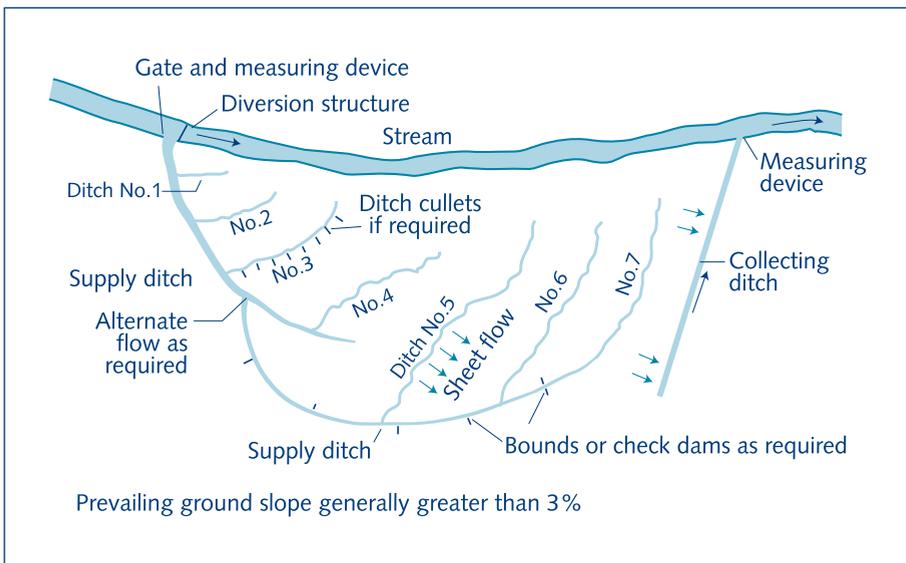


Fig. 6.3. Ditch and furrow method

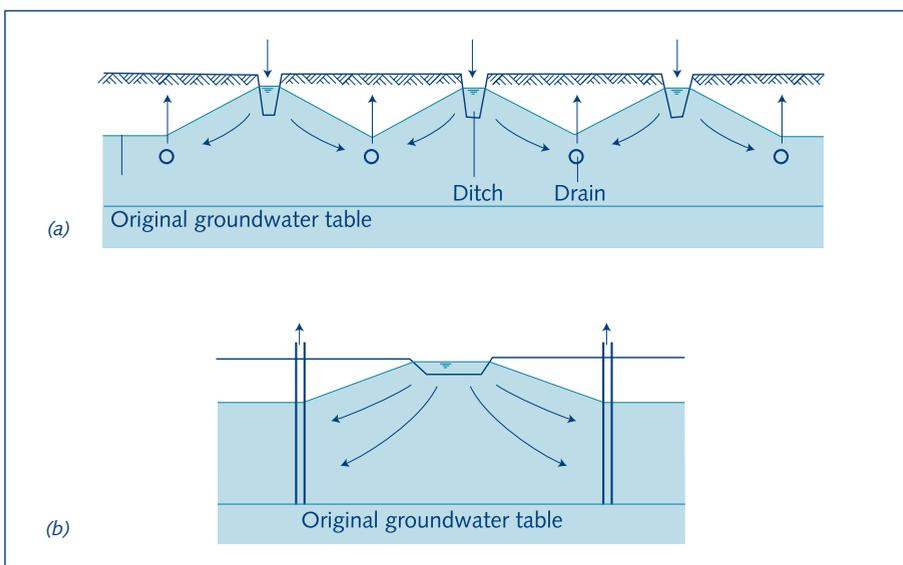


Fig. 6.4. Direct recharge in shallow (a) and deep aquifer (b) with drains and boreholes as recovery method

Direct artificial recharge in shallow aquifers with a high infiltration rate can be accomplished with ditches. In that case drains or galleries are used for groundwater recovery (Fig. 6.4.a). In deeper aquifers the groundwater recovery is via wells or boreholes (Fig. 6.4.b). If the infiltration rate is low, then the infiltration area must be enlarged by larger ditches and shorter ditch intervals.

a. Lateral ditch pattern

The stream water is diverted to the feeder canal/ditch, from which smaller ditches are made at right angles. The rate of flow from the feeder canal to these ditches is controlled by gate valves. The furrow depth is fixed by the topography and also to achieve maximum wetted surface and uniform velocity. The excess water is routed to the main stream through a return canal, along with residual silt.

b. Dendritic (tree-like) pattern

The water can be diverted from the main canal to a series of smaller ditches spread in a dendritic pattern. The branching continues until practically all the water is infiltrated in the ground.

c. Contour pattern

Ditches are excavated following the ground surface contour of the area. When the ditch comes close to the stream a switchback is made and the ditch is made to meander back and forth to traverse the spread area repeatedly. At the lowest point downstream the ditch joins the main stream, thus returning the excess water to it.

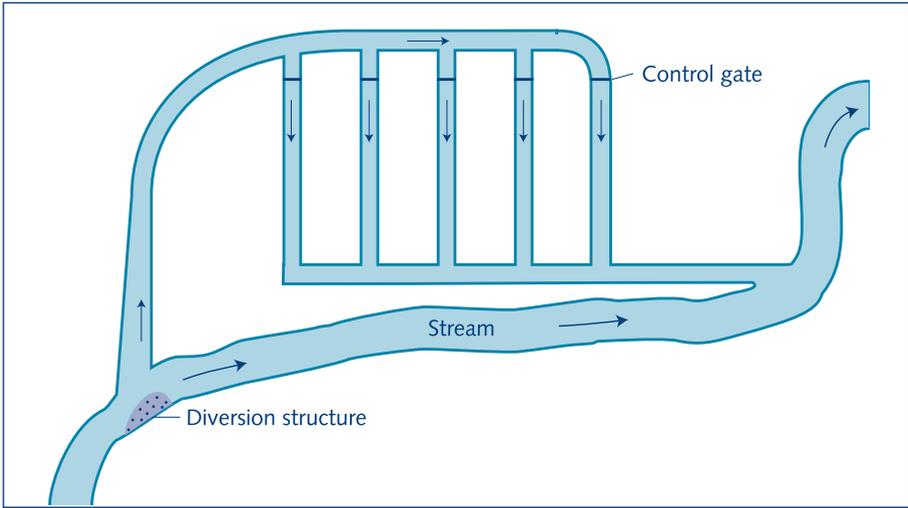


Fig. 6.5. Lateral ditch pattern

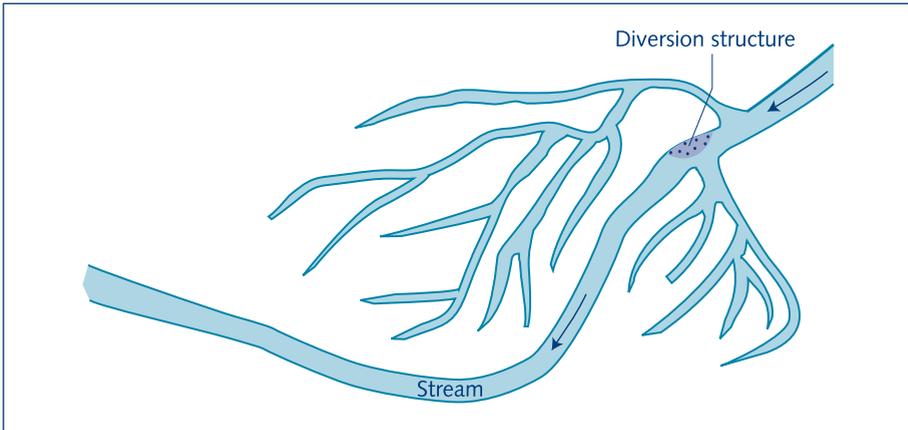


Fig. 6.6. Dendritic ditch pattern

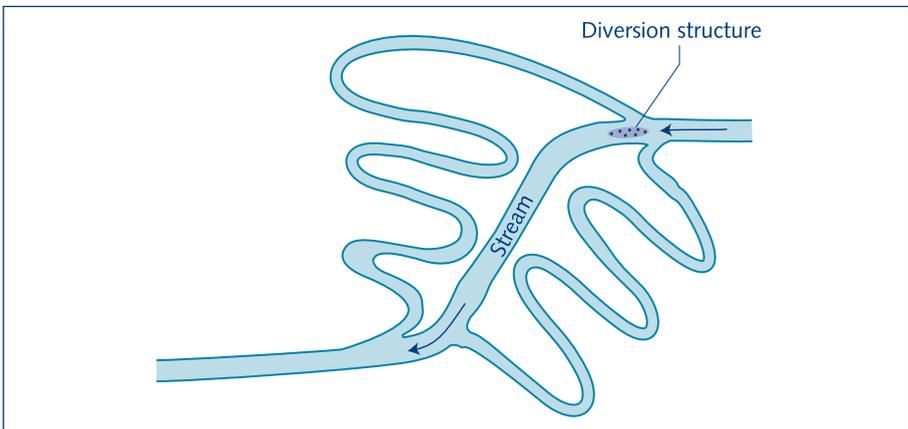


Fig. 6.7. Contour ditch pattern

Recharge basin

Artificial recharge basins are either excavated or are enclosed by dykes. They are commonly built parallel to ephemeral or intermittent stream channels (Fig. 6.8). They can also be constructed at other locations where a canal or any other water source provides the water.

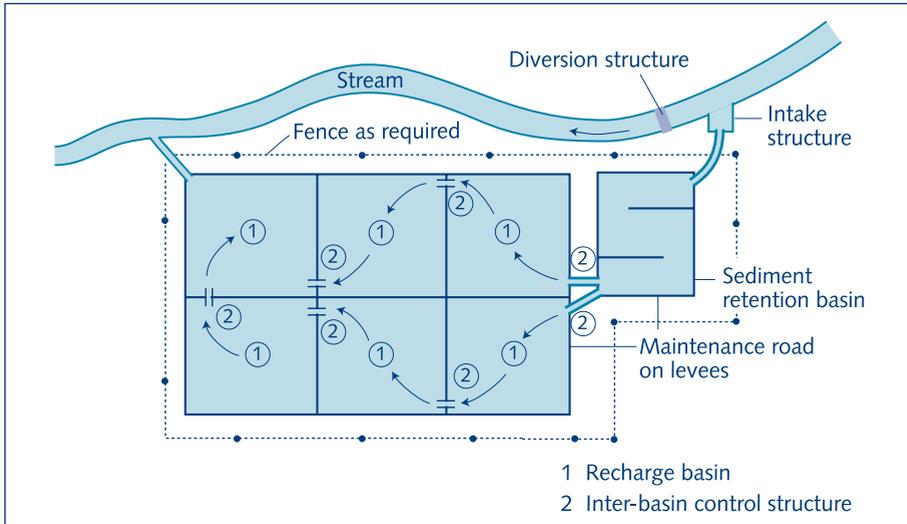


Fig. 6.8. Recharge basin

In alluvial areas multiple recharge basins are generally constructed parallel to the stream. The advantages of multiple basins are:

- water contact time is longer for the stored water;
- suspended material in the source water is reduced as water flows from upstream basins to those below;
- periodic maintenance such as scraping etc. to restore infiltration rates can be done by bypassing the basin concerned.

Run-off conservation structures

Rainfall is a major source of water but it is not evenly distributed throughout the year. During the monsoon period, surplus water is wasted in the form of surface run-off. Water resources planning should address this phenomenon by making efforts to harvest rainwater, especially during rainy seasons.

The main aim of the rainwater harvesting is to conserve the generated surface run-off by collecting it in reservoirs, both surface and sub-surface. The objectives of the rainwater conservation in groundwater reservoirs are:

- Increase the availability of groundwater
- Enhance sustainable yield of aquifers

- Improve quality of groundwater through dilution
- Arrest declining trends of water levels
- Prevent depletion of groundwater reservoirs in areas of over exploitation
- Decrease menace of floods on local and regional areas
- Reduce pressure on storm drains in urban areas
- Enhance the quality of the environment

Rainwater harvesting methods have to be site-specific. The choice and effectiveness of any particular method is governed by local geology, hydrogeology, terrain conditions, total rainfall and its intensity, etc. The rainwater harvesting process includes collection of rainwater, conveyance to a suitable place and then storage in a surface and/or sub-surface groundwater reservoir. The methods listed below are in vogue for conservation of rainwater.

Hilly and open fields

- Basins/percolation tanks
- Check dams
- Ditch and furrows
- Recharge pits and shafts
- Injection tube wells and dug wells
- Sub-surface dams

Urban Areas

- Injection wells/dug wells
- Recharge trenches with injection wells
- Recharge shafts

In areas receiving low to moderate rainfall mostly during a single monsoon season and not having access to water transferred from other areas, water conservation is necessarily linked to “in situ” precipitation. Multi-purpose measures are desirable, that are mutually complimentary and conducive to soil and water conservation, afforestation and increased agricultural productivity. Different measures are applicable in run-off zones, recharge zones and storage zones of a watershed. The structures widely used are (i) gully plug; (ii) bench terracing; (iii) contour bund; (iv) small weirs; and (v) percolation tank.

Gully plug. Gully plugs are the smallest run-off conservation structures built across small gullies and streams rushing down the hill slopes and carrying run-off from tiny catchments during rainy seasons. Usually the bund is constructed by using local stones, earth and weathered rock, brushwood, and other such local materials. Sand bags or brickwork may also be used to strengthen the bund.

Bench terracing. Sloping lands with surface gradients up to 8% and with adequate soil cover can be levelled by bench terracing to bring them under cultivation. Bench terracing helps in soil conservation and in holding run-off water on the terraced areas for longer, resulting in increased infiltration recharge. A map of the watershed should be prepared by level surveying and suitable benchmarks fixed. A 0.3m contour map is the most suitable. Depending on the land slope, the width of individual terraces should be fixed. It should never be less than 12m. The upland slope between two terraces should not be more than 1:10 and the terraces should be levelled.

Contour bunding. Contour bunding is a watershed management practice to build up soil moisture storage. This technique is adopted generally in low rainfall areas. The monsoon run-off is impounded by putting bunds on the sloping ground all along a contour (Fig. 6.9). The flowing water is intercepted before it attains erosive velocity by keeping suitable spacing between two bunds. The spacing depends on the slope of the area and the permeability of the soil. Lower soil permeability means closer spacing of bunds.

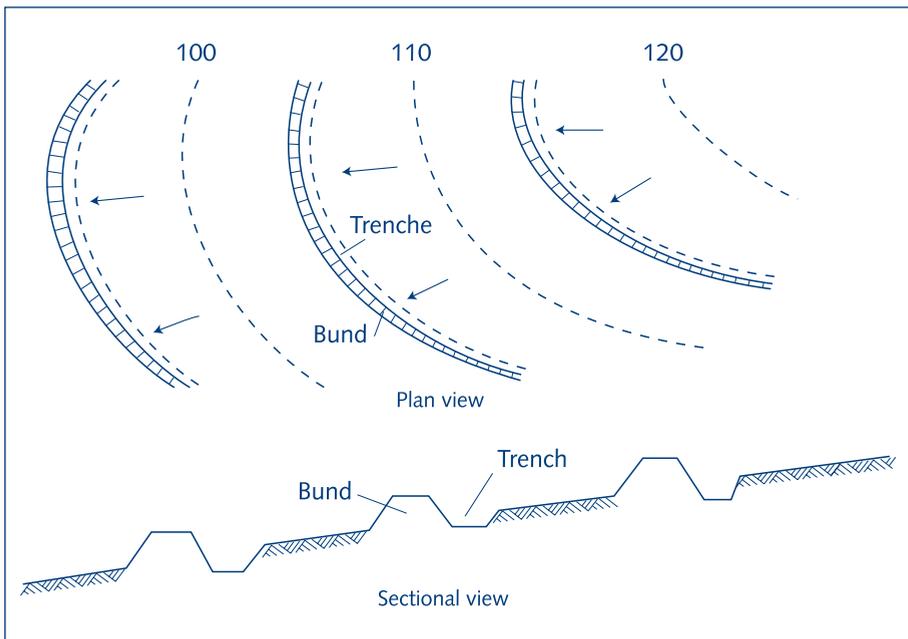


Fig. 6.9. Schematic diagram of contour bunding

Small weirs. A series of small weirs are made across selected stream or irrigation canal sections, so that the stream flow is hindered and water is retained on pervious soil/ rock surfaces for longer. As compared to gully plugs, weirs are constructed across bigger irrigation canals or second order streams with gentler slopes. The reservoir behind the weir acts like a mini percolation tank.

Percolation tanks. Percolation tanks are artificially created surface water bodies submerging a highly permeable land area so that surface run-off is made to percolate and recharge the groundwater storage (Fig. 6.10).

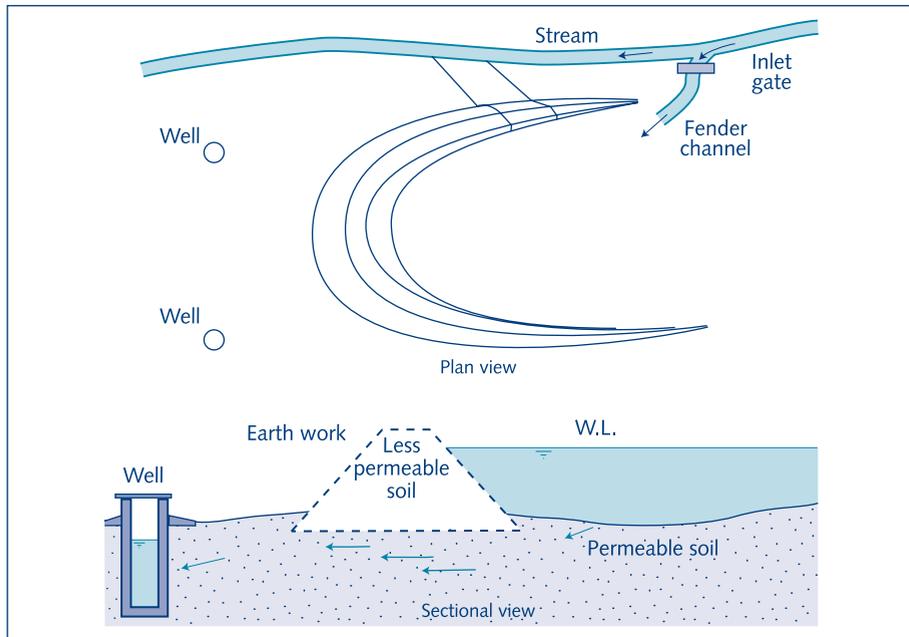


Fig. 6.10. Schematic diagram of percolation tank

For a proper functioning of the percolation tank the hydrogeological characteristics of the site are of utmost importance. The rocks or soil to be submerged should have high permeability, and weathering of the rocks should be laterally extensive and not just localised. Rainfall patterns should ensure that the percolation tank gets filled every year. The soils in the catchment area should preferably be light and sandy to avoid silting up of the tank bed. The canal or stream chosen to create a percolation tank should have an adequate catchment area. The influent run-off volume during a rainy season should range between 300,000 and 800,000 m³. The size of a percolation tank is governed by the percolating capacity of the strata in the tank bed rather than yield of the catchment. Normally percolation tanks are designed for storage capacity of 200,000- 600,000 m³. The depth of impounded water provides the recharge head. Hence, the tank design should result in a minimum height of ponded water of 3 to 4.5 m, but not more than 6 m.

Stream channel modification

A natural drainage channel can be modified so as to increase infiltration by detaining stream flow and increasing the stream bed area in contact with water. The channel is so modified that the flow gets spread over a wider area, increasing contact with the percolating river bed. The method includes (i) widening, levelling, scarifying or ditching

the stream channel; (ii) L-shaped finger levees or hook levees constructed by bulldozer at the end of the high flow season; and (iii) low head check dams that allow flood flows to pass over safely.

Stream channel modification methods are generally applied in alluvial areas. However, they can be effectively used also in hard-rock areas where thin river alluvium overlies a good phreatic aquifer or the rocks are extensively weathered or highly fractured in and around the stream channel offering scope for artificial recharge.

Surface irrigation

As well as the five surface spreading techniques described in previous sections, recharge of aquifers occurs in a less controlled way through excessive surface irrigation.

Under well-managed modern irrigation practices, a measured amount of irrigation is practised to avoid excess seepage losses (unintended recharge). However, often in irrigation practices the farmers tend to use excessive amounts of water by flooding the fields whenever water is available. The large number of unlined irrigation canals also contribute significantly to the groundwater recharge. Irrigation of land with poor drainage facilities may lead to the development of water logging and salinisation of large areas.

Surface irrigation systems have thus caused an unintended recharge in many areas and groundwater capacity has increased. However, the use of motorised pump sets for lifting groundwater for irrigation has caused substantial drops in groundwater levels. This has resulted in wells/boreholes drying up, leaving the rural poor in particular with serious drinking water supply problems.

Sub-surface techniques

Recharge pits

Phreatic aquifers are not always hydraulically connected to surface water. On a regional scale impermeable layers or lenses form a barrier between the surface water and the water table, making water spread methods less effective. For effective recharge of the shallow aquifer, the less permeable horizons have to be penetrated to make the aquifer directly accessible. Recharge pits are one option. They are excavations of variable dimensions that are sufficiently deep to penetrate less permeable strata (Fig. 6.11). Recharge pits differ from gravity head recharge wells as the latter do not necessarily reach the unconfined aquifer and the recharging water has to infiltrate through the vadose zone.

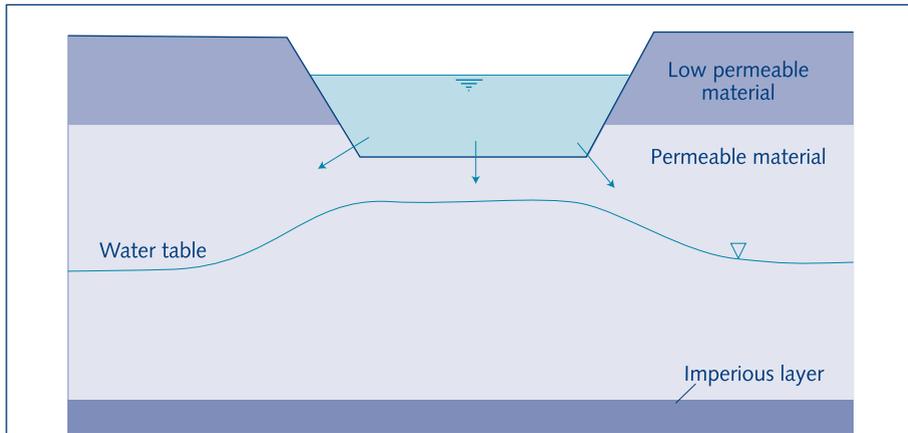


Fig. 6.11. Recharge pit

The larger the cross-sectional area of the pit bottom, the more effective it will be. The actual area required depends on the design recharge volume and the permeability of the underlying strata. Therefore the permeability has to be determined.

The steep side slopes and low permeability of these strata mean that sedimentation occurs only on the bottom and that clogging of side walls is limited. The side-wall slope should be about 2:1. The bottom area of open pits may require periodic manual cleaning. If the recharge pits have a filter pack, then the upper layer of the filter pack also requires periodic cleaning or replacement.

Recharge shafts

In cases where an aquifer is located deep below the ground surface and overlain by poorly permeable strata, a shaft is used for artificial recharge. A recharge shaft is similar to a recharge pit but much smaller in cross section (Fig. 6.12).

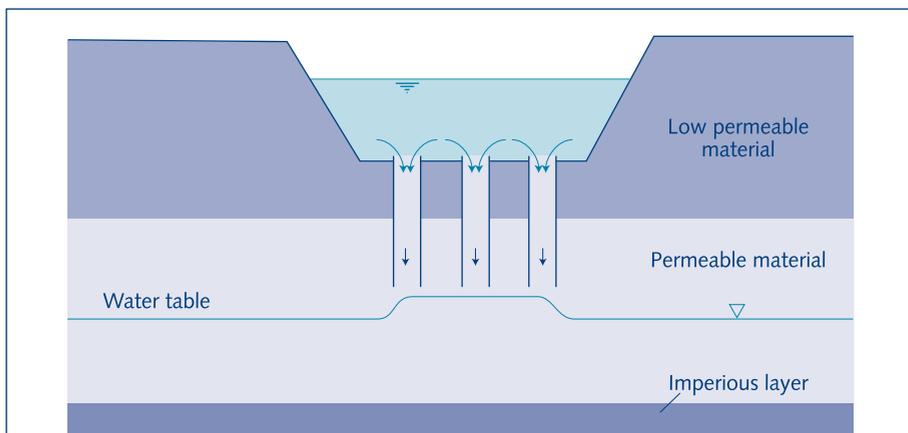


Fig. 6.12. Recharge shafts

A recharge shaft may be dug manually if the strata is non-caving. In good hard clays it is possible to dig deep shafts down to 25 to 30 m depth without lining. For still deeper shafts, bailing, or drilling by reverse circulation or the direct rotary method may be required. When manually dug, the shafts need to be about 2 m in diameter at the bottom. In case of drilled holes the diameter may not exceed beyond 0.8–1 m. The shaft should end in more permeable strata below the confining layer. It may not touch the water table. Unlined shafts should be backfilled, initially by boulder cobbles and then the top few metres by a gravel and sand filter. If the filter material gets choked after some time because of suspended solids in the raw water settled on the upper layers, it should be dug out or bailed out and a fresh filter provided. Siltation and clogging of deeper infilling is difficult to remove. Biotic growth may also clog the infilling. Choked-up shafts are difficult to clean and have to be abandoned. Deeper shafts dug out in caving strata need lining or casing. It is not necessary to backfill lined shafts completely. A few metres thick gravel or coarse sand filter can be placed at the bottom. In case of clogging, the filter material can be removed by bailing and replaced with clean filter material. In lined shafts, the recharge water may be fed through a smaller conductor pipe reaching up to the filter pack.

6.4 Water harvesting technology options

There are many ways of harvesting rainwater. Chapter 7 goes into detail about the most popular rainwater harvesting methods. Here, we discuss three water harvesting systems that can be adopted with the involvement of the local people and achieve maximum value for the resources they demand. They are rooftop harvesting, surface run-off and underground harvesting. It is important to realise that each of these systems has its own characteristics, limitations and advantages.

Rooftop rainwater harvesting

In rooftop rainwater harvesting, the rainwater is collected from roofs of buildings and stored in a groundwater reservoir for beneficial use in future. Advantages:

- Provides water supply self sufficiency
- Reduces the cost of pumping
- Reduces soil erosion in urban areas
- Inexpensive and simple and can be adopted by individuals
- Utilises the rainfall run-off, which usually goes to sewers or storm drains
- Improves the quality of existing groundwater through dilution
- Rainwater may be harvested at place of need and may be utilised at time of need
- In saline or coastal areas it provides good quality (fresh) water and may help in maintaining a balance between the fresh-saline water aquifers
- On islands, due to limited fresh water aquifers, it is a preferred source of water for domestic use
- In deserts it provides an opportunity to store water

Design of recharge techniques

1. Abandoned dug well (Fig. 6.13)

A dry/unused dug well can be used as a recharge structure. The recharge water is guided through a pipe to the bottom of the well or at least below the standing water level, to avoid scouring the bottom and trapping air bubbles in the aquifer. Before using the dug well as recharge structure, its bottom should be cleaned and all the fine deposits should be removed. Recharge water should be silt free. The well should be cleaned regularly. Periodic chlorination should be used to control bacteriological contamination. This technique is suitable for large buildings, having a roof area of more than 1000 m².

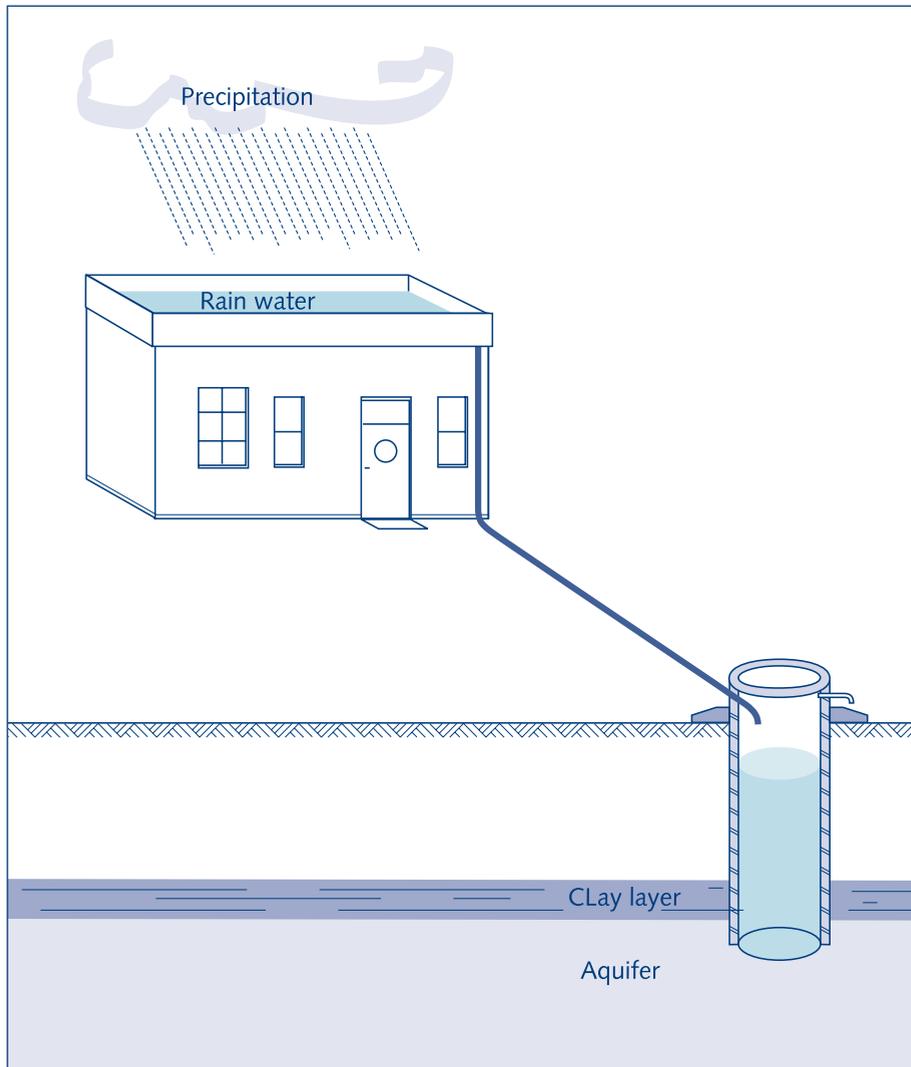


Fig. 6.13. Recharge through abandoned dug well

2. Abandoned borehole (Fig. 6.14)

An abandoned borehole can be used for recharge. The structures are suitable for small buildings having a roof area up to 150 m². Water is diverted from the rooftop to the borehole through a 50-100 mm diameter pipe. Recharge water should be silt free.

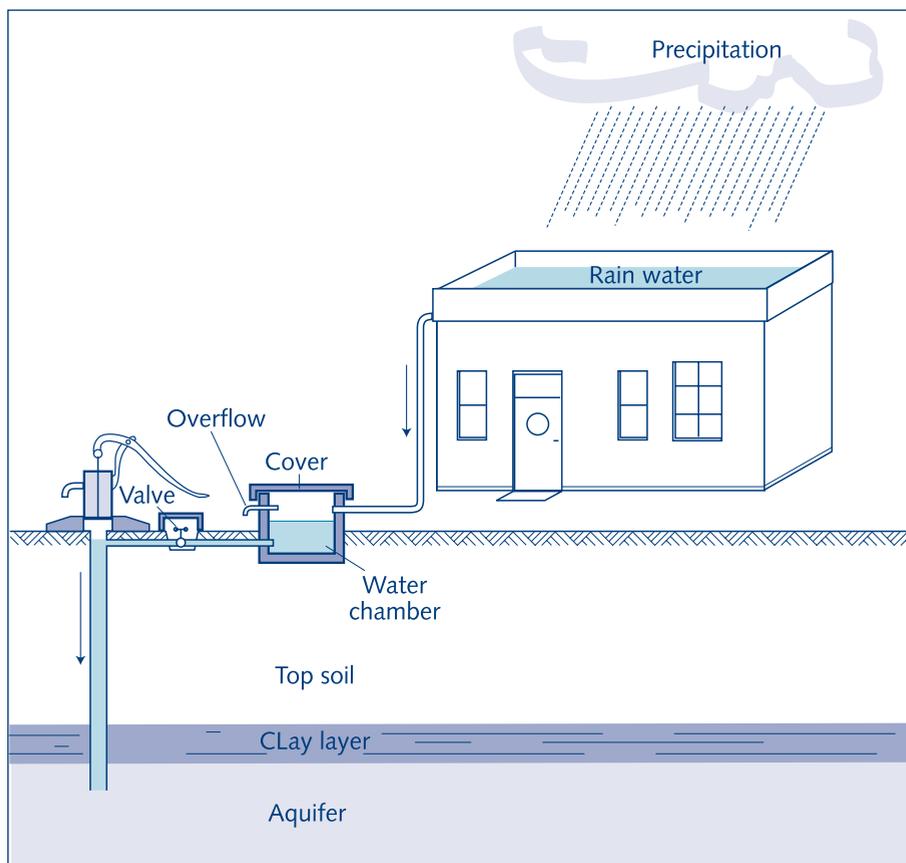


Fig. 6.14. Recharge through abandoned borehole

3. Recharge pit (Fig. 6.11)

Recharge pits are constructed for recharging a shallow aquifer. They are generally 1-2 m wide and 2-3 m deep. After excavation, the pits are refilled with pebbles and boulders. Water to be recharged, should be silt free. Cleaning of the pit should be done periodically. It is suitable for small buildings having a roof top area up to 100 m². Recharge pits may have any shape, i.e. circular, square or rectangular. If the pit is trapezoidal, the side slopes should be steep enough to avoid silt deposition.

4. Recharge trench (Fig. 6.15)

Recharge trenches are constructed if permeable strata of adequate thickness are available at shallow depth. The shallow trench is filled with pebbles and boulders.

Trenches are constructed across the land slope. The trench may be 0.5-1 m wide, 1-1.5 m deep and 10-20 m long depending upon the availability of land and roof top area. It is suitable for buildings having a roof area of 200 to 300 m². Cleaning of the trench should be done periodically.

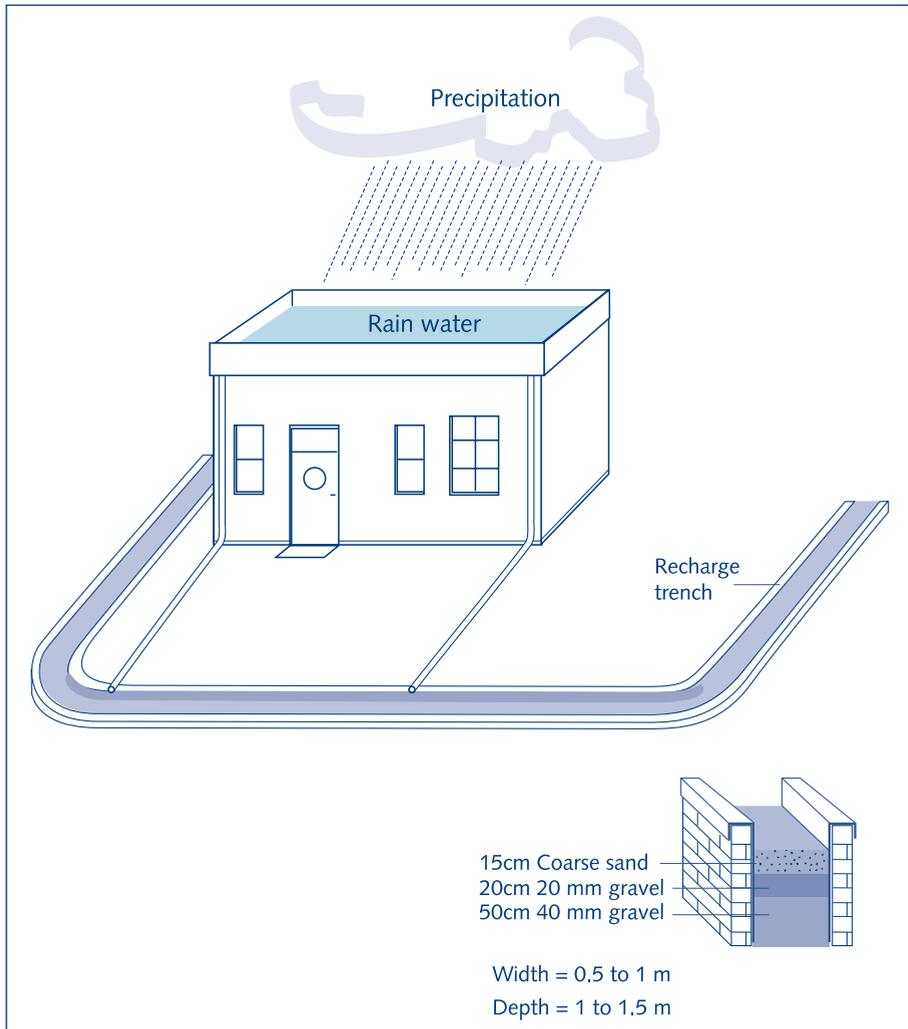


Fig. 6.15. Recharge through trench

5. Gravity head recharge well or borehole (Fig. 6.16)

Boreholes and wells can be used as recharge structures. This technique is suitable where land availability is limited and the aquifer is deep and overlain by impermeable strata (e.g. clay). The rooftop rainwater flows to the well and recharges under gravity. Recharge water should be silt free. The technique is most suitable for areas where the groundwater level is deep. The number of recharging structures for the limited area around the buildings is determined by the rooftop surface area and aquifer characteristics.

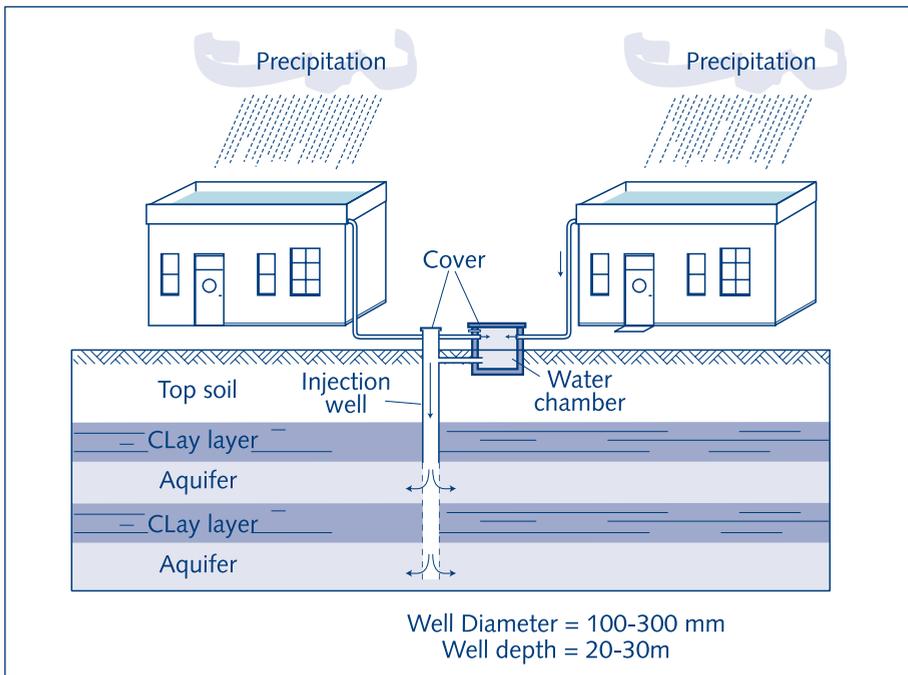


Fig. 6.16. Gravity head recharge well or borehole

6. Recharge shaft (Fig. 6.12)

A recharge shaft may be dug or drilled. Diameters of recharge shafts vary from 0.5-3 m depending upon the availability of water to be recharged. They are constructed where the shallow aquifer is located below strata with low permeability. They should end in more permeable strata (e.g. sand). Depths of recharge shafts vary from 10-15 m below ground level. The recharge shaft is backfilled with boulders, gravels and coarse sand. It should be cleaned periodically by scraping the top layer of sand and refilling it.

Surface catchment systems

Surface catchment water harvesting systems are large-scale communal schemes that collect and store water running off a specific part of the local landscape. This entails either a rocky outcrop or an area of compacted or clay-rich soil. The former is coupled with a rock masonry dam and the latter with a semi-circular clay earth dam.

Masonry check dam

A masonry rock catchment dam may consist of a single straight wall or a number of sections of differing heights or lengths, depending on the shape of the site and the desired size of the reservoir. Wall dimensions range from 2-6 m in height and 10-60 m in length.

Dams are constructed on rocky outcrops, either in rock-top slope areas or lowlands where individual inselbergs or depressions in the river surface are found. The site for the

dam and the bottom of the reservoir should be free from rock fissures or fractures that might drain the water away from the site. Aerial photographs and a field survey could assist in the detection of fractures and in the selection of possible sites.

The foundations must be on almost flat, unweathered rock surfaces or rock surfaces sloping slightly backwards to other reservoirs. This ensures the dam's stability and simplifies the design (reduced need for reinforcement). The reservoir should preferably be deep, minimising reservoir surface area so that evaporation losses are minimised. The dam should not exceed a maximum height of 5 m for the simple masonry wall design.

Small dam systems

Small earth dam

The small earth dams discussed in this section are semi-circular or curved banks of earth, generally not more than 3 m in height and 60 m in length. They are built mainly by manual labour, animal traction and light machines (Fig. 6.17). They can be maintained by the user community. Larger constructions are beyond the community-based approach described in this document.

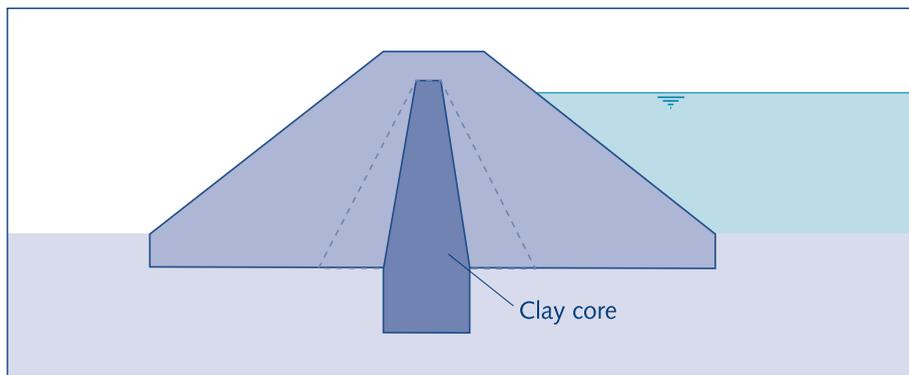


Fig. 6.17. Cross-section of a small earth dam with an impermeable clay core

General design features

In designing simple manually-constructed earth dams, the following design features must be taken into account:

- Ensure a sound foundation that avoids seepage under the dam.
- The dam dimensions must be large enough to ensure stability.
- An outlet pipe system and water tap point (or a transmission pipe to water points closer to the users) should be constructed to abstract water downstream of the dam.
- Two stone spillways are constructed to avoid water overtopping and eroding the dam walls.
- The upstream wall is fully covered with stones to protect it from wave and run-off damage.

- Clay should be used as the primary construction material to achieve an impervious dam and avoiding seepage. Care should be taken in selecting and compacting the clay.
- The dam should be fenced off, for example with live thorn fencing or cut thorn bush to keep livestock from walking along the dam's sides and damaging the structure.

Sand dams

A sand dam is a concrete or masonry barrier constructed in an ephemeral river. Upstream of the sand dam the reservoir fills with sediments carried by the river water during high discharge periods. These sediments may be loose rock, stones, and coarse and finer sand. The river water carries away the fine sand and fine suspended solids. Therefore the risks of siltation are small. The filling-up process of the sand dam may take several years, depending on the sediment transport in the river. If mainly finer sediments are present in the river then the sand dam is best built in stages. This is to avoid smaller particles being retained, limiting the permeability of the sand dam body. In the wet season the high velocity flood waters prevent the silt and mud carried by the river from settling on the sand dam body.

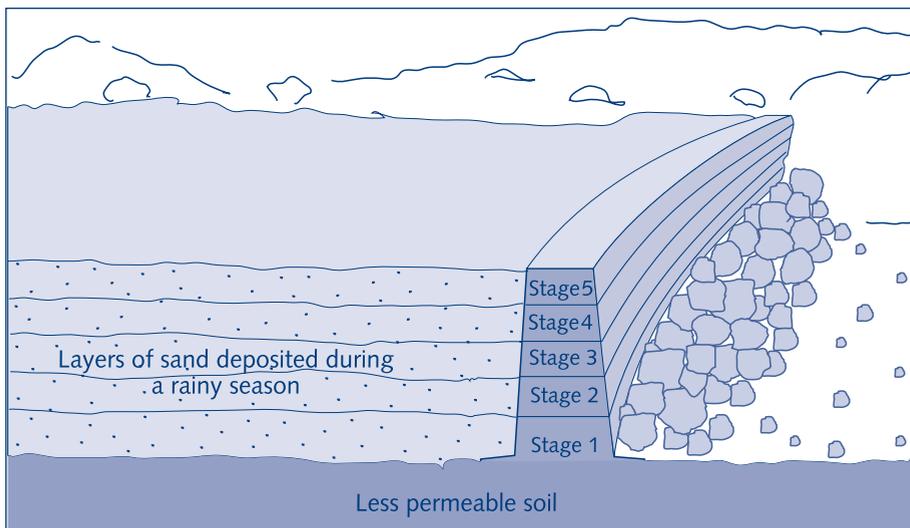


Fig. 6.18. Sand dam (Adapted from Nissen-Petersen, E., 1986)

The water is stored in the pores of the accumulated coarse material in the reservoir of the sand dam. Provided that the bed and walls of the reservoir are impermeable, the water may be stored for long periods. The fact that the water is stored in a sand bed greatly reduces the evaporation losses of the water. Therefore, sand dams are particularly suitable in arid and semi-arid areas with high evaporation rates.

The building can be done in stages, starting with a dam height of some 1.5 m. Annually, depending on the filling rate of the reservoir, the height of the dam can be increased by 0.5-1.0 m. For low dams, a development in stages is not needed.

To retain the stored water in the sand dam selecting the right location is important. The geology in which the dam is to be constructed must be as impermeable as possible to avoid seepage. Weathered and fissured rocks, and sand or coarse medium soils are unsuitable sites. The right foundation is also a key factor for the stability and performance of the sand dam.

Anchoring the dam to the banks of the river needs special attention. Particularly when the soils are soft, floods tend to go around the dam, eroding the land and possibly changing the river course away from the dam and destroying the land. Constructing long wings of sufficient height and growing plants will reduce this risk. The inclusion of a spillway in the centre of the dam will also help to reduce erosion.

Protective measures should also be taken against erosion at the downstream side where the water passes over the dam. A hard surface of rocks, boulders, etc. will prevent the erosion substantially.

Water is withdrawn from the sand dam by a drain pipe (a perforated pipe surrounded by a gravel pack) or from a well dug into the sand bed in or next to the dam. Usually the water can be used without further treatment as the coarse material of the sand dam acts as a filter.

Sub-surface barrier

Sub-surface barriers are used to retain seasonal sub-surface flows and facilitate the abstraction of water through wells or boreholes. To achieve this, an impermeable barrier – either of clay or masonry – is constructed across the river bed from the surface down to an impermeable layer below.

General design features for a sub-surface clay dam (Fig. 6.19)

- The construction of a clay dam should commence immediately after the main rainy season and should be completed before the next rainy season.
- Clay is the primary construction material. Careful selection and compacting of the clay ensures an impervious dam and avoids seepage through fissures and cracks.
- The foundation must be sound and watertight. This avoids seepage under the dam that can lead to loss of stored water.
- The dam must be sufficiently extended into the banks to avoid seepage around the sides of the dam.
- The dam must be two metres wide all the way down to the foundation.

- The top of the dam needs to be protected against erosion from stream forces.
- Rocks should be piled against the banks, both upstream and downstream to protect them from erosion.
- The dam should be located where the river bed is narrower and the sand layer becomes thinner.

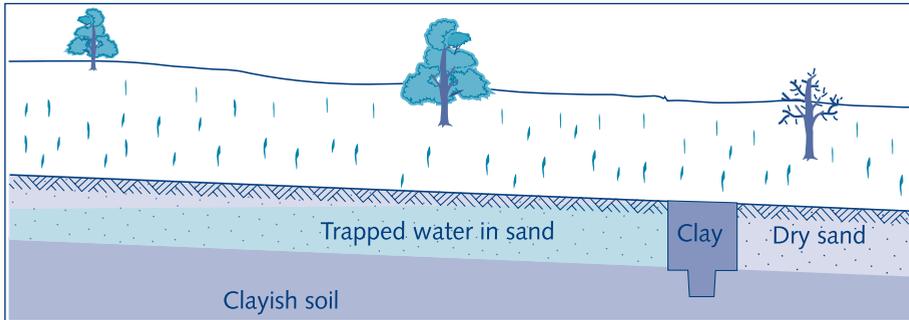


Fig. 6.19. Sub-surface clay dam (Adapted from Nissen-Petersen, E., 1986)

General design features for a sub-surface masonry dam (Fig. 6.20)

- The construction of a masonry dam should commence immediately after the main rainy season and must be completed before the next main rainy season starts.
- The dam should be 50 cm wide.
- The height of the dam depends on the depth of the bedrock layer.
- The dam should be located where impermeable bedrock is less than 5 m below the river bed.
- A sound, watertight foundation must be constructed to avoid seepage under the dam.
- A spill-over apron must be constructed to protect the downstream side of the dam from erosion caused by flowing water.
- The dam should be extended with two wing walls into the river bank to prevent seepage between the river banks and the dam.
- The top of the dam and side walls must be protected against erosion from flowing water.

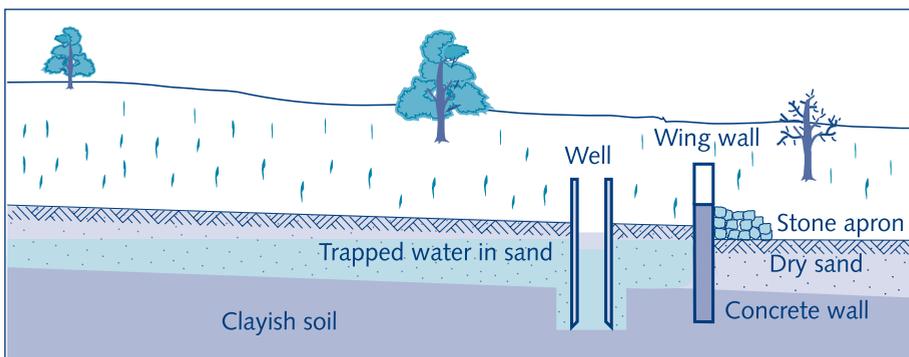


Fig. 6.20 Sub-surface masonry dam (Adapted from Nissen-Petersen, E., 1986)

Percolation tank

Introduction

Percolation tanks (Fig. 6.10) are being recognised as a sound technology not only for rainwater harvesting but also for augmenting groundwater recharge in hard-rock terrain. Hard-rock areas with limited to moderate water holding and water yielding capacities often lead to water scarcity due to inadequate recharge, indiscriminate withdrawal and mismanagement. In this context, percolation tanks are increasingly adopted as a tool for ensuring sustainable development of groundwater. The percolation tank is more or less similar to check dams or small weirs with a fairly large storage reservoir. The tank can be located either across small streams by creating low elevation check dams or in uncultivated land adjoining streams, by constructing a delivery canal connecting the tanks and the stream. Percolation tanks are artificially created surface water bodies that are submerging a land area with adequate permeability to facilitate sufficient percolation of impounded surface run-off to recharge the groundwater.

Design aspects

The design of percolation tanks involves detailed consideration of the following aspects:

- The catchment yield needs to be calculated for long-term average annual rainfall.
- The design of the dam is based on:
 - the topographical setting of the impounded area to calculate the height and length of the dam wall, its gradient, width and the depth of the foundation, taking into account the nature of the underlying formation;
 - details of the cut-off trench, to reduce seepage losses;
 - height of stone pitching on the upstream slope to avoid erosion due to ripple action;
 - suitable turving on the downstream slope to avoid erosion from rain;
 - upstream and downstream slopes to be moderate so that shear stress is not induced in the foundation beyond a permissible limit;
 - stability of the dam.
- Percolation tanks are normally earth dams with masonry structures only for the spillway. Construction materials consist of a mixture of soil, silt, loam, clay, sand, gravel, suitably mixed and laid in layers and properly compacted to achieve stability and water tightness. The dam is designed not to be over-topped, by providing adequate length of waste weir and freeboard.
- A waste weir is provided to discharge water when the dam overflows. Once the discharge is known the length of the waste weir is decided depending on the volume of maximum permissible flood discharge and permissible flood depth from the waste weir.
- Measures already indicated for the protection of catchment areas of rock dams hold good in the case of percolation tanks also.

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Web sites

There are no specific artificial recharge websites, but there are several interesting experiences documented and disseminated via internet. Use any search engines and general or more specific key words to find valuable information on the net.

7 Rainwater harvesting

John Mbugua

7 Rainwater harvesting

7.1 Introduction

Rainwater catchment and storage reservoirs have been constructed since ancient times in several parts of the world. Some have been preserved to this day. Rainwater is harvested as it runs off roofs, or over natural ground, roads, yards, or specially pre-pared catchment areas. Historical sources mention the use of rainwater for domestic water supply some 4000 years ago in the Mediterranean region. Roman villages and cities were planned to take advantage of rainwater for drinking water supply. In the hills near Bombay in India, the early Buddhist monastic cells had an intricate series of gutters and cisterns cut into the rock to provide domestic water on a year-round basis.

In Europe and Asia rainwater harvesting was used widely for the provision of drinking water, particularly in rural areas. In many countries it is still being practised. However, where piped water supplies have been provided, the importance of rainwater as a source of supply has diminished.

On some tropical islands rainwater continues to be the only source of domestic water supply. In arid and semi-arid areas where people mostly live in scattered or nomadic settlements, rainwater harvesting can be a necessary means of providing water for domestic purposes. This is especially the case where groundwater resources are unavailable or costly to develop. In many developing countries, rainwater is used to supplement the piped water supply.

Rainwater harvesting means capturing the rain where it falls or capturing the runoff in farms, villages or towns and taking measures to store that water and keep it clean.

Rainwater harvesting can be undertaken through a variety of ways:

- Capturing run-off from roof tops
- Capturing run-off from local catchments
- Capturing seasonal floodwater from local streams
- Conserving water through watershed management

Harvesting rainwater has several functions:

- Providing water to people and livestock
- Providing water for food and cash crops
- Increasing groundwater recharge
- Reducing storm water discharges, urban floods and overloading of sewage treatment plants
- reducing seawater ingress in coastal areas

Rainwater harvesting should be considered as a source of domestic water in countries where rain comes in storms of considerable intensity, with intervals during which there is little or no rainfall. It requires adequate provision for the interception, collection and storage of the water.

Food security is also a great concern in countries where population pressure is growing while food production declines. Among other things, decreased agricultural production is due to poor soil fertility management. Soil loss and reduced moisture retention of the soil occur because of accelerated run-off. There is an urgent need to halt progressive soil degradation through more and better rainwater harvesting systems. If storm water run-off is controlled and stored in water harvesting systems, it can be used to improve water and food security.

The goal of good watershed management is to hold water where it falls as precipitation, at the same time protecting the land from erosion. By various technical and vegetative measures, the water can be slowed down on its path towards the main stream in the valley and redirected to feed a series of land and water harvesting structures at each level and to replenish the groundwater table. There is, therefore, eminent sense - ecological, financial and political - in promoting community and household-based water harvesting systems.

A domestic rainwater harvesting system can consist of a collection surface, a storage tank, and guttering or channels to transport the water from one to the other. Other peripheral equipment is sometimes incorporated, such as a first-flush system to divert the initial dirty water that contains roof debris built up during prolonged dry periods; filtration equipment; and settling chambers to remove debris and contaminants before the water enters the storage tank or cistern.

User regimes

Rainwater harvesting is used in many different ways. In some parts of the world with abundant rainfall and short dry spells, only a small storage capacity is required. However, in arid areas where people will depend totally on rainwater harvesting to supply their domestic water needs throughout the dry season, a sufficiently large collection surface area and storage capacity will be required to provide enough water to meet the needs of the user. Between these two extremes, a wide variety of different user patterns or regimes are possible.

Some of the variables that determine the patterns of usage for rainwater harvesting are:

- Total rainfall quantity
- Rainfall pattern
- Collection surface area

- Storage capacity
- Daily consumption rate and number of users
- User demands from men and women
- Socio-economic differences in demand
- Cost of the system(s)
- Alternative water sources
- Restricted use options
- Unplanned water use
- Intermittent use options
- Partial use of rainwater
- All water needs from harvested rainwater

7.2 Collection surface

Roof of a house/dwelling

For domestic rainwater harvesting the most common surface for collecting water is the roof of the dwelling. Many other surfaces can be and are used. Reasonably pure rainwater can be collected from house roofs made of tiles, slates, (corrugated) galvanised iron, aluminium or asbestos cement sheeting. Most thatched roofs are not suitable because of the colouring and increased turbidity of the water caused by organic material. However, thatched roofs made of certain palm leaves can make good catchments. With very corrosive rainwater, the use of asbestos cement sheeting for roof catchment requires some caution. Asbestos fibres may be leached from the roof material leading to relatively high asbestos concentrations in the collected rainwater. However, there is no evidence that asbestos fibres in drinking water pose adverse health effects. On the other hand, inhalation of high concentrations of asbestos fibres present in the air may cause lung cancer. As a result, production of asbestos products has been banned and removal of old asbestos cement sheeting needs special care. Plastic sheeting is economic but often not durable. Newly developed roofing materials are bituminous felt and sisal-reinforced paper. Painting the roof for water-proofing may impart taste or colour to the collected rainwater, and should be avoided.

Ground catchment

Ground catchments are used for collecting rainwater run-off. Part of the rainfall will be lost as it serves to wet the ground, is stored in depressions, or disappears through evaporation. Laying tiles, concrete, asphalt, or plastic sheeting to form a smooth impervious surface on the ground can reduce such losses considerably. Sometimes simply compacting the surface is adequate.

The amount of rainwater that can be collected in a ground catchment depends on whether the catchment is flat or sloping, and on the water tightness of the top layer.

Through preparation of the ground surface, a sufficiently rapid flow of the water to the point of collection and storage can be assured in order to reduce evaporation and infiltration losses. The portion of rainfall that can be harvested ranges from about 10% for a pervious, flat ground catchment, to some 90% for a sloping strip catchment covered with impervious materials.

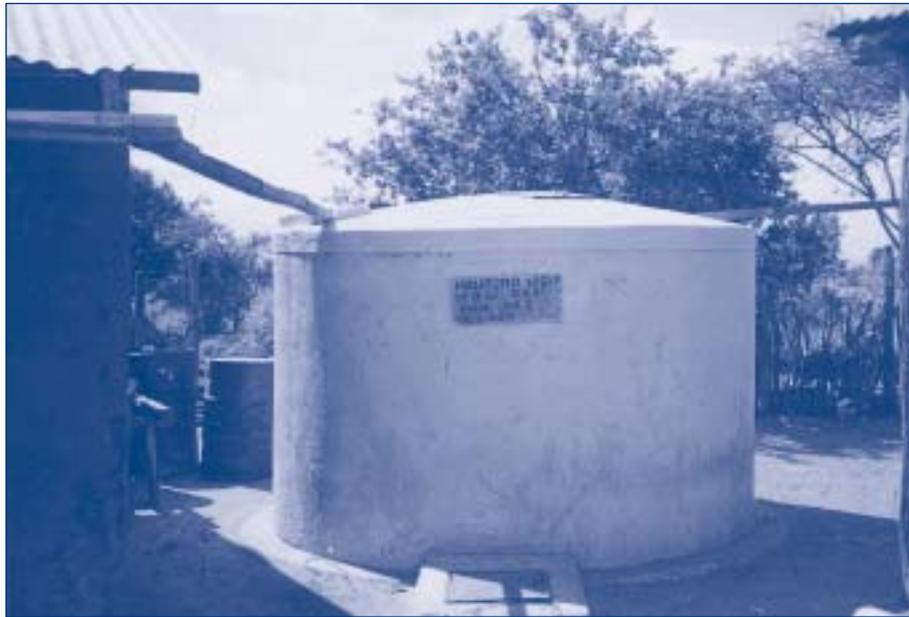


Fig. 7.1. Simple roof catchment and storage

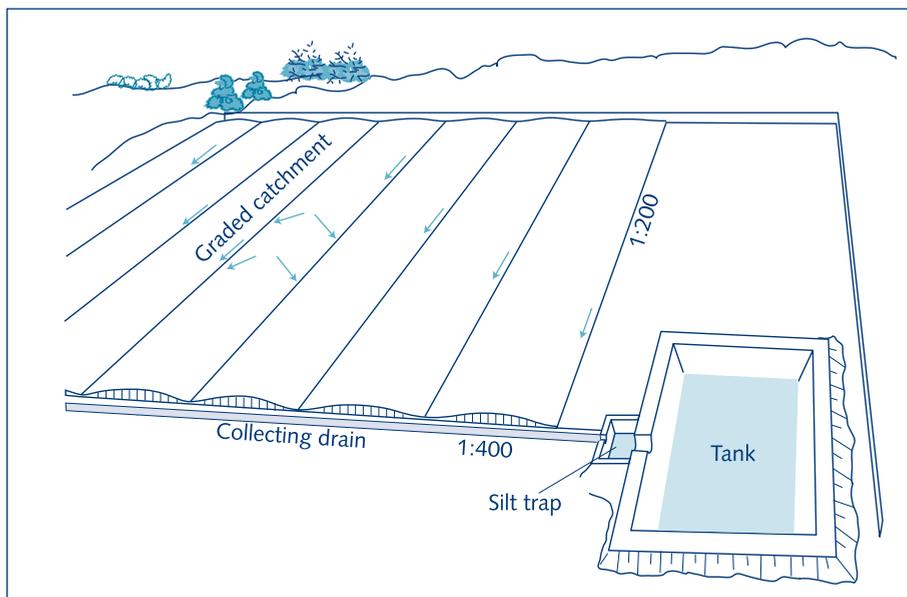


Fig. 7.2. Large ground catchment

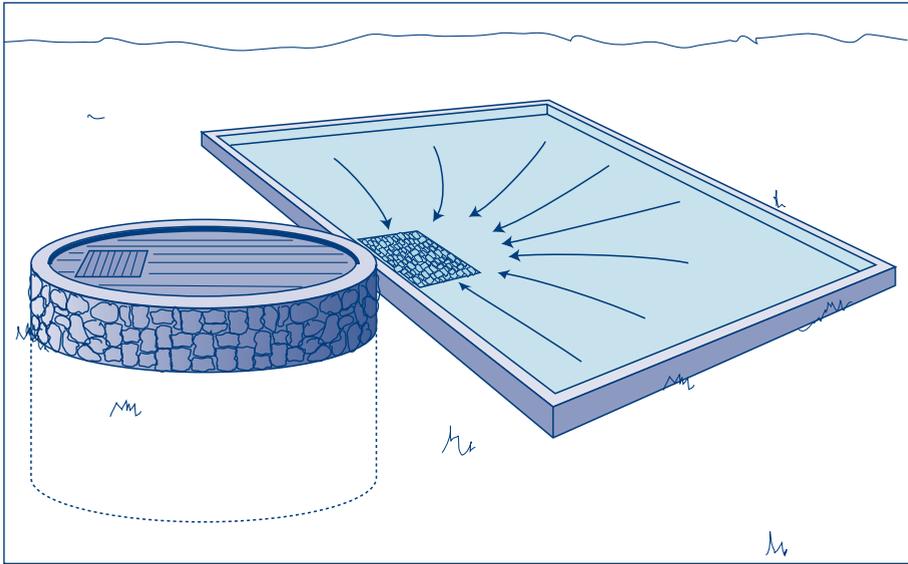


Fig. 7.3. Small ground catchment

Land alteration for run-off in surface water catchment systems includes constructing ditches along contours, clearing rocks and vegetation, and simple soil compacting. Attempts are often made to achieve reduced infiltration losses of rainwater in the ground catchment area. In rolling hills careful soil compacting may be sufficient to attain good catchment efficiency. In flat terrain, sub-division into small, sloping strips will be needed as well as appropriate preparation of the ground surface.

Rock catchment

Some places such as at the tops of bare hills with rock outcrops or low lands (e.g. around inselbergs) are suitable to form a ground catchment referred to as a rock catchment. Dips and gullies form good sites for reservoirs. Run-off water intercepted by the rock is channelled through stone gutters and bunds across contours of the slopes to reservoirs where water is stored for use. The site for the dam and the bottom of the reservoir should be devoid of rock fissures or fractures that might drain the water away from the site. From the reservoir, water may be channelled by gravity to communities living downstream. The type, size and shape of the storage facility depend on site characteristics and water demands.

Transport of rainwater

For harvesting rainwater from roofs, guttering is used to transport the water to the storage vessel. Guttering comes in a wide variety of shapes and forms, ranging from the factory type made of PVC, zinc, copper or aluminium to home-made guttering using bamboo, ferro-cement, timber or folded metal sheets. Guttering is usually fixed by hooks to the building just below the roof and catches the water as it falls from the roof. The roof guttering should slope evenly towards the down-pipe (at 0.8-1% slope)

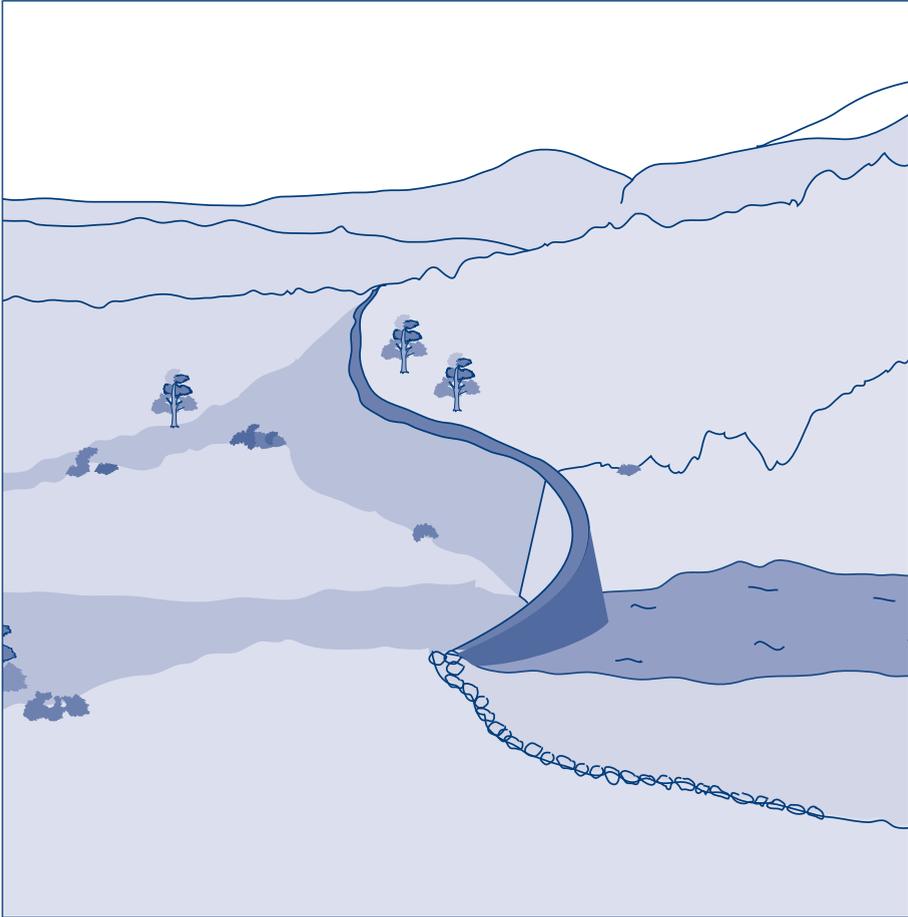


Fig. 7.4. Rock catchment

because if it sags, pools will form that can provide breeding places for mosquitoes. For the same reason, pools of stagnant water from the overflow must be avoided. Gutters and down-pipes in normal circumstances will cost between 5 and 15% of the total system cost depending on local prices and conditions.

Run-off coefficient

The run-off coefficient is the ratio of the volume of rainwater that runs off a surface to the volume of rainwater that falls on that surface (Gould and Nissen-Petersen, 1999). This includes also losses from spillage in the gutter system. A carefully designed and constructed gutter system is essential for any catchment system to operate effectively. A properly fitted and maintained gutter down-pipe system is capable of diverting more than 90% of all rainwater run-off from a good roof catchment made of corrugated iron into the storage tank, but the long-term collection efficiency is usually between 80 and 90% (Ree, 1976). Table 7.1 gives the ranges of run-off coefficients for roof and ground catchments.

Table 7.1 Run-off coefficients for selected surfaces

Roof catchments	
Sheet metal	0.8-0.9
Cement tile	0.60-0.70
Clay tile (machine made)	0.30-0.40
Clay tile (hand-made)	0.25-0.30
Ground catchments	
Concrete-lined	0.75
Cement soil mix	0.30-0.40
Buried plastic sheet	0.30-0.35
Compacted loess soil	0.10-0.20

Sources: Zhu and Liu, 1998 and others

Foul flush

Dust, dead leaves and bird droppings will accumulate on the roof during dry periods. The first new rains will wash these off. It may be helpful to arrange the down pipe so that the first water from each shower (the “foul flush”) can be diverted from the clear water container and allowed to run to waste. To safeguard the quality of the collected rainwater, the roof and guttering should be cleaned regularly. In Jamaica, for example, bird droppings have been reported to cause health hazards (salmonellosis) when harvesting rainwater from roofs. A wire mesh should be placed over the top of the down-pipe to prevent it from becoming clogged with washed off material.

The simplest way of diverting the first flush is to turn the inflow pipe away from the inlet of the storage tank, and put it back when the water is clean. This, however, requires that somebody is around at the moment the rains start.

A simple system is to collect a certain volume (about 20 litres) of first flush in a plastic or metal container (e.g. attached to the wall) (Fig. 7.5); when it is full the floating rubber ball closes off the container and “clean” rainwater flows to the storage tank. The tank can be easily drained. Another, more expensive, arrangement for diverting the first rainwater running from the roof is shown in figure 7.6. There are several other diversion systems.

Another arrangement is an underground storage tank receiving rainwater that overflows from a vessel placed above the ground (Fig. 7.7). The surface vessel thus occasionally contributes water to the underground tank.

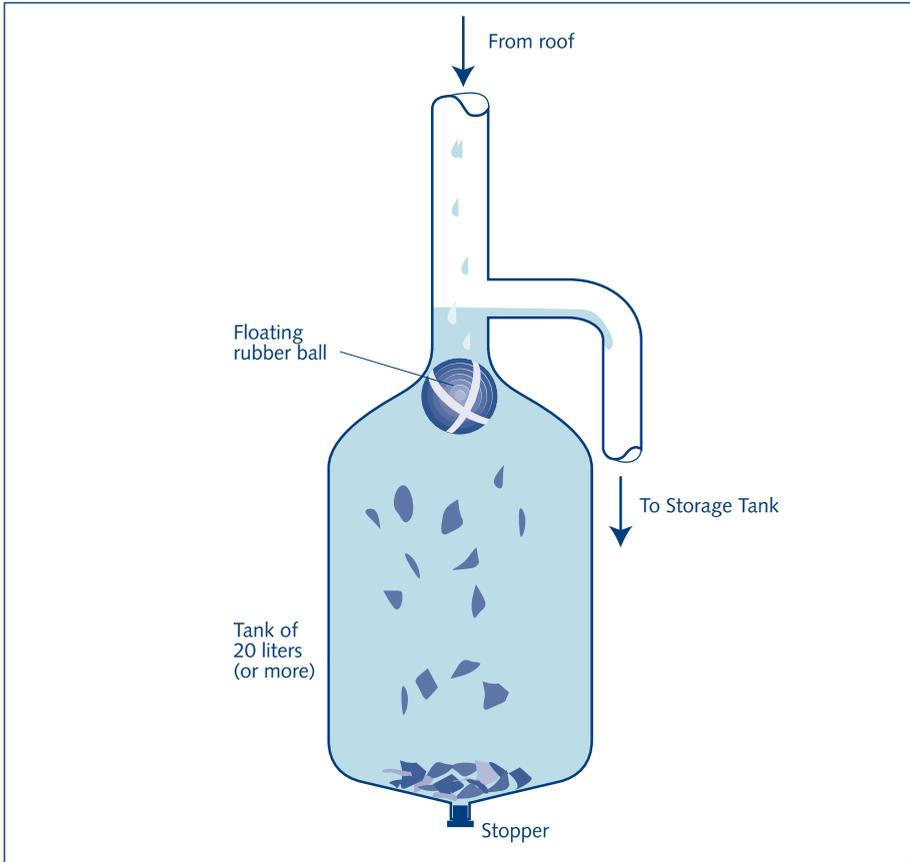


Fig. 7.5. Simple first flush diversion system.

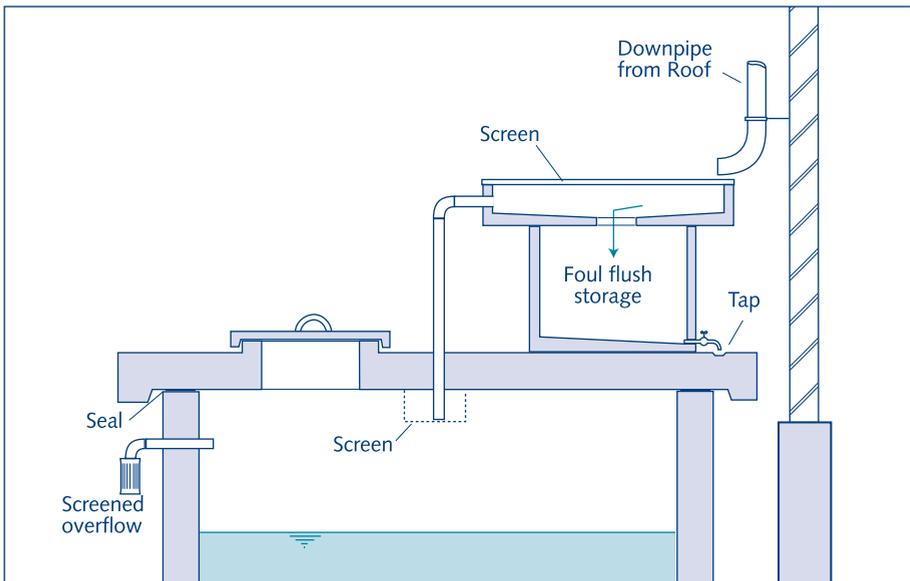


Fig. 7.6. Arrangement for diverting the first foul flush

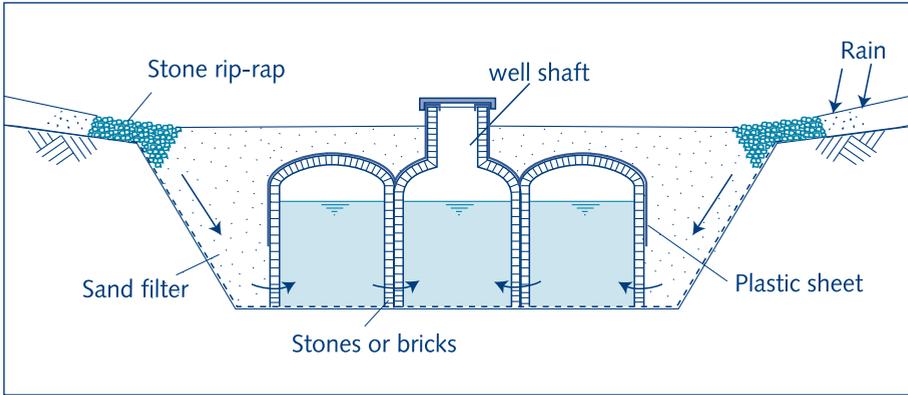


Fig. 7.7. Roof catchment and storage of rainwater (withdrawal by simple handpump)

7.3 Filtration systems and settling tanks

A wide variety of systems are available for treating water before, during and after storage. The level of sophistication also varies from extremely high tech to very simple techniques. An ingenious filter has been developed that fits into a vertical down-pipe and acts as both filter and first flush system.

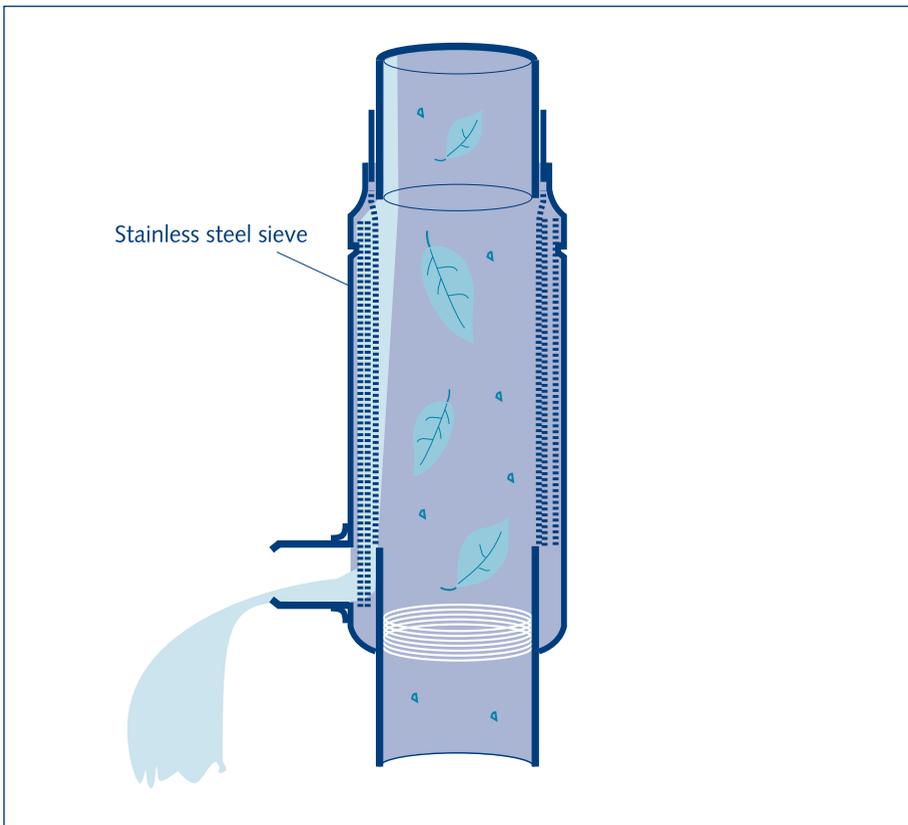


Fig. 7.8. Rainwater filter system built in down-pipe (Adapted from WISY-Catalogue, Frankfurt, Germany)

Filtration systems using sand, stone, gravel or charcoal or a combination of these as filter material filter are often used for filtering rainwater entering a tank. This type of filter is only useful where the inflow is slow, otherwise the system can overflow.

Settling tanks and partitions can be used to remove silt and other suspended solids from water. These are effective but add significant additional cost.

7.4 Storage provisions

The storage tank is usually the biggest capital investment element of a rainwater harvesting system for domestic water. It therefore requires the most careful design to provide optimal storage capacity while keeping the cost as low as possible.

There are an almost unlimited number of options for storing water. For small storage capacities in many developing countries you will find plastic bowls and buckets, jerry cans, clay or ceramic jars, cement jars, and old oil drums. For storing larger quantities of water, the system will usually require a tank or a cistern. Storage tanks are classified as (i) above-ground storage tanks and (ii) cisterns or underground storage vessels. These facilities can vary in size from one cubic metre to up to hundreds of cubic metres for large projects.

The shapes also vary: cylindrical, spherical, square, rectangular or a combination of them. The choice of materials, size and shape, of tank or cistern varies widely depending on locally available materials and technical and economic considerations.

Whichever type of storage is selected, adequate enclosure should be provided to prevent any contamination from humans or animals, leaves, dust or other pollutants entering the storage container. A tight cover should ensure dark storage conditions so as to prevent growth of algae and breeding of mosquito larvae. Open containers or storage ponds are generally unsuitable as sources of drinking water.

Below-ground storage facilities have the general advantage of being cool, and they will suffer practically no loss of water through evaporation. Provided that certain soil conditions are satisfied, underground storage tanks are substantially cheaper than above-ground tanks. There can also be a further saving in space and cost of construction where the container is moulded directly in the ground by simply compacting the earth. An example - shown in figure 7.9 - is feasible if the soil is soft, stable and fairly permeable.

In many African countries semi-spherical underground water tanks have been constructed. In this case a roofing material becomes necessary to control evaporation.

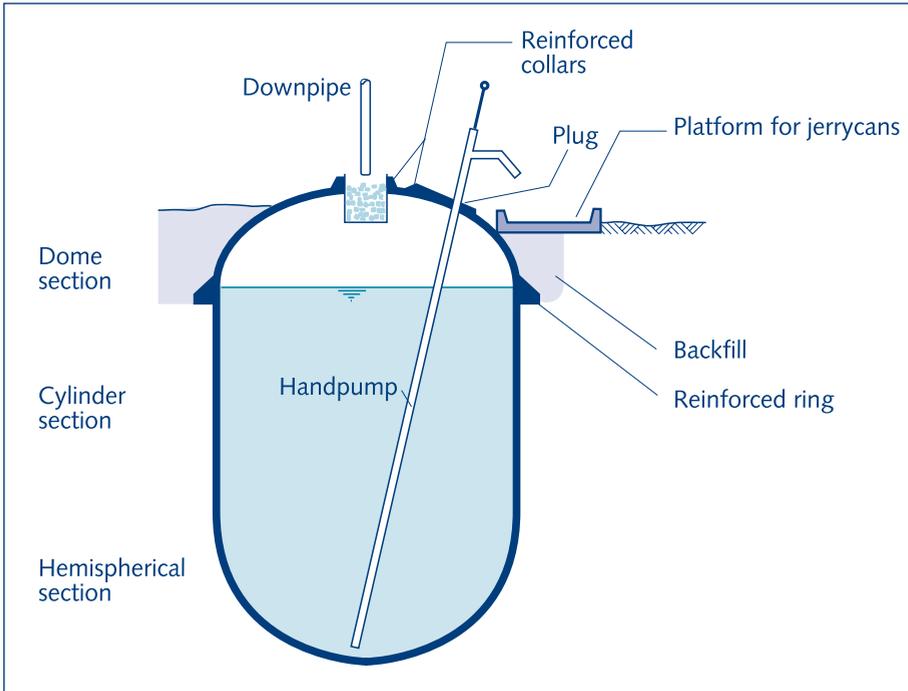


Fig. 7.9. Simple underground rainwater storage well (Adapted from Development Technology Unit, School of Engineering, University of Warwick <http://www.eng.warwick.ac.uk/DTU/pubs/wp/wp49/index.html>)

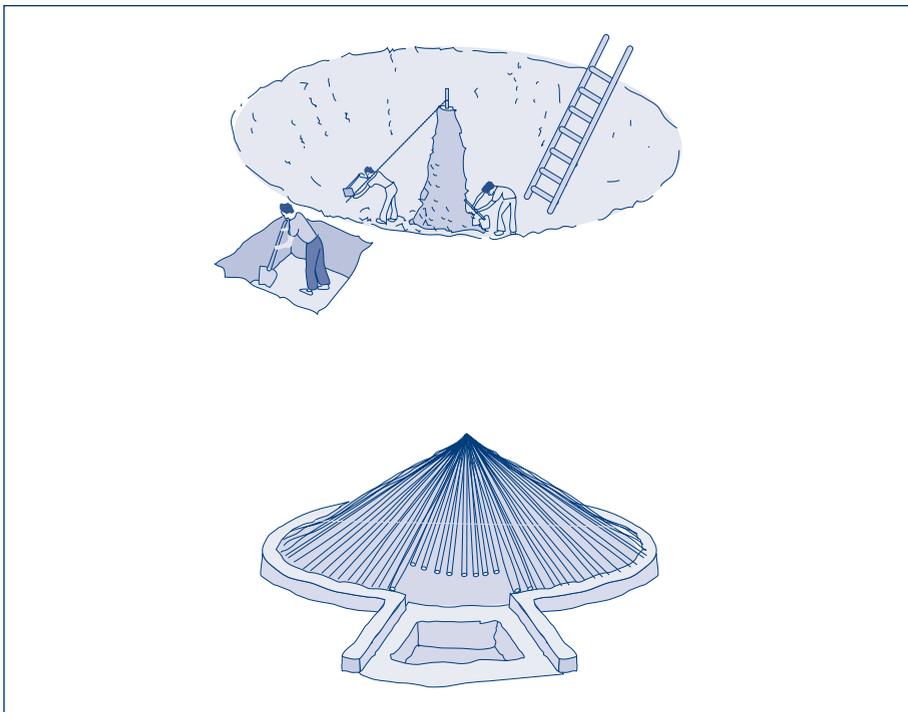


Fig. 7.10. Semi-spherical underground water tank (Adapted from UNICEF, 1986)

Materials used for storage tanks

Galvanised iron sheets were used until plastic technology came onto the market in the early 1990s. The plastic tanks have solved one major problem experienced in corrugated iron tanks, which is the rusting of the tank causing it to leak. This problem usually occurs within 3-5 years, if the tank is not cleaned after every season, or at least once a year. Other common construction materials are masonry, reinforced concrete and ferro-cement.

For larger storage volumes, tanks or cisterns constructed of brick or stone masonry are used most. Typically the walls are cylindrical and bonded by cement mix. Where large storage volumes are built, and certainly for tank heights exceeding 2 m, reinforcement along the outside edges becomes necessary. This can be conveniently provided by means of one or more tightened steel bands around the outer circumference of the tank. Placing some suitable cover (e.g. galvanised iron sheets) over a supporting framework commonly provides the roofing of this type of tank.

Increasingly popular are ferro-cement tanks in which wire is used for the reinforcement of walls and bottom that are formed by plastering cement. Such tanks are quite economical although competition comes from the plastics. The most common tank size range is between 1000 and 5000 litres. Although ferro-cement tanks are technically feasible up to 100 m³, most have a volume less than 12 m³. The construction of ferro-cement tanks requires good skills for the correct application of the reinforcement (weld mesh, chicken mesh and galvanised wire) and the correct mixing and application of the cement mortar. Cracks are easily repaired with mortar, and leaks are sealed by applying sealant on the inside of the tank.

There are a number of reasons why these materials are preferred over plastics or galvanised iron. Benefits accrue from the way building a tank is organised. Plastic tanks have to be brought from outside the village, and so local labour is minimal. With ferro-cement tanks, although the cement and reinforcement come from outside, the labour is local and benefits the village economy. Women's groups in the rural areas participate much more in a ferro-cement tank than they do in a plastic one. Future maintenance of the ferro-cement tanks can also be carried out in the village, unlike repair of a leaking plastic tank.

In some areas, like Kenya, plastic tanks cost twice as much as the ferro-cement tank per cubic metre capacity. Ferro-cement rainwater jars of 2500 litres have been very popular in Thailand and Myanmar, where millions of these tanks have been constructed. A ferro-cement jar and cistern are shown in figure 7.11. Detailed technical information on storage tanks and simple lifting devices is available from the Development Technology Unit, Warwick University (see web sites).

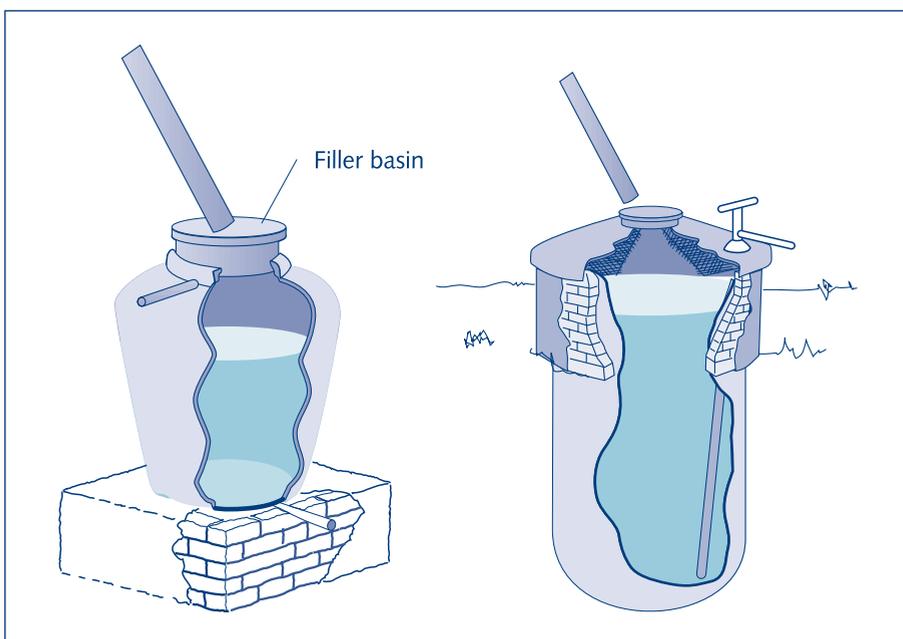


Fig. 711. (a) Ferro-cement jar and (b) partially underground tank
 (Adapted from: Ferro-Cement Jar (TR-RWH06) and Partially Below Ground tank (TR-RWH01)
 Development Technology Unit, School of Engineering, University of Warwick)

Sand dams and subsurface dams

Although sand dams and subsurface dams are mainly used for retaining groundwater flow, they are mentioned here because the water often comes from rainwater run-off. In the case of a subsurface dam, the water in the river is trapped in the sand riverbed by an underground dyke (constructed of clay, masonry or concrete). In the case of a sand dam, a low dam is built across the river (often on hard formations). The dam blocks the flow of sand (and water). The height of the sand dam can be gradually increased over the years, and more sand (and water) is retained by the sand dam. The retained water in the sand body can be collected through wells or other collection systems. More details are given in chapter 6 on artificial recharge.

Gender aspects

In many places, such as Kenya in East Africa, Kerala in India and the Fiji islands, women's groups or cooperatives construct rainwater tanks. The technology builds on women's tasks and skills in plastering and their gender needs for a nearby source of high quality drinking water. Culturally, it is also often more acceptable that women masons construct rainwater tanks in compounds when the men in the households are away during the day.

7.5 Sizing a rainwater harvesting system

Usually the main calculation when designing a water harvesting system will be to size the water tank, cistern, or subsurface dam correctly to give adequate storage capacity. The storage requirement will be determined by a number of interrelated factors. They include:

- Local rainfall data and weather patterns
- Collection surface (roof or ground) or size of reservoir possible in case of a subsurface dam
- Run-off coefficient (varies from 0.15-0.9 depending on collection surface and slope)
- User numbers and consumption rates or water needs for other uses including productive use

The following are additional rainwater harvesting features that also affect tank sizing:

- Where users are able and willing to adjust their consumption downwards during dry seasons or when they find water levels in the tank lower than average, tank size can be reduced
- Partial rainwater harvesting systems, either where it is accepted that rainwater will not meet all needs throughout the year or where rainwater is only used to meet specific water needs like cooking/drinking
- The reliability of the alternative used where the tank runs dry

Estimating domestic water demand

Estimating household annual water demand simply means multiplying mean daily water use per person by the number of household members and by 365 days. Because of the scarcity of water, people in arid and semi-arid regions usually consume very little water, far less than the official design figures of about 40 lcd. The daily water use differs by season, being less in the cooler periods than in the warmer seasons. Water consumption also depends on the availability of water. Provision of better water supply, including easier access and more water of better quality, is likely to raise the consumption per capita. Therefore, having water in a tank next to the house will increase the consumption, particularly during the rainy season when the tank will be filled up again. The current household water consumption is best assessed by a survey among households. The survey needs to include different water uses, including productive uses.

Determination of the capacity of the storage tank and the surface of the catchment

The required storage capacity depends on many factors, including domestic water demand and its seasonal variation; availability of other (seasonal) water sources (and distance, quantity and quality); characteristics of the dry season(s) (e.g. distinctive short periods without any rain); rainfall characteristics (total rainfall and intensity); and availability of catchment area, type of catchment and roofing/surface material used.

The simplest method to calculate the required water volume is to use the following formula:

$$V = (t \times n \times q)$$

Where:

V = volume of tank

t = number of days in the dry period

n = number of people using the tank

q = total average consumption per capita per day (including for productive use)

To be on the safe side, a 20% increase of tank volume is suggested.

If there are no reliable monthly rainfall data, the tank volume can be established by assuming that the tank will be full at the start of the dry season and only has to satisfy the water needs for the dry period.

The required roof catchment can be calculated from the expected annual water demand to be met by rainwater (including for productive use but also considering other water sources).

Annual consumption is $Q_a = n \times q \times 365$

Required roof catchment is $A = Q_a / (C_r \times p)$

where:

C_r = run-off coefficient

p = annual precipitation

Another simple method that can be used to estimate the most appropriate storage tank capacity for maximising supply is to represent roof run-off and daily water consumption graphically. (Gould and Nissen-Petersen, 1999). This method will give a reasonable estimation of the storage requirement, but daily or weekly data should be used for a more accurate assessment, especially for climates with year-round rainfall.

Other methods are mass curve analysis, statistical methods and computer-based methods. They all need accurate rainfall data. Rainwater catchment models need reliable data otherwise the results from the model (e.g. from computer-based methods) are unreliable. But if correctly applied, computer-based models may give optimum cost-effective designs (see relevant web sites listed at the end of the chapter).

7.6 Health and water quality

Health benefits

Rainwater harvesting has several benefits for family health:

- Women and other drawers of water spend less time collecting water and so save time for household tasks, income-generating activities, and rest, all of which have health benefits for them and for the whole family
- Fewer accidents to unattended infants because water is available at the home yard
- Better nutritional status of the family because saved time can go to keeping a vegetable garden (also using the stored rainwater if there is enough)
- Fewer female and child back injuries resulting from long and steep journeys with heavy water loads
- Health benefits arising from use of more water, and clean and safe water
- More water available for productive use that has an effect on the general well-being of the family, including health

Quality concerns

There have been concerns that rain may pick up unhealthy substances whilst falling through the atmosphere, running down a roof or whilst resting in a storage tank. The probability of finding truly airborne indigestible pathogenic viruses or bacteria seems low and of finding larger airborne pathogens negligible. Interest therefore focuses mainly upon contamination from roofs and the influence of water storage on reducing or increasing pathogens.

Contamination of water might arise from the roofing material itself or from substances that have accumulated on a roof or in a gutter. Roof paints including bitumen may entail some risk to health and/or may impart an unpleasant taste to roof water and should be avoided. Not surprisingly the safety of water harvested from “asbestos” (asbestos-reinforced cement mortar) roofs has been queried, but the consensus is that the danger of developing cancer from ingested asbestos is very slight (Campbell, 1993). The danger from inhaled asbestos dust is, however, so high that working with asbestos sheeting, for example sawing it, without special protection is now generally banned in industrialised countries. The iron in a rusting roof will also enter the run-off, but in such small quantities that it does not prejudice either health or taste.

Metal roofs are comparatively smooth and are therefore less prone to contamination by dust, leaves, bird droppings, and other debris than rougher tile roofs. They may also get hot enough to sterilise themselves. However, contamination may be substantial on all roof types and it has been common for many years to design *first-flush diverts* into rainwater harvesting systems. During a dry spell, debris builds up on roofs, so that the initial run-off during the first following rainfall can be full of sediment and highly turbid.

Overhanging trees make this sediment problem worse, as well as increasing the likelihood of bird and rodent droppings. A common strategy therefore is to divert to waste the first, say, 20 litres of run-off at the beginning of each rainfall. This can be done automatically using appropriate devices, of which some are commercially available (see section 7.2 and 7.3). Where the rainy seasons are well defined, it can be done manually by temporarily displacing the down-pipe. If this first flush is excluded, rainwater gives modest levels of turbidity and typically medium levels of bacterial contamination (e.g. <10 FC per 100 ml). Modern “no-maintenance” separators, or more traditional screens, cloth or sand filters will reduce turbidity and contamination further and any good tank design will reduce it further still.

7.7 Integrated and holistic nature of rainwater harvesting

Watershed approach

Run-off arising from uncontrolled rainwater is the greatest agent of soil erosion, which subsequently gives rise to other forms of land degradation. This is especially so in arid and semi-arid lands where the rainfall is erratic and the soil fragile. In order to sustain the productive natural resource base of the areas concerned, controlling and conserving water from run-off must conserve the environment. The amount of water resulting from rain on a bare soil is about 50% of the rainfall. Water harvesting therefore serves the dual purpose of preserving the environment and providing water which can also be used for productive purposes.

Groundwater recharge

Properly managed rainwater is essential for groundwater recharge. Good soil and water conservation and agro-forestry practices facilitate infiltration of rainwater to recharge groundwater. The upper ten metres or so of groundwater is shallow groundwater and is the main source of water for trees, hand-dug wells, subsurface dams and shallow boreholes. More details can be found in chapter 6, “Artificial recharge”.

Other activities related to rainwater harvesting

Run-off farming, micro-irrigation and flood diversion are methods of putting rainwater to good use whenever it falls or immediately afterwards. All these activities are captured in a new approach to land rehabilitation and people's development called watershed management. See chapter 5, “Integrated Water Resources Management”.

7.8 Conclusions and recommendations

Organisation and development

The main goal of watershed management that includes rainwater catchment is to hold the water where it falls as precipitation, and to protect the land from erosion at the same time. In this way water becomes available for domestic and productive uses with many benefits that significantly contribute to poverty alleviation in rural areas and small towns. For individual households it is often not possible to work on the improvement of their water situation for domestic and productive use. But by grouping in grassroots groups in which members continue to work together and support each other after the NGO or government department has left, initiatives around rainwater catchments become very feasible and sustainable. Hundreds of projects have shown that to be true. But there are certain points, such as leadership, trust, ownership, etc., that need to be seriously addressed. Strong institutional bedding of the community-based organisation and the supporting organisations is needed. More details on a participatory approach are in chapter 2 on planning and management.

Finance

Although the initial capital costs of rainwater harvesting may be high, it yields benefits whose value may exceed the cost of the system. Rainwater harvesting systems are suitable for an incremental approach. If used for income generating activities, the rainwater harvesting system pays back within 2-4 years; returns can be partly re-invested. Nevertheless, small subsidies for rainwater harvesting are cost effective, as they make smaller systems more feasible. Direct funding to community groups is efficient, provided there is an effective and transparent backup system provided by a trusted NGO or consultant. Micro-financing should be considered for small rainwater-based enterprises.

Housing policy

In view of the seriousness of the water and food crisis in both rural and urban areas, governments should seriously consider including domestic rainwater harvesting systems in all new public housing projects and advocating (and subsidising) these systems in private housing. Rainwater harvesting contributes also to a reduced demand from the municipal piped water supply.

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Web sites

Rainwater research and publications: <http://www.eng.warwick.ac.uk/DTU/pubs/rwh.html>

Garnet: <http://www.lboro.ac.uk/departments/cv/wedc/garnet/tncrain.html>

IRCSA (International Rainwater Harvesting Systems Association):
<http://ms2.pccu.edu.tw/~98710704/>

CSE (Centre for Science and Environment): <http://www.cseindia.org/>

Simtanka - free software for simulating performance of rainwater harvesting systems:
<http://www.geocities.com/RainForest/Canopy/4805/>

Discussion groups

Discussion forum on rainwater: <mailto://rwh-request@jiscmail.ac.uk> or
<http://info.lut.ac.uk/departments/cv/wedc/garnet/rwheconf.html>

8 Spring water tapping

Andrew Tayong

8 Spring water tapping

8.1 Introduction

Springs are found mainly in mountainous or hilly terrain. A spring may be defined as a place where a natural outflow of groundwater occurs.

Spring water is usually fed from a sand or gravel water-bearing soil formation called an aquifer, or a water flow through fissured rock. Where solid or clay layers block the underground flow of water, it is forced upwards to the surface. The water may emerge either in the open as a spring, or invisibly as an outflow into a river, stream, lake or the sea (Fig. 8.1). Where the water emerges in the form of a spring, it can easily be tapped. The oldest community water supplies were, in fact, often based on springs and they remain a favoured source, because the water usually has a high natural quality and intake arrangements are relatively straightforward. That suits both the engineers helping to design the water supply system, and the community members who will have to look after it. Because of their popularity, most natural springs have been developed in one way or another as drinking water sources. However, a proper feasibility study, application of some basic design principles and vigilance in protecting the spring and its catchment area will usually lead to improvements in the quantity, quality and sustainability of many such supplies. As in the rest of the book, there is an overriding principle that community members should be fully informed and closely involved in decisions about the tapping, use and protection of spring water sources.

A key reference on spring catchment is the SKAT manual *Spring Catchment* of 2001.

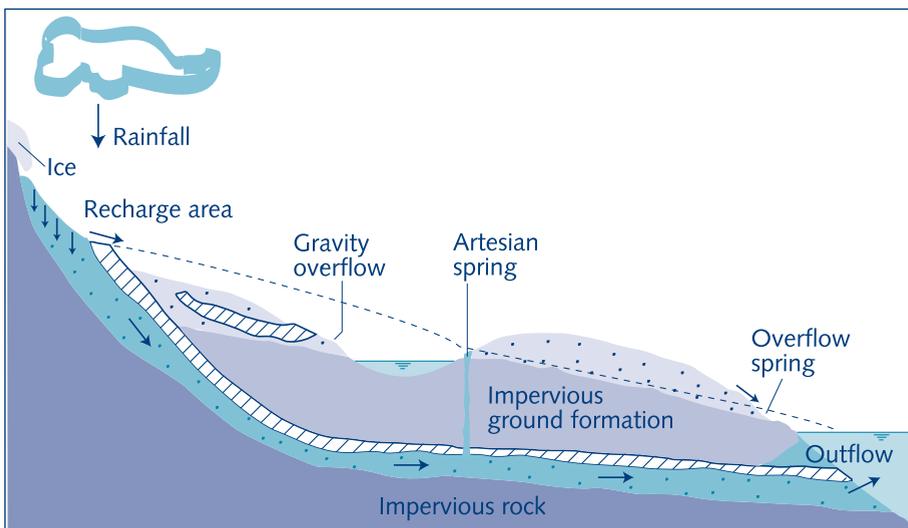


Fig. 8.1. Occurrence of springs

8.2 Identification of spring source

Local people, especially women (as drawers of water), but also farmers, hunters and grazers, have a good knowledge of the location of springs and their characteristics. These people are the primary sources of information in the identification process. In the dry season, green vegetation in a dry area may also be an indication of a spring source.

Some springs form small ponds where animals drink and people may well also scoop water from there. Others flow as small streams in valleys and can be traced back to the source. The source, though, is not necessarily the first upstream point at which the stream emerges from the ground. In some cases streams may be buried for quite a length and there can be added risks of contamination unless the investigation continues further upstream to locate the true spring.

8.3 Types of spring sources

Springs are classified according to the conditions under which water flows to them. Some surface under pressure, while others do so as a result of discontinuities in the strata that held the water underground. For instance, in a seepage or filtration spring the water percolates from many small openings in porous ground, while in fracture springs the water comes from joints or fractures in otherwise solid rock, and for tubular springs the outflow opening is more or less round. To understand the possibilities of water tapping from springs, the distinction between gravity springs and artesian springs is most important. A further sub-division can be made into depression springs and overflow springs.

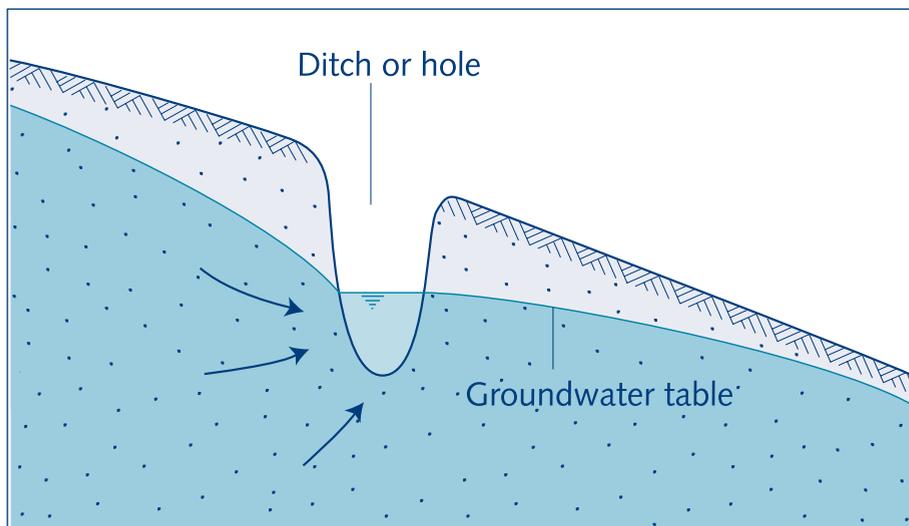


Fig. 8.2. Gravity depression spring

Gravity depression springs

Gravity springs occur in unconfined aquifers. Where the ground surface dips below the water table, any such depression will be filled with water (Fig. 8.2).

Gravity depression springs usually have a small yield and a further reduction occurs when dry season conditions or nearby groundwater withdrawals result in the lowering of the groundwater table.

Gravity overflow springs

A larger and less variable yield from gravity springs is obtained where an outcrop of impervious soil, such as a solid or clay fault zone, prevents the downward flow of the groundwater and forces it up to the surface (Fig. 8.3). At such an overflow spring, all the water from the recharge area is discharged. The flow will be much more regular than the recharge by rainfall. Even so, an appreciable fluctuation of the discharge may occur and in periods of drought some springs may cease to flow completely.

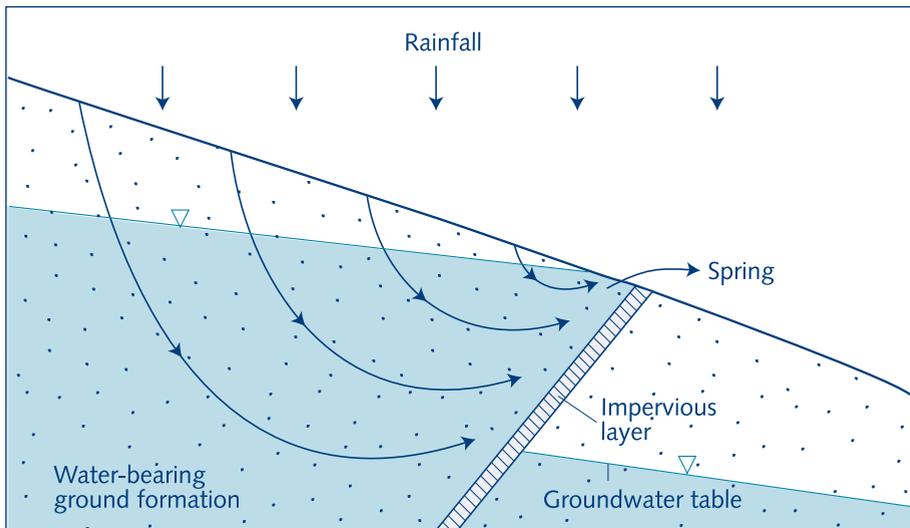


Fig. 8.3. Gravity overflow spring

Artesian depression spring

Artesian groundwater is prevented from rising to its free water table level by the presence of an overlying impervious layer. That is the reason why artesian groundwater is under pressure. Artesian springs are the sites where the groundwater comes to the surface.

Artesian depression springs are similar in appearance to gravity depression springs. However, the water is forced out under pressure so that the discharge is higher and there is less fluctuation. A drop of the artesian water table during dry periods has little influence on the artesian groundwater flow (Fig. 8.4).

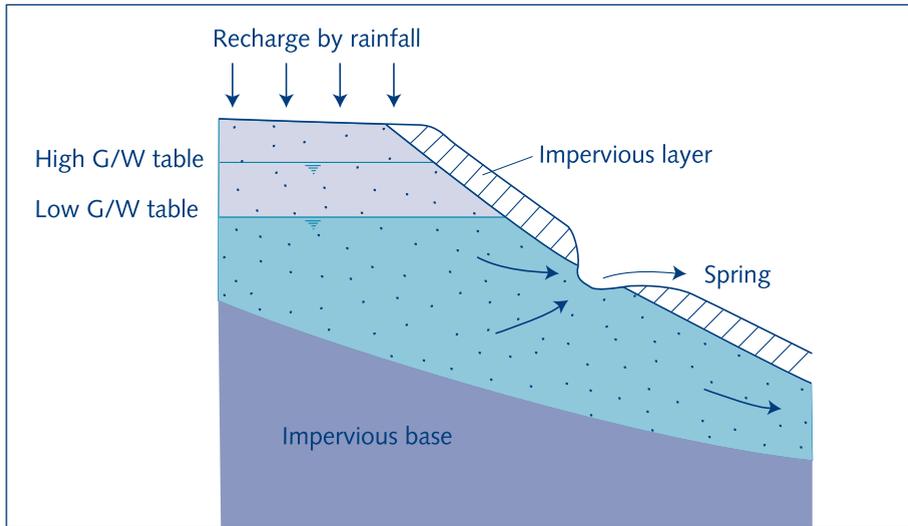


Fig. 8.4. Artesian depression spring

Artesian fissure spring

Artesian fissure springs (Fig. 8.5) form an important variant of this type of spring. Again the water emerges under pressure, this time through a fissure in the impervious overburden. Fissure springs exist in many countries and are widely used for community water supplies.

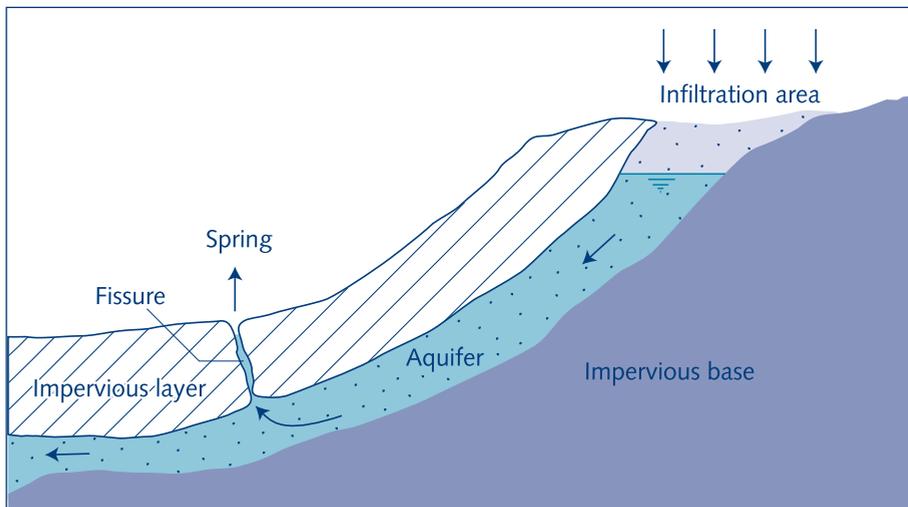


Fig. 8.5. Artesian fissure spring

Artesian overflow spring

Artesian overflow springs often have a large recharge area, sometimes a great distance away (Fig. 8.6). The water is forced out under pressure; the discharge is often considerable and shows little or no seasonal fluctuation. These springs are very well

suited for community water supply purposes. Artesian springs have the advantage that the impervious cover protects the water in the aquifer against contamination. The water from these springs is usually bacteriologically safe.

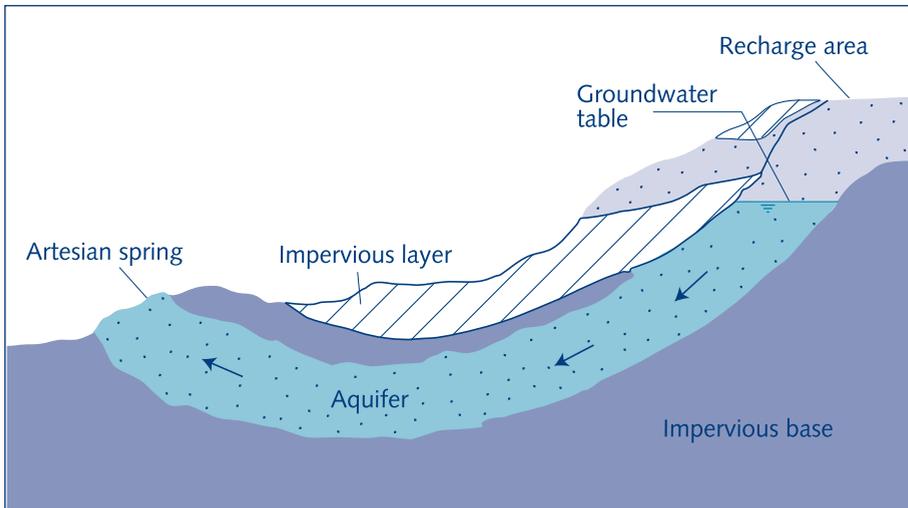


Fig. 8.6. Artesian overflow spring

8.4 Feasibility study

The feasibility study of a spring source aims to investigate the quality and the quantity of water from the source. It provides the information and data for design of a water supply system. The study also needs to take any possible environmental impact into consideration. Local people are important sources of information and should be involved in decisions about the feasibility of developing any particular spring. Aspects to be considered include the quantity and quality of the water, its reliability, current and future uses, and socio-cultural acceptability for a domestic water supply. A representative feasibility study is therefore best done together with a team of community members.

Rapid environmental assessment

A rapid assessment of potential environmental impact is a sensible first step. This involves identifying possible environmental consequences of developing a spring. These can include risks of landslides, erosion, or contamination of the source. The environmental assessment includes investigating the flow direction of surface run-off above the spring; human activities and water uses in the catchment area, i.e. habitation, farming, grazing, etc.; and the type of plants growing in the catchment or recharge area.

If there are people living in the catchment/recharge area, they are likely to contaminate the groundwater through their own waste and their activities such as cattle holding or

agriculture using artificial fertilizer or chemicals. But it may be very difficult to relocate them. If the groundwater contamination risks are too high, then such locations are not suitable. Some trees and plants are undesirable too. Eucalyptus trees, for instance, compete for water with the spring and can significantly reduce the yield. Raffia palms, though harmless, increase the iron content of the water, changing its taste and colour enough to deter consumers. If these kinds of plants are around, the best solution is to make the community members aware of the repercussions and hold discussions about moving or removing them.

Spring water quality

In general, spring water is of good quality. Pathogenic contamination is unlikely if the source meets certain criteria. These include the thickness of the soil layer, the type of soil and the velocity of infiltration of the surface water. The soil formation should be thick enough for natural filtration and biological action to remove pathogenic organisms before the water enters the aquifer feeding the spring. The type of soil determines the speed of the flow through the voids in the soil and so influences the purification mechanisms and the concentration of suspended solids. If the soil layer is not thick enough, any human activity should be restricted or even forbidden in the catchment area. Otherwise, local farmers may be allowed to conduct some agricultural activities in the catchment area (but outside the protection area around the spring) under some restrictions such as no use of artificial fertilizers or harmful chemicals. This may contribute to the protection of the catchment because they have a direct interest in protecting the area for their crop and their water supply.

One of the key signs of a good spring is that the water maintains a constant temperature throughout the day. This temperature is just below the average air temperature. The water should also be colourless. Variation of water temperature during the day and colouration of water shortly after rains are indications of a poor quality spring source. It may have its water-bearing soil layer not deep enough or rapid infiltration of surface water through the topsoil.

If users note these temperature and colour changes, but they still favour the spring over alternative water sources, then the community needs to be advised to include water treatment as part of the water supply system. This treatment may be incorporated in the intake chamber or outside the spring. For small communities, household-based water treatment may be the most feasible option.

Laboratory testing of the water quality is a necessary part of the feasibility study even if there are no changes in the water temperature and colour. Testing should extend over a reasonable period of time, especially if there are human activities in the recharge area of the source.

The spring water quantity

The quantity of water a spring produces is known as its yield. Information about the yield is crucial in the decision-making process for the tapping of a spring. Yield is studied in terms of flow rate and consistency. Variation in the yield of a spring during the dry season and the rainy season is an important criterion to determine whether the spring is a suitable source. If the ratio between the highest yield in the rainy season and the yield in the dry season is below 20, then the spring has an acceptable consistency and can be regarded as a reliable source in both wet and dry seasons. Take into account that the highest and the lowest yield do not occur at the beginning of the rainy season and at the end of the dry season but typically a couple of weeks (or even months) later, depending on the soil characteristics.

It follows that a proper feasibility study of a spring source should last for at least one year. A longer duration is preferred as there may be dry and wet years. The study will indicate the variation in yield of the spring throughout the year, and the maximum, minimum and estimated average flow. The average yield will reveal if more than one spring is needed to meet the daily water demand of the user population. If the overall yield is limited, users should be guided towards using the spring water primarily for drinking and cooking because of quality, and secondarily for other domestic purposes. A participatory study increases the people's sense of ownership and the appreciation for proper water management.

The yield is highly influenced by the water storage capacity of the aquifer. When the water velocity in the saturated stratum gets too high, the pores through which the water passes tend to become choked so that the flow becomes considerably reduced. Limestone and volcanic rock areas do not experience this effect.

In many cases, more than one spring is tapped for the same water supply system. Ideally, this should be done in a phased manner, with the community water committee judging in advance when augmentation is going to be necessary. As the population and/or its productive activities increase, daily water demand also increases, resulting in potential water shortages in the system. Shortages will occur first during dry seasons and a few weeks or months into the rainy season. They can be prevented by proper planning with the community water committee.

There may not always be time for a long study of spring yield. Water supplies are key development indicators used by politicians in rural areas for campaign purposes. This political influence sometimes does not allow the technician and the community to study the source for the required duration. Under such influences, critical study periods are used to give a rapid estimation of the yield. The best time to use is the transition period of dry and rainy seasons when the flow is minimal. The times of peak and minimum

spring yield do not necessarily correspond to the peak and minimum rainfall periods. In fact, the lowest spring yield usually occurs about a few weeks to several months into the rainy season. So, if the study terminates at the end of the dry season, the lowest yield level of the spring may not be noticed. The "critical" study should extend for about four months into the rainy season. The chances of recording peak yield still remain unlikely, but that is less critical than the minimum yield.

Estimating spring yield

A spring yield is measured in litres per second (l/s). The measurement process involves two selected trained villagers who measure the discharge from the spring over the study period. The process starts with the construction of an earth dam. Spring water retained by the dam is drained through a pipe. One villager collects the water with a container of a known volume while the other measures the time needed to fill the container (Fig. 8.7). The pipe diameter and container size are chosen such that the water outflow will not fill the measuring container in less than five seconds. Sometimes several pipes are used. Four readings are taken during the day and day averages are calculated, expressing the discharge in l/s. This is repeated once every week for the measuring period. In this way, the minimum and maximum yields are determined.

Studies of catchments with many springs channelled into a single supply point must be carried out carefully for reasons of back-pressure effects. Simply adding the yields from each individual spring together is not enough. An excessive flow could build up a back pressure and cause some springs to divert their courses. In some cases it may lead to permanent damage to the catchment. The designer needs to study the flow characteristics of any collection chambers or pipes and ensure that each spring outlet flows freely. The yield of springs may vary from a few litres per minute to several thousands per minute.



Fig. 8.7. Measuring spring yield. Photo by Andrew Tayong

8.5 Design and construction

The design and construction of a spring-fed water supply for a specific location must (i) be appropriate for the specific local conditions, (ii) prevent pathogenic contamination and pollution, (iii) be reliable in terms of quantity, and (iv) have no adverse environmental consequences.

Because of their small yield and the difficulty of providing adequate sanitary protection, gravity depression springs (Fig. 8.2) cannot be recommended for community water supplies. The presence of such a spring, however, indicates shallow groundwater that may be withdrawn using drains or dug wells. These can be covered and protected against contamination.

Design

The major components in the design of a spring-source water supply system include the actual spring water collection area – where water from the aquifer is actually being channelled to a single discharge point – the supply pipe, the collection chamber, and the outlet to a storage tank (Fig. 8.8).

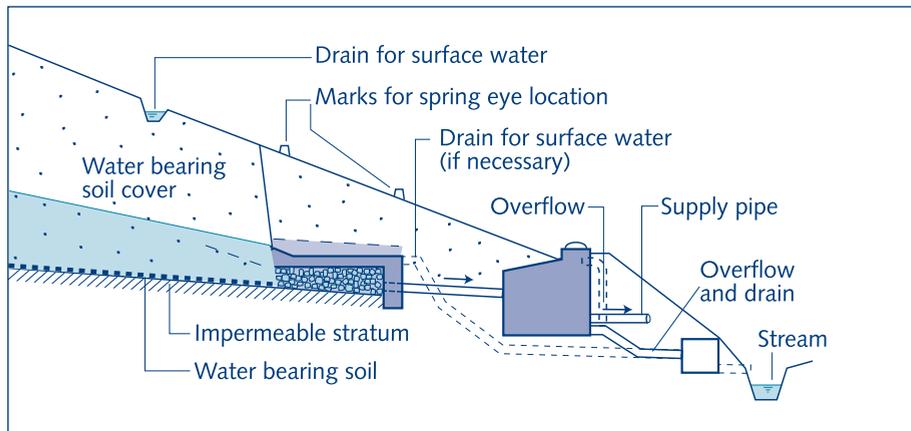


Fig. 8.8. Lay-out of a spring water collection area (Adapted from Helvetas Cameroon, 1985)

The collection area is a critical part and involves the tapping of water from the aquifer. Two methods are used to collect the groundwater. One is by dry stone masonry and the other is by perforated pipes (Fig. 8.9).

Experience has shown that roots may grow in the collection area. The roots can become so dense that they obstruct the flow, thereby reducing output from the source and also generating back pressure that could cause damage to the aquifer hydraulics. Easy access to the collection area enables the caretaker to remove the obstructing roots.

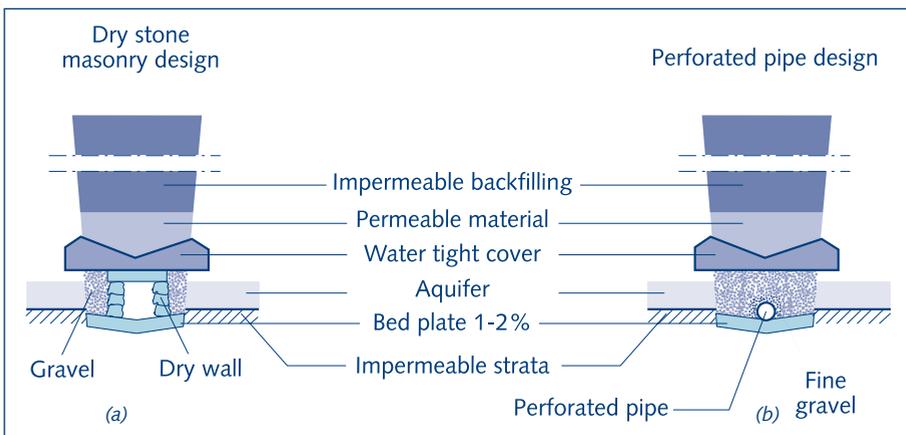


Fig. 8.9. Types of water tapping methods: (a) dry stone masonry, (b) perforated pipes
Source: Helvetas Cameroon, 1985

Other modifications in the design are made mainly to reduce costs and make the system more appropriate (fig 8.10).

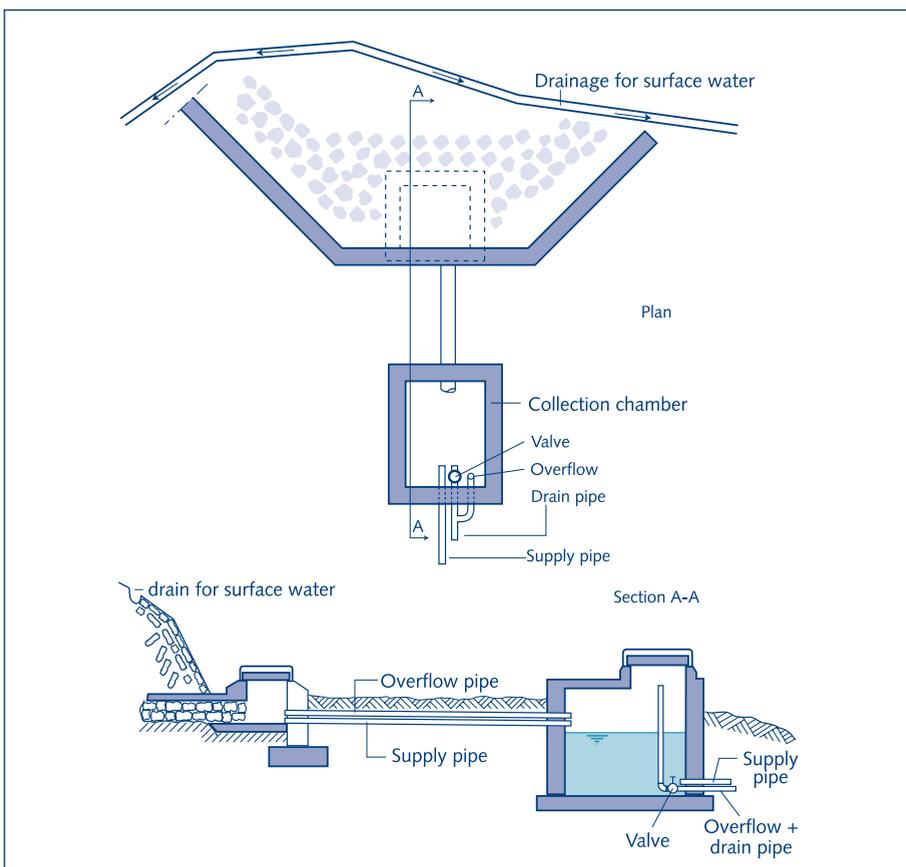


Fig. 8.10. Low cost option for collection area
Source: Helvetas Cameroon, 1985

Two supply pipes are used in the collection area to channel the drain water to the collection chamber. The first pipe channels all the discharge during times of low yield. The second ensures that there is no excess water backing up in the collection area during maximum yields, as this could obstruct the natural flow in the aquifer and create back pressure.

Tapping of gravity springs

The spring collection area is the heart of a water supply system that uses a spring source. Once constructed and backfilled, access to collection area, for instance to correct errors, is very cumbersome. Thus, care and experience is needed for proper spring construction. Figure 8.11 illustrates a detailed view of a spring collection area design.

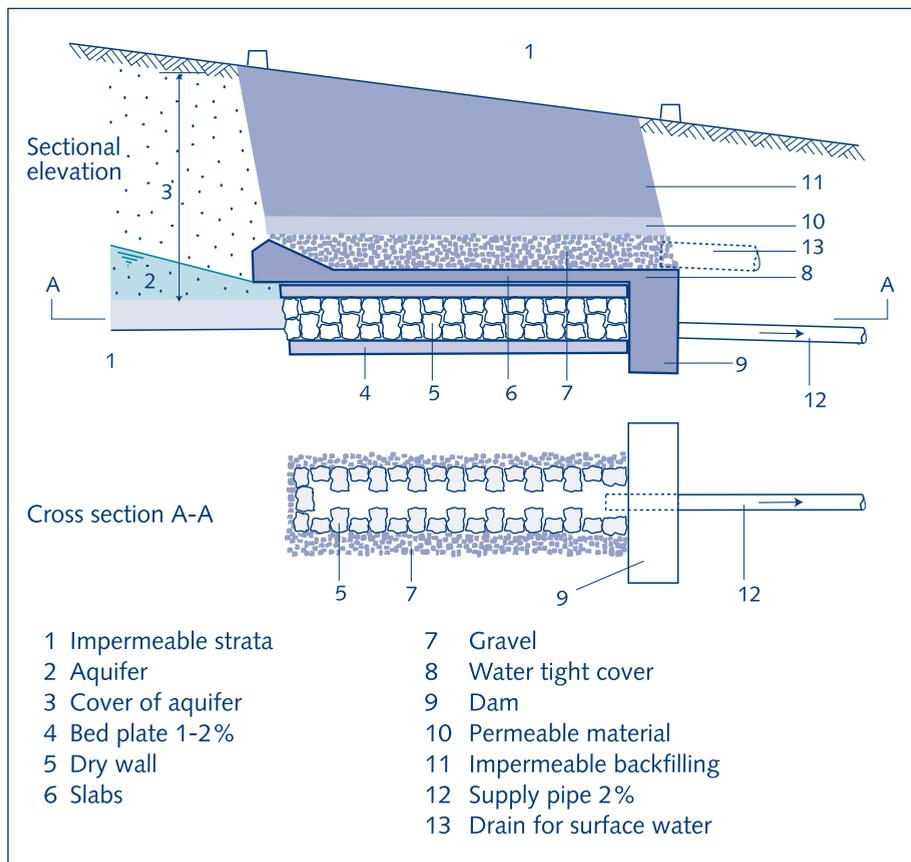


Fig. 8.11. Detailed design of a spring collection area
Source: Helvetas Cameroon, 1985

The major parts of the construction are the permeable construction and the barrage (dam). The permeable construction is a package of filter material made of rocks, stones and gravel that allows water to drain into the supply pipes. Perforated pipes surrounded by a gravel filter package are sometimes used instead of dry stone masonry but the

latter is preferable. The barrage can be a concrete dam or a stone masonry construction controlling the drain and directing water into the supply pipes. It also carries the load of the backfilling. The floor of the permeable construction and the perforated pipes slopes at about 2%.

Construction of the collection chamber should take into consideration that it acts as a sedimentation tank as well. Thus, access must be provided for regular cleaning.

Construction starts with excavation from where the spring emerges to the surface. This is done carefully to avoid disturbing the natural flow of water from the spring. There should be regular drainage during the digging. Decreasing flow during the digging is an indication of different springs from different directions. In such a case, digging should be extended to catch all the springs (Fig. 8.12).

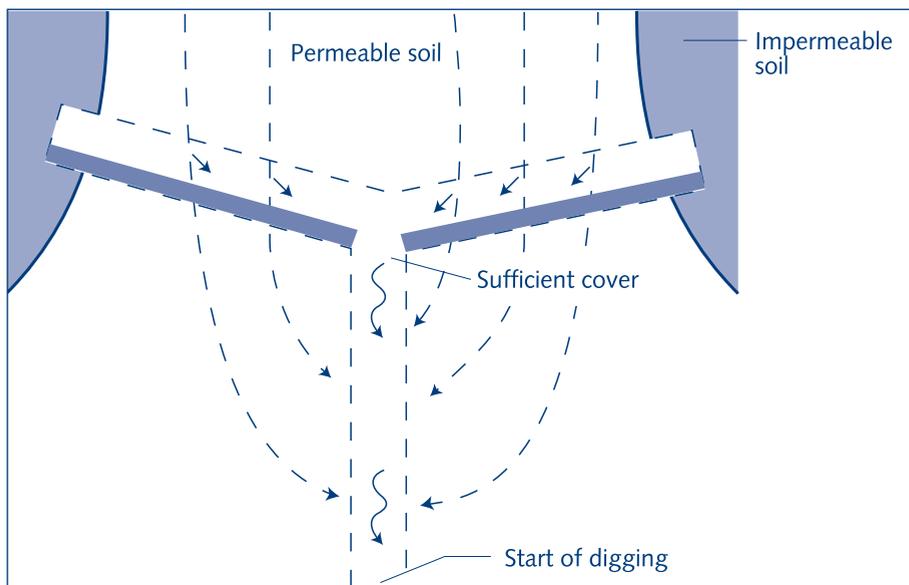


Fig. 8.12. Excavation of gravity spring source
Source: Helvetas Cameroon, 1985

Tapping artesian springs

In outward appearance, artesian depression springs are quite similar to gravity depression springs but their yield is greater and less fluctuating, as the water is forced out under pressure. Excavation and construction of artesian springs are as shown in figure 8.13.



Fig. 8.13. Excavation and construction of artesian springs
Source: Meuli and Wehrle, 2001

To tap water from an artesian depression spring, a wall extending a little above the maximum level to which the water rises under static conditions should surround the seepage area. For sanitary protection the spring collection area or spring "eye" should be covered (Fig. 8.14).

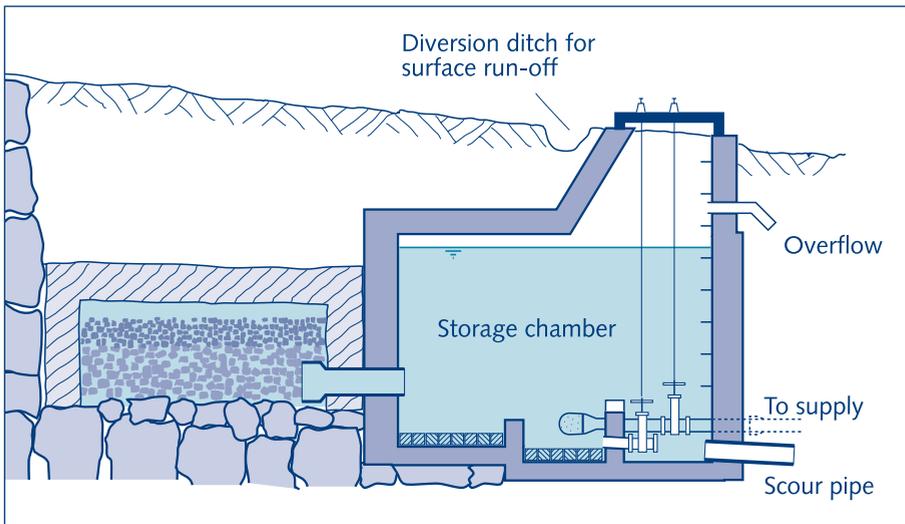


Fig. 8.14. Artesian depression spring

For artesian depression springs that cover a large area, a system of drains is commonly used to channel the collected water into a storage chamber. From there it flows into the water supply system. To protect the water quality, the recharge area should be cleared and kept clear of all debris. For granular top layers, it may be necessary to cover the recharge area with layers of graded gravel to trap fine suspended solids.

Fissure springs are similar to artesian depression springs, but the water rises from a single opening, so that the intake works can be small (Fig. 8.15). Some increase in capacity may be obtained by removing obstacles from the mouth of the spring or by enlarging the outflow opening (Fig. 8.16). The localised outflow of water from the spring makes sanitary protection easy to arrange.

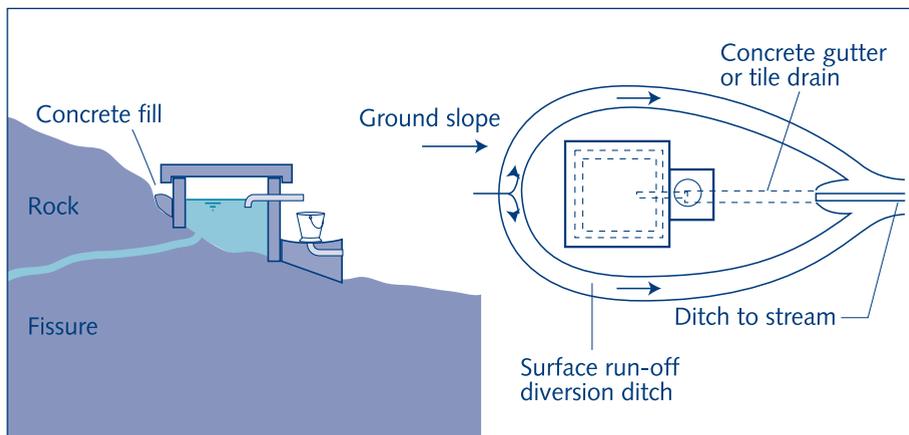


Fig. 8.15. Fissure spring of small capacity

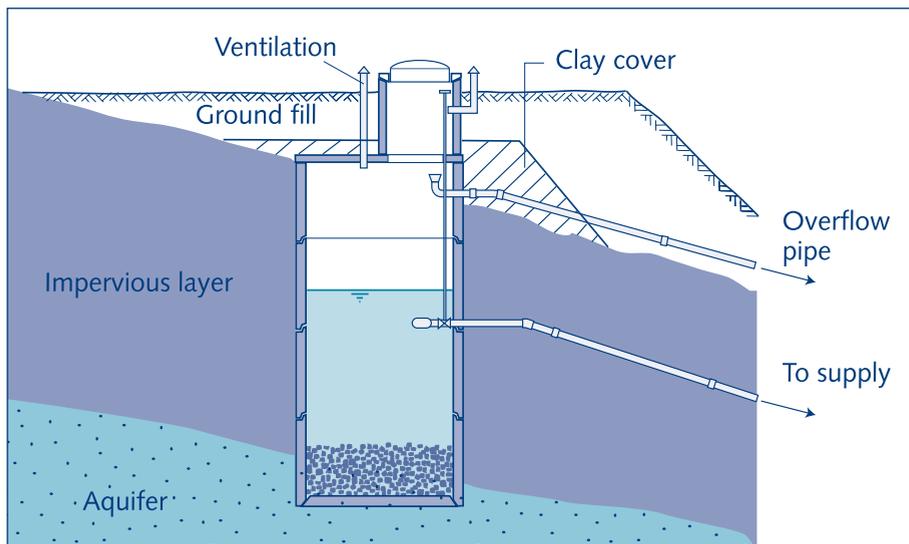


Fig. 8.16. Fissure spring of large capacity

Where the outflow of water occurs at only one point, the spring water can be tapped in a small intake construction. For a large lateral spring, a retaining wall should be constructed over its full width, with the borders extending into the over-lying impervious layers and the base of the wall reaching into the bedrock. In this way, leakage of water and any risks of erosion and collapse are avoided.

8.6 Protection of catchment and direct spring surroundings

Catchment protection

Protection of the catchment has two main objectives: (i) to improve the recharge of the aquifer, and (ii) to prevent contamination of the groundwater. Catchment protection therefore involves planning, implementation and motivation to refrain from or substantially reduce human and economic activities that could cause adverse effects on the quality and quantity of the water from the spring. Production of fodder grass and crops not requiring application of chemicals are permitted, but the feeding of animals on this fodder grass should be outside the catchment area. Protection rules and other decisions are preferably made by the local (user) communities after analysing the situation and the most suitable actions that can be taken locally.

Although trees/plants stabilise the soil and reduce erosion, they should be selected to avoid those that will compete for water (such as eucalyptus). Useful trees are, for example, pine or indigenous species (consult the Forestry Department and use local wisdom).

There are many problems associated with the protection of the catchment, including land ownership, change of habits and traditional beliefs. For individuals, reluctance to surrender their land for communal interest is a common problem. Some may consider the area to be very fertile and therefore not want to sacrifice it. Others link it to their ancestors' origins, and regard it as sacred. These problems bring conflicts in the community. Some local governments now have a legal status which allows them to regulate demarcation and intervention in an area chosen for catchment protection.

Protection of the spring surrounding

The immediate area around the spring (at least with a 50 m radius) must be fenced with barbed wire or an alternative barrier decided by the community. In this area definitely no human activities such as farming, grazing and hunting are allowed. Protection activities here include soil conservation, erosion control, drainage work and planting of trees, shrubs and grasses.

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Web sites

<http://www.skat.ch/>

Discussion groups

<http://www.lboro.ac.uk/departments/cv/wedc/garnet/>

9 Pumping

Reviewed and adjusted by Patrick Okuni

9 Pumping

9.1 Introduction

Water pumping technology developed in parallel with the sources of power available at the time. Indeed one can say that our first ancestors who cupped their hands and lifted water from a stream chose the 'pumping' technique appropriate to them. Modern devices such as centrifugal pumps have reached a high state of development and are widely used, particularly in developed countries, only because suitable power sources such as diesel engines and electric motors became available.

For small communities in developing countries, human and animal power is often the most readily available power for pumping water, particularly in rural areas. Under suitable conditions wind power is of relevance. Solar energy can also have potential. Diesel engines and electric motors should only be used if the necessary fuel or electricity supplies are reliably available, together with adequate maintenance and spare parts.

A wide range of pump types is available on the market. Prevailing local conditions and management capacities determine the type that is most suitable and sustainable. While it may seem obvious that effective involvement of users, the private sector and support organisations is important in the choice of pumping technology, the fact remains that it is frequently disregarded. Too often technical capacities of users and local support are over-estimated, resulting in pumps not being properly operated and maintained, and eventually to their breakdown.

Participation by representatives of the different user groups, including women and children of different ages, in selecting and trying out the pumps, helps to ensure that a type is chosen that is suitable and acceptable to them. Productive use of the pumped water generally has a very positive effect on the upkeep and lifetime of the pump. It also helps when users learn about the proper way of operating a specific type of pump and the underlying reasons, and set up and implement a system for proper operation as part of local participatory planning and management of the service. When local interest is not generated, lack of local funds or incentives to invest in O&M and replacement of pumps means that their condition degenerates quickly.

9.2 Power sources for pumping

Human power

The simplest pumps of all are those operated by human power. In this category come a range of handpumps and footpumps. They are capable of lifting relatively small amounts of water. Using human power for pumping water has important benefits for

small communities in developing countries:

- The power requirements can be met from within the users' group.
- The capital cost is generally low.
- The discharge capacity of one or more manual pumping devices is usually adequate to meet the domestic water requirements of a small community, including, if needed, for small-scale productive uses within households.
- Design developments during the last 20 years mean that pumps can be repaired and maintained by appropriately trained local caretakers (men or women).

The power available from human muscle depends on the individual, the environment and the duration of the task. For work of long duration, for example eight hours per day, a healthy man is estimated to produce 60-75 watts (0.08-0.10 horsepower). This value must be reduced for women, children and the aged. It also must be reduced for high temperature, and work environments with high humidity. Where the pump user and the pump are poorly matched, much of the power input is wasted, for example, when a person operates a pump from a stooped position. Tests and user evaluations help to bring out problems, such as rejection of footpumps because pregnant women and young children could not easily operate them or the movement was not culturally acceptable.

Animal power

Draught animals are a common and vital source of power in many developing countries. Animals may be used for pumping water for irrigation as well as for human consumption. The most efficient use of animals is at fixed sites where they pull rotating circular sweeps or push treadmills to drive slow moving, large displacement pumps with gears. However, with the increased access to other sources of energy, the use of animal power for water pumping is declining.

Wind power

The use of wind power for pumping water should be feasible if

- winds of at least 2.5-3 m/s are present 60% or more of the time;
- the water source can be pumped continuously without excessive drawdown;
- storage is provided, typically for at least 3 days' demand, to provide for calm periods without wind;
- a clear sweep of wind to the windmill is secured, i.e. the windmill is placed above surrounding obstructions, such as trees or buildings within 125 m; preferably the windmill should be set on a tower 4.5-6 m high;
- windmill equipment is available that can operate relatively unattended for long periods of time, e.g. six months or more. The driving mechanism should be covered and provided with an adequate lubrication system. Vanes, and sail assemblies should be protected against weathering.

By far the most common type of wind-powered pump is the slow-running wind wheel driving a piston pump. The pump is generally equipped with a pump rod that is connected to the drive axis of the windmill. Provision can be made for pumping by hand during calm periods.

The wind wheels range in diameter from about 2-6 m. Even though the windmills themselves may have to be imported, strong towers can usually be constructed from local materials.

Modern windmills are designed to ensure that they automatically turn into the wind when pumping. They are also equipped with a *pull-out* system to turn the wheel automatically out of excessive wind that might damage the windmill, i.e. stronger than 13-15 m/s. The *sails* or fan blades can be de-signed in such a way that they furl automatically to prevent the wheel from rotating too fast in high winds. The windmill will normally not begin pumping until the wind velocity is about 2.5-3 m/s. Fig. 9.1 shows several typical arrangements for windmill-pumped water supply systems.

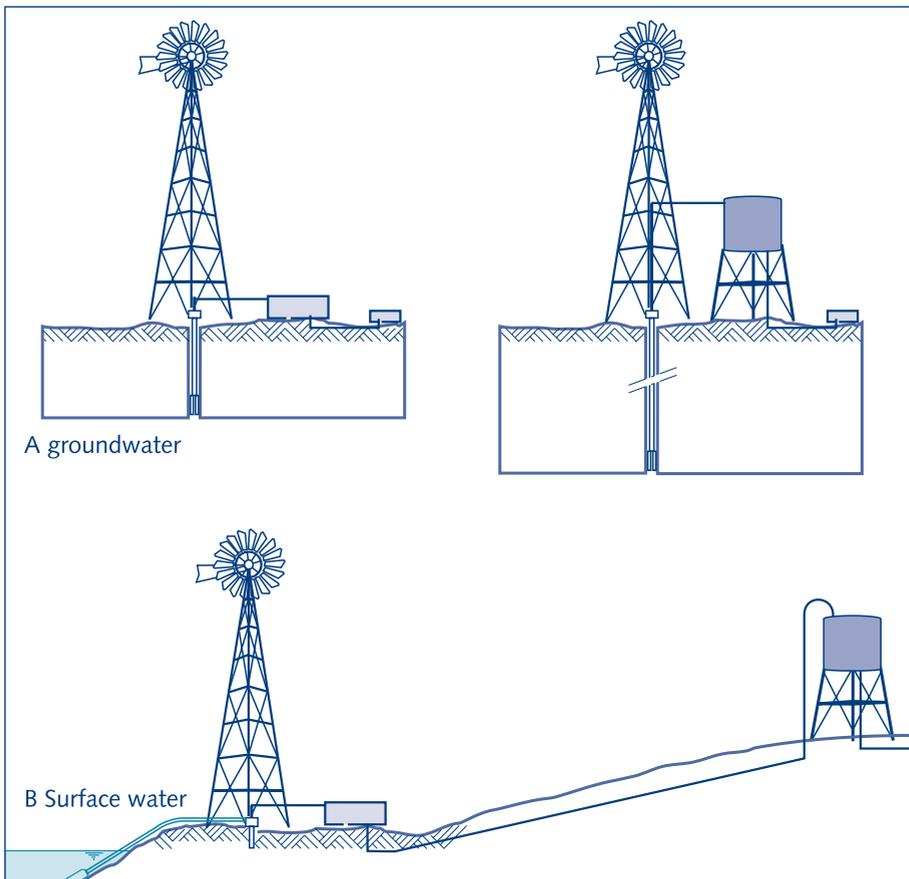


Fig. 9.1. Windmill-pumped water supply systems

Electric motors

Electric motors generally need less maintenance and are more reliable than diesel engines. They are therefore preferable as a source of power for pumping, provided a reliable supply of electric power is available. The electric motor should be capable of carrying the workload that will be imposed, taking into consideration the various adverse operating conditions under which the pump has to work. If the power requirement of a pump exceeds the safe operating load of the electric motor, the motor may be damaged or burnt out. Attention must also be paid to the characteristics of the motor and the supply voltage.

There is sometimes a tendency to use general-purpose motors offered by the manufacturers without giving due consideration to the characteristics of the particular pump used. This results in frequent failure or burning out of the motor. The squirrel-cage motor is mostly selected for driving a centrifugal pump as it is the simplest electric motor manufactured.

Diesel engines

Diesel engines have the important advantage that they can operate independently at remote sites. The principal requirement is a supply of fuel and lubricants and these, once obtained, can be easily transported to almost any location. Diesel engines, because of their ability to run independently of electrical power supplies, are especially suitable for driving isolated pumping units such as raw water intake pumps.

Diesel engines may be used to drive reciprocating plunger pumps as well as centrifugal pumps. Gearing or another suitable transmission connects the engine with the pump. For any diesel-driven pump installation, it is generally prudent to select an engine with 25-30% surplus power to allow for a possible heavier duty than under normal conditions.

Extensive use of electric and/or diesel pumps for drinking water supply and irrigation may cause shallower dug wells to fall dry. This may cause great problems for poor people - especially poor women – who depend on the wells as their domestic water source.

Solar power

Solar panels made of photovoltaic cells arranged into an array convert sunlight into a direct current (DC). This current is used to drive a submersible pump of the DC type. Another option is that the DC is first converted to alternating current (AC) as most submersible pumps on the market are of the AC type. On the other hand, converting DC means a substantial loss of energy (about 25%). In photovoltaic pumping systems, the pump works whenever there is adequate sunshine, and this is independent of the ambient temperature.

The water is pumped into a storage tank to cater for water demands during the periods that there is no sunlight and therefore no pumping. Solar powered pumps can lift water up to 100-200 m, but the system is most economical up to a pumping head of 50 m.

The solar powered pumping system is an attractive option for remote areas where power and fuel supply is difficult and expensive. The only requirement is sufficient sunlight. The investment cost is high due to the (still) high price of solar panels, but operation and maintenance costs are low. The solar system is vulnerable to vandalism and theft because solar panels have many applications.

When solar, electrical or diesel pumps are installed, it is important to assess who will decide on their installation and use, and who will manage and benefit from them. Often, different groups are involved, such as cattle owners, women from better-off families who use the water also productively, and women from poor families who use the water only for domestic uses. The interests of the more powerful groups then often prevail, unless each group is equitably involved in decisions and management.

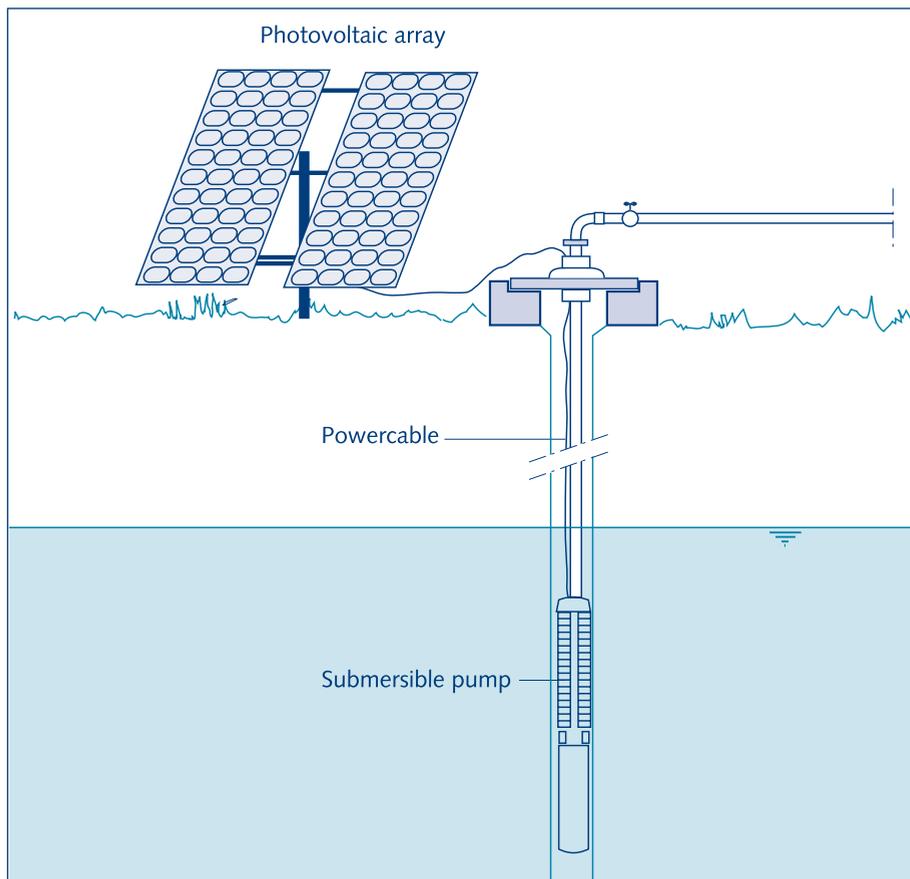


Fig. 9.2. Solar powered pump system

9.3 Types of pumps

The main applications of pumps in small community water supply systems are:

- Pumping water from wells
- Pumping water from surface water intakes
- Pumping water into storage reservoirs and the distribution system, if any

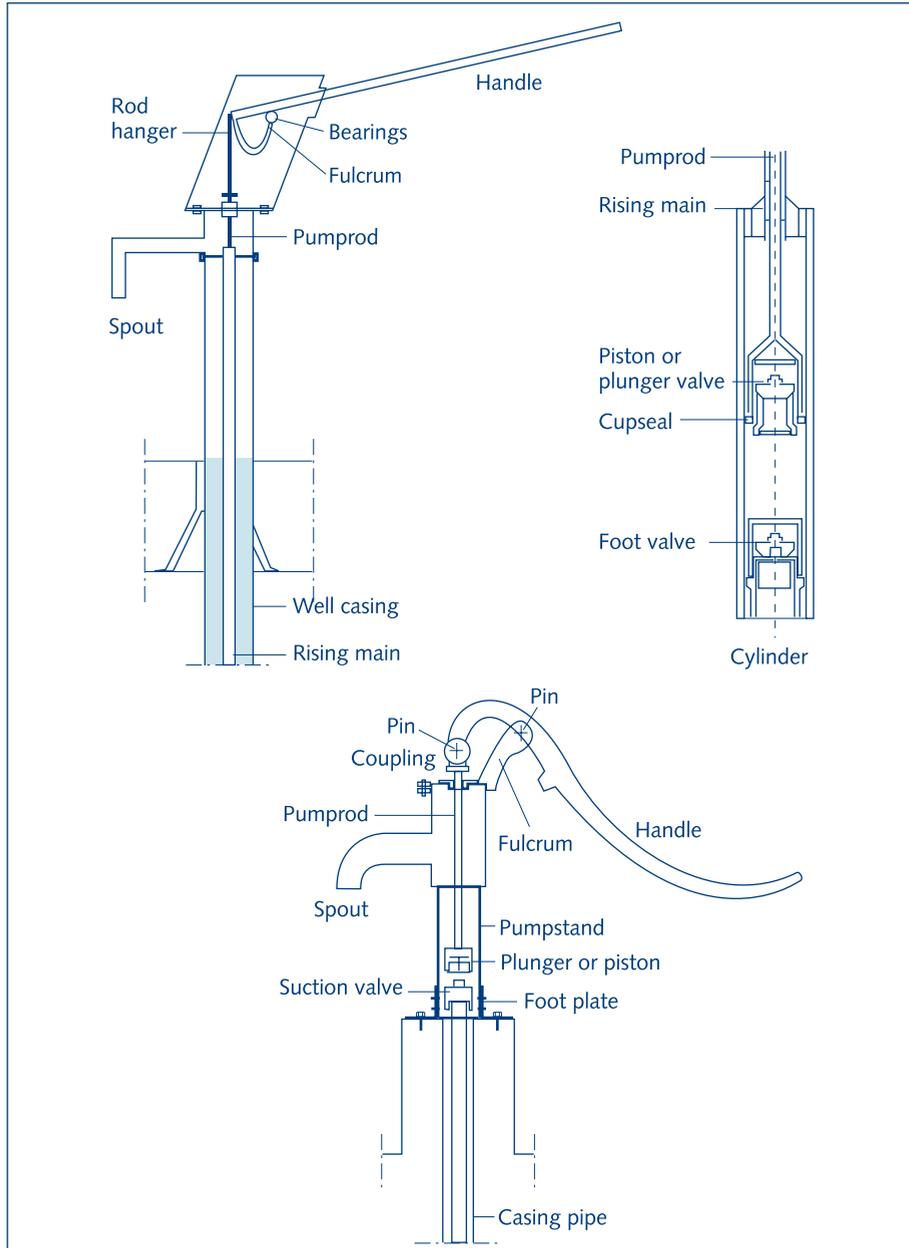


Fig. 9.3. Types of pumps

Based on the mechanical principles involved, these pumps may be classified as follows:

- Reciprocating²
- Rotary (positive displacement)
- Diaphragm
- Axial-flow (propeller)
- Centrifugal
- Air lift

Another type of pump with limited application in water supply systems is the hydraulic ram. Table 9.1 gives characteristics of the various types of pumps.

Table 9.1 Information on types of pumps

Type of pump	Depth range	Characteristics and applicability
1. Reciprocating (plunger)		Low speed of operation; hand, wind or motor powered; efficiency range 25–60%
a. Suction	Up to 7 m	Capacity range: 0.5-1 l/s; suitable to pump against variable heads; valves and pump buckets require maintenance attendance
b. Suction: treadle pump	Up to 6 m	capacity range: 0.5-2.5 l/s; mostly used for irrigation but also feasible for water supply if water is treated
c. Lift (direct action)	Up to 15 m	As for suction
d. Low lift: rower pump	Up to 3-6 m	capacity: 0.5-2 l/s mostly used for irrigation but also feasible for water supply if water is treated
e. Lift (high lift)	Up to 180+ m	As for suction
2. Rotary (positive displacement)		Low speed of operation; hand, animal, wind powered
a. Rope pump	Up to 45 m	Capacity range: 0.2-1.0 l/s Discharge constant under variable heads

2 Reciprocating pumps have a plunger (or piston) that moves up and down (reciprocates) in a closed cylinder for positive displacement of water. On the upward stroke the plunger forces water out through an outlet valve, and at the same time water is drawn into the cylinder through an inlet valve. The downward stroke brings the plunger back to its starting position, and a new operating cycle can begin.

V

Type of pump	Depth range	Characteristics and Applicability
b. Helical rotor	25-150 m Usually submerged	Using gearing; hand, wind or motor powered; good efficiency; best suited to low capacity – high lift pumping
3. Diaphragm (rubber diaphragm in cylinder)	Up to 45 m	Capacity: 0.2-0.3 l/s; hydraulic pressure on diaphragm
4. Axial-flow	5-10 m	High capacity low-lift pumping; can pump water containing sand or silt
5. Centrifugal		High speed of operation - smooth, even discharge; efficiency (range 50-85%) depends on operating speed and pumping head
a. Single-stage	20-35 m	Requires skilled maintenance; not suitable for hand operation; powered by engine or electric motor
b. Multi-stage shaft-driven	25-50 m	As for single stage; motor accessible, above ground; alignment and lubrication of shaft critical; capacity range 25-10,000 l/min
c. Multi-stage submersible	30–120 m	As for multi-stage shaft-driven; smoother operation; maintenance difficult; repair to motor or pump requires pulling unit from well; wide range of capacities and heads; subject to rapid wear when sandy water is pumped
6. Air lift	15-50 m	High capacity at low lift; very low efficiency especially at greater lifts; no moving parts in the well; well casing straightness not critical

9.4 Technology selection

Recent research and development (R&D) has focused on ease of operation and maintenance, and corrosion resistance. Corrosive water seriously affects galvanised iron and mild steel, reducing the life span of components and causing deterioration of water quality. Even stainless steel can show some corrosion (mainly galvanic³). Components made from uPVC, polypropylene and engineering plastics are increasingly being adopted. However, plastics may extend over time due to weight. Fibreglass is also being tested for pump rods in deep settings. These materials are light in weight (easing lifting) and corrosion resistant, and are likely to have a greater application in the near future.

Pump selection criteria

In selecting a pump type for a specific purpose the following technical criteria need to be considered:

- Rate of delivery required
- Vertical distance from pumping level to delivery level
- Variations expected in water levels at the source
- Durability of basic components (including corrosion resistance)
- Weight of below ground parts
- Availability and cost of spares
- Ease of maintenance

Apart from these technical criteria, several institutional and community criteria have significant influence on the sustainability of the functioning and use of the pump (see also chapter 2):

- Involvement of both men and women from the different user groups in the choice of the most suitable pump (that can also be used by children and pregnant women); choice of pump location(s); the selection of women and men who will operate, maintain and manage the water supply system and go for training; and the financing system through which the various costs will be met
- Assessment of ability and willingness to pay for the expected cost of operation and maintenance (O&M), repair and management
- Institutional and organisational capacity at community level to manage the water supply service, including technical capacity for O&M
- Representation of the different user groups in management and training for maintenance and management of the water supply
- Institutional and organisational capacity outside the community to support the community in their tasks – governmental agencies, NGOs, water associations and the private sector could provide this support

3 Galvanic corrosion is caused by contact between two materials with different electrochemical potentials in contact with an electrolyte such as water.

9.5 Reciprocating pumps

The type of pump most frequently used for small water supplies is the reciprocating (plunger) pump⁴. Several types may be distinguished:

- Suction and lift pumps
- Free delivery and force pumps

Suction pumps (shallow well)

In the suction pump, the plunger and its cylinder are located above the water level usually within the pump stand itself (Fig. 9.4).

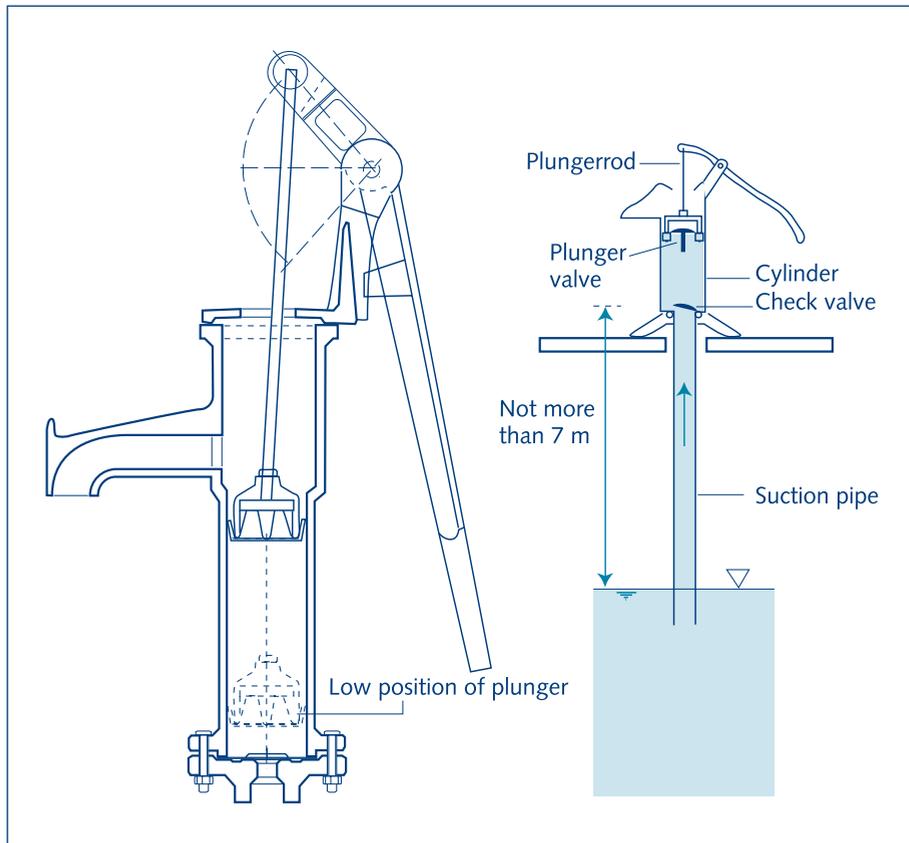


Fig 9.4. Suction pump (for shallow well)

Because of its reliance on atmospheric pressure (about 10 m of water column), the suction pump gives a high discharge up to a lifting height of 7 m (at high altitudes 4 m), beyond which it becomes unstable. In addition it cannot be used when the water table drops. Suction pumps are simple to install and maintain.

4 While this section focuses on the reciprocating plunger pump, the principles outlined also apply to other types of positive displacement pumps.

Treadle pump

In certain Asian countries, the treadle pump is commonly used as a small-scale irrigation pump. It is also suitable for drinking water supply provided the water is treated afterwards at household level. The pump is relatively easy to manufacture locally and therefore suitable for areas with limited technical development.

The pump is operated by the legs, which can produce more power than arms. Therefore, the capacity, about 1 l/s, is higher than other human-powered pumps. The treadle pump is a typical suction pump and therefore the maximum water lift is 7 m.

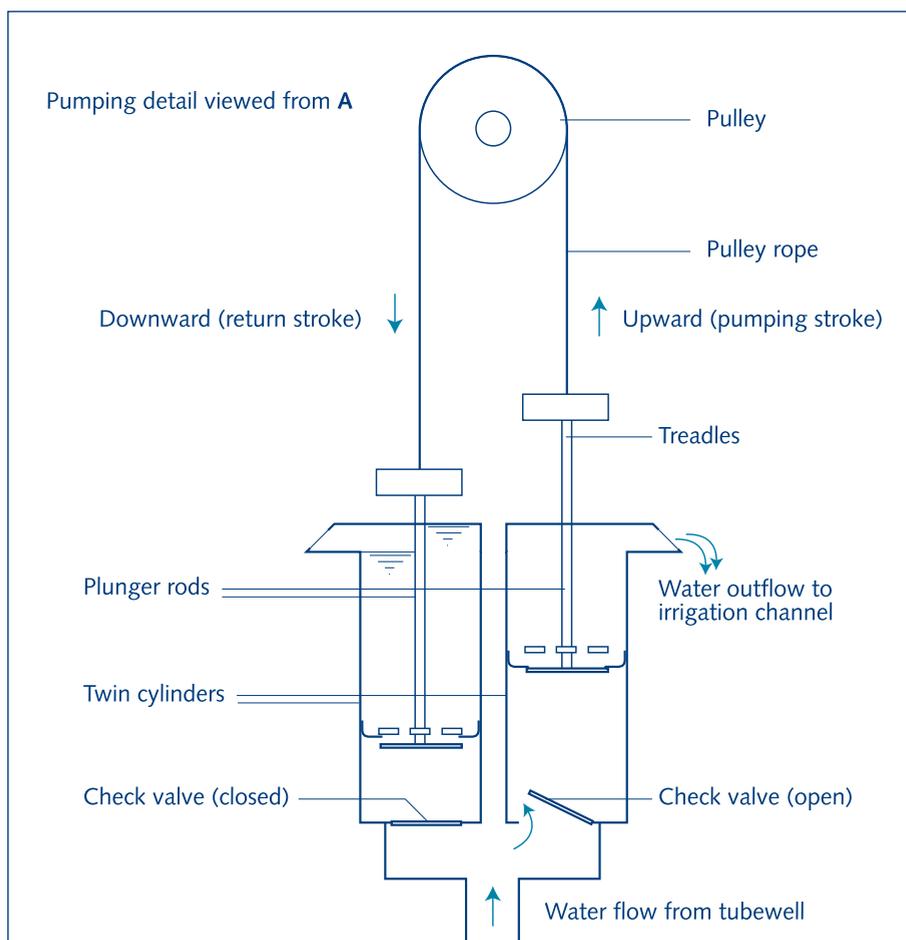


Fig. 9.5. Treadle pump

Rower pump

The rower pump is another commonly used small-scale irrigation pump, which can be used as a drinking water pump provided the water is treated before drinking. It is also a suction pump, so the maximum pumping lift is 7 m. It can be easily constructed with PVC pipes and simple valves and piston. The investment costs are low.

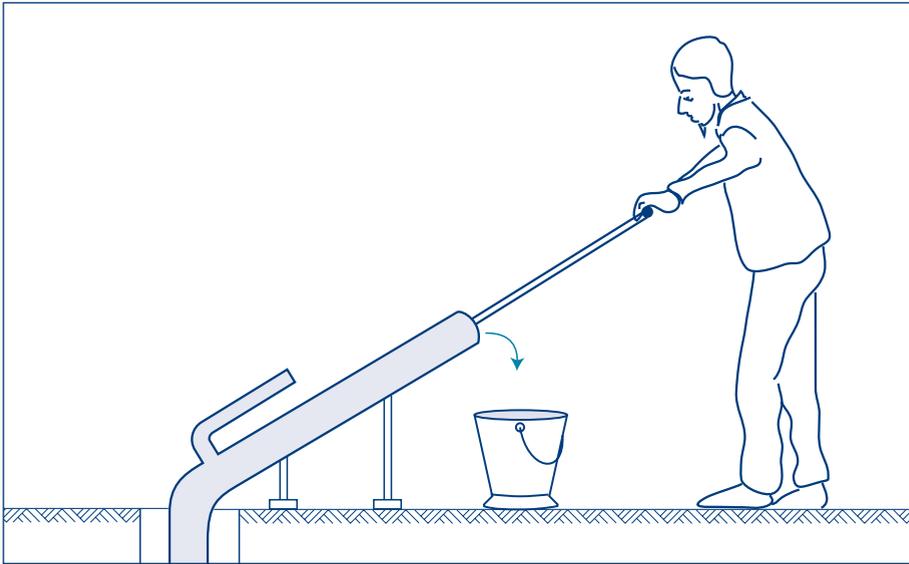


Fig. 9.6. Rower pump

Lift pumps (deep well pump)

In lift pumps, the cylinder and plunger are located below the water level in the well. The cylinder has to be submerged in the water to ensure priming of the pump. In terms of definitions, *deep* or *shallow* well refers to the depth of the water level (usually the chosen setting for the cylinder), not the depth to the bottom of the well (Fig. 9.8).

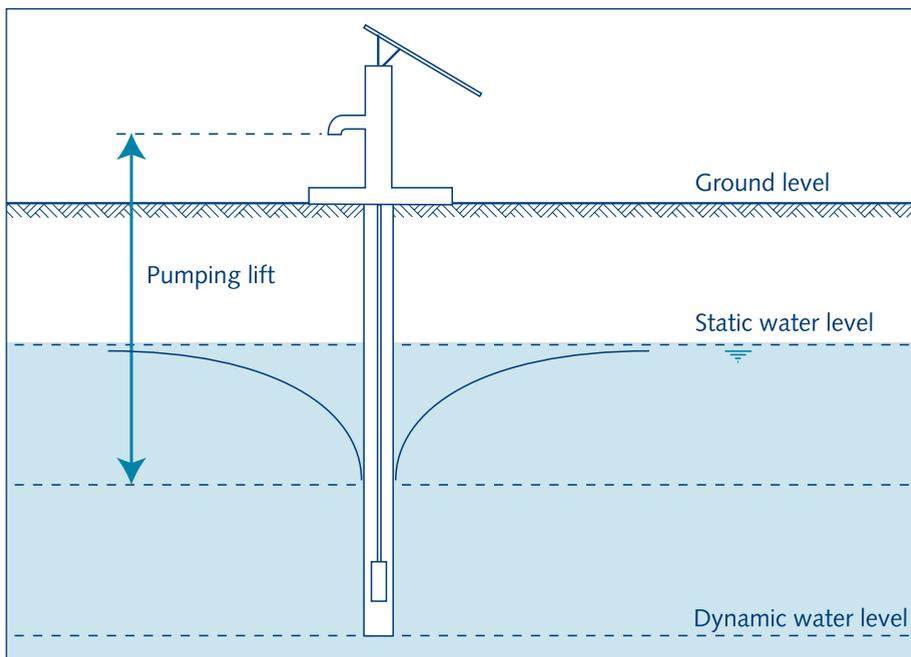


Fig. 9.7. Pumping lift

Because of the fact that the cylinder and plunger are located below the water level, this pump can lift water from wells as deep as 180 m or even more. The forces created by the pumping increase with the depth to the water table. Also the problems associated with reaching the cylinder, deep in the well, for maintenance and repair are much more difficult than in shallow well pumps. Thus the design of pumps for deep well use is more critical and complicated than for suction pumps. An example of a deep well lift pump is shown in figure 9.8.

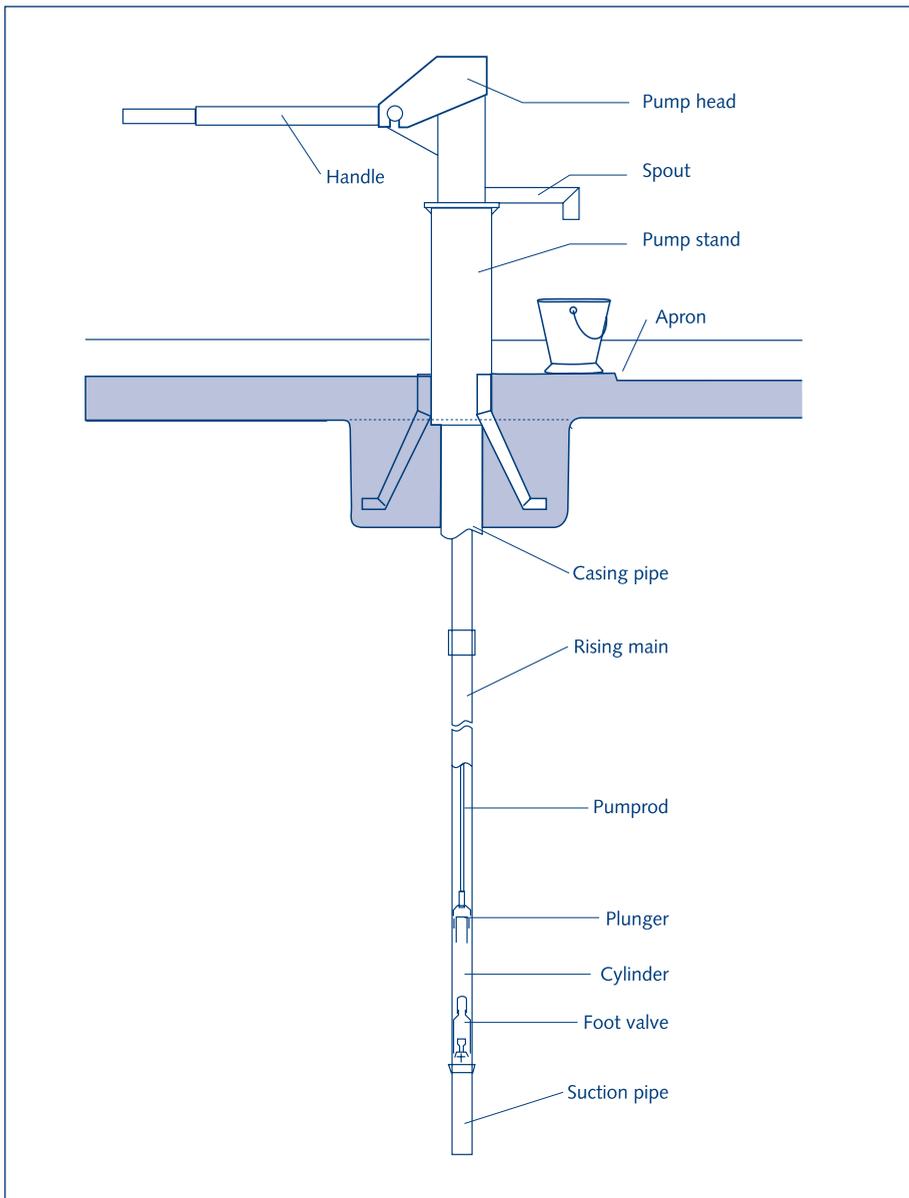


Fig. 9.8. Lift pump (deep well)

Direct action handpumps are effective up to 15 m, and are self priming, which reduces the risk of contamination. They do not rely on lever action and bearings, which eases maintenance. They are simple and light (with semi-buoyant rods), and have a high discharge.

High lift pumps are required beyond 15 m. They are sometimes used at shallower settings for reasons of standardisation.

Force pumps

Force pumps are designed to pump water from a source and to deliver it to a higher elevation or against pressure. All pressure-type water systems use force pumps. They are enclosed so that the water can be forced to flow against pressure. Force pumps are available for use on shallow or deep wells (Fig. 9.10).

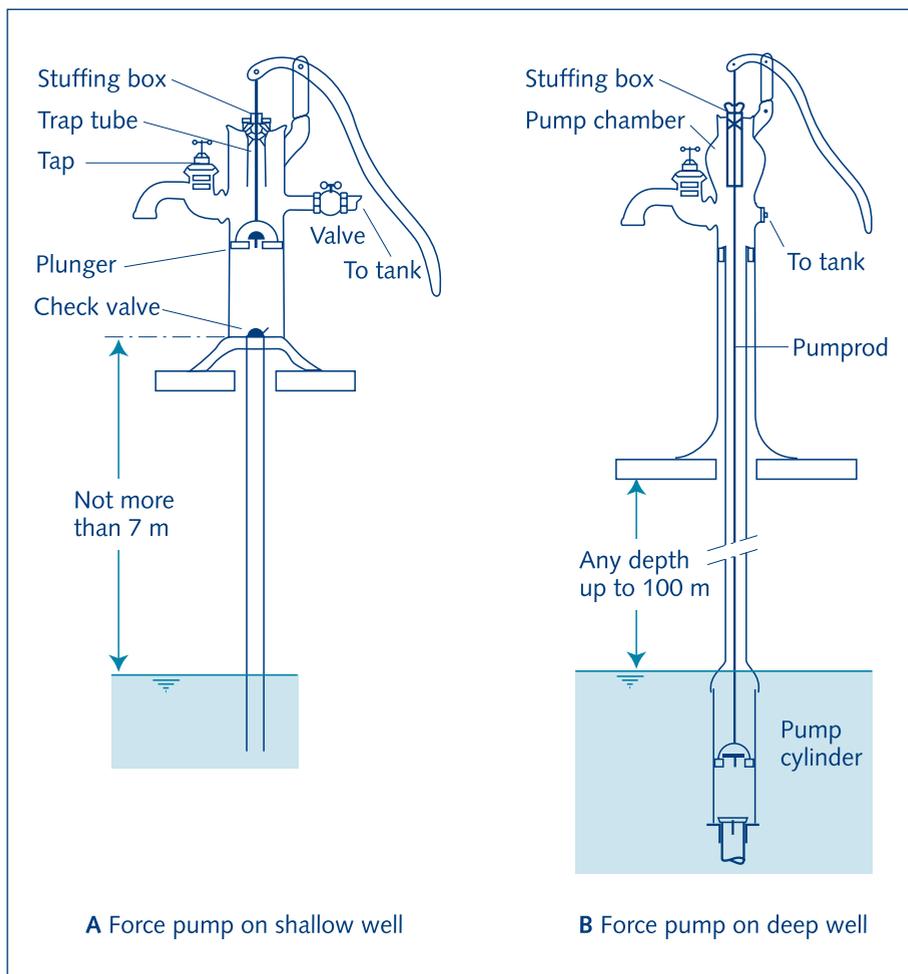


Fig. 9.9. Force pumps

A shallow-well force pump is shown in figure 9.9a. Its operating principle is the same as that of the suction pump discussed earlier, except that it is enclosed at the top and, therefore, can be used to force the water to elevations higher than the pump. For this, either a separate connection or a hose or pipe is fitted to the spout.

Force pumps usually have an air chamber to even out the discharge flow. On the upstroke of the plunger, the air in the air chamber is compressed and on the downstroke the air expands to maintain the flow of water while the plunger goes down. The trap tube serves to trap air in the air chamber, preventing it from leaking around the plunger rod.

The operation of a deep well force pump (Fig. 9.9b) is the same. The principal difference is that, like the lift pump, its cylinder is down in the well. The pump can therefore lift water from depths greater than 7 m.

Diaphragm pump

Diaphragm pumps are positive displacement pumps. There are two entirely different types.

Conventional diaphragm pumps are more commonly used as dewatering pumps than as pumps for drinking water supply. The main part of the pump is its diaphragm, a flexible disc or tube made of rubber or metal. Non-return valves are fitted at the inlet and outlet (Fig. 9.10).

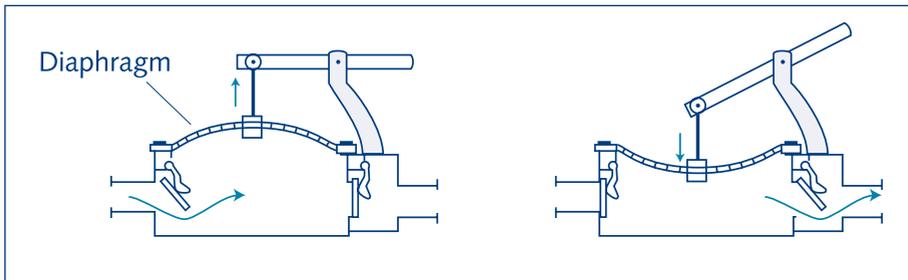


Fig. 9.10. Traditional diaphragm pump

The edge of the diaphragm is fixed to the rim of the water chamber and the centre is flexible. A rod fastened to the centre moves it up and down. As the diaphragm is lifted, water is drawn in through the inlet valve, and when it is pushed down, water is forced out through the outlet valve. These pumps are self priming.

The second type – the hydro-pump – uses the diaphragm pumping principle. The special feature in its design is the use of a small rubber tube in the pumping element that expands when water flows in and is contracted by the hydraulic action created by the pump pedal. This contraction of the diaphragm forces water through the plastic delivery

hose to the spout (Fig. 9.11). The pump needs considerable power to operate, and the diaphragm needs regular cleaning. An advantage is that it is not sensitive to sand and other small soil particles. Only the French company Vergnet manufactures this pump.

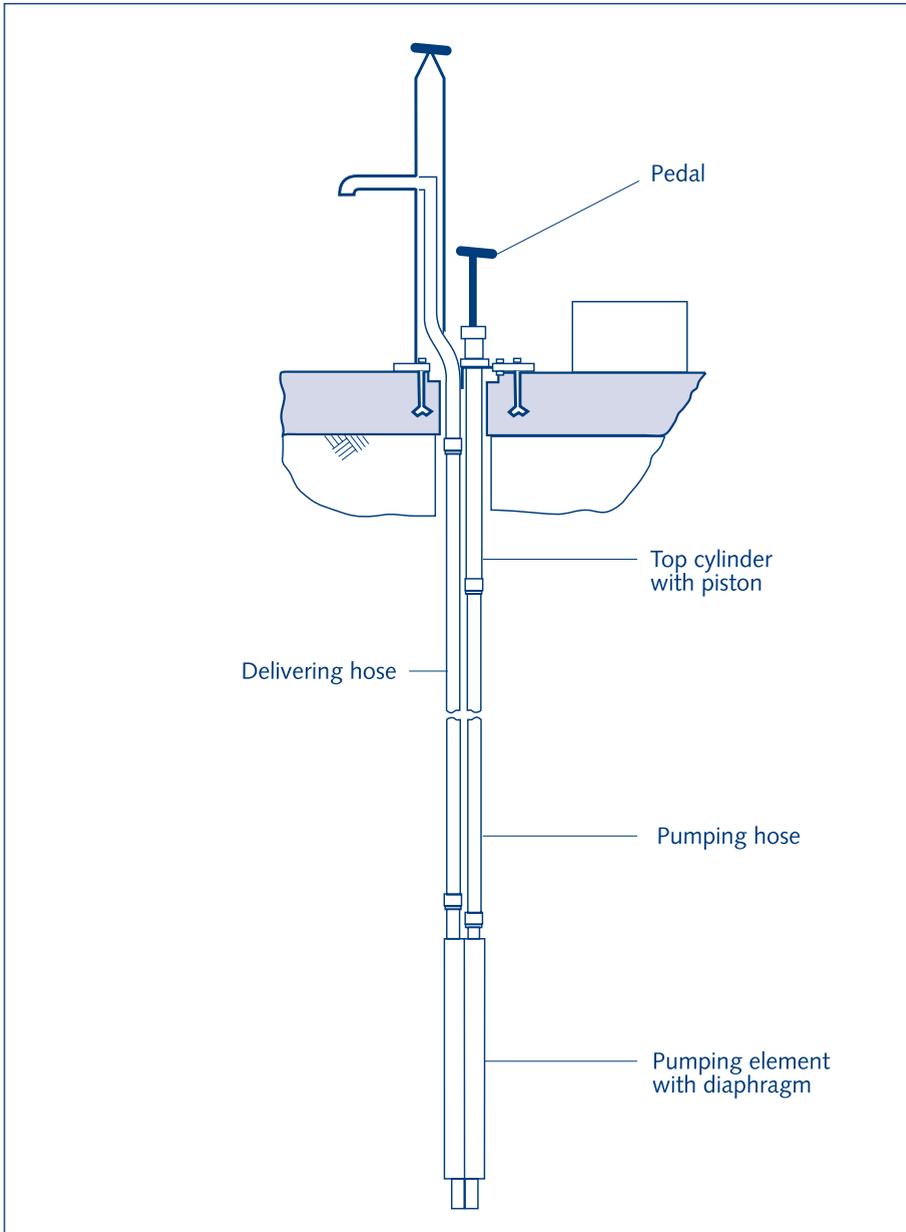


Fig. 9.11. Diaphragm foot-operated pump (hydro-pump)

9.5 Rotary (positive displacement) pumps

Bucket or rope pumps

These pumps use a continuous chain of small buckets, discs or knots on an endless rope moving over a wheel to lift water. Each bucket or disc carries water from the bottom of the well and empties it into a spout at the top. In the rope pump, the rope and discs pass through a tightly fitting riser pipe, and lift the water as they move. Rope pumps can be used for storage tanks, shallow dug wells and also boreholes. Because of their capacity, they are used at single- or multiple-family level or for small communities. They are very suited to local manufacture and maintenance. The investment cost is low and it is therefore attractive as a family pump (and irrigation pump). The rope pump needs frequent maintenance (e.g. the rope) but it can be repaired by the users with easily available material.

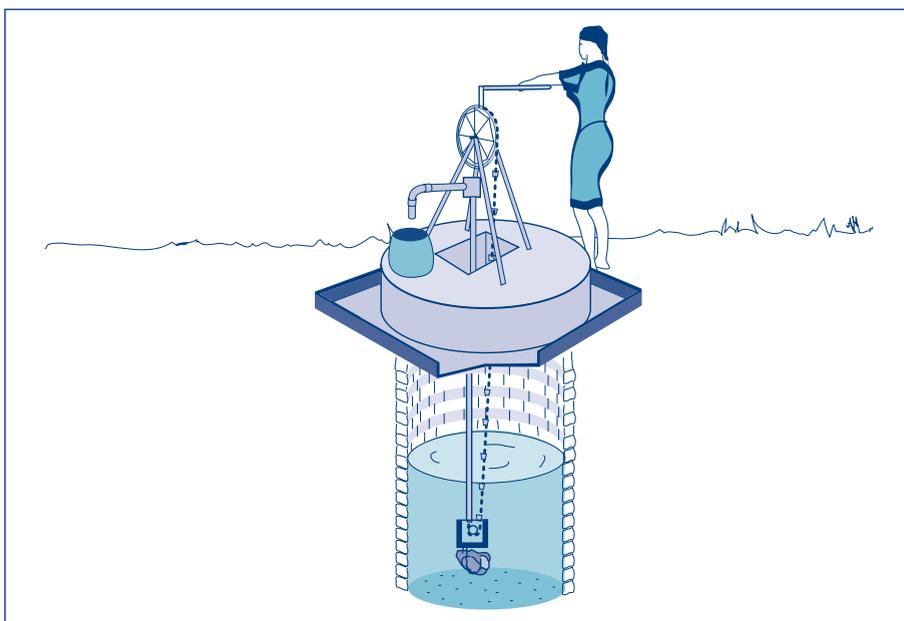


Fig. 9.12. Rope pump

Helical rotor pump

The helical rotor pump consists of a single-thread helical rotor that rotates inside a double-thread helical sleeve, the stator (Fig. 9.13). The meshing helical surfaces force the water up, creating a uniform flow. The water output is proportional to the rotating speed, and can be varied simply by changing a pulley. As the rotor and stator provide an effective continuous seal, the helical rotor pump requires no valves. Helical rotor pumps are available for use in 4-inch (100 mm) or larger tube wells. Although relatively expensive, these pumps have given good service on deep wells in parts of Africa and Asia, where they are known as “Mono” pumps after their British manufacturer.

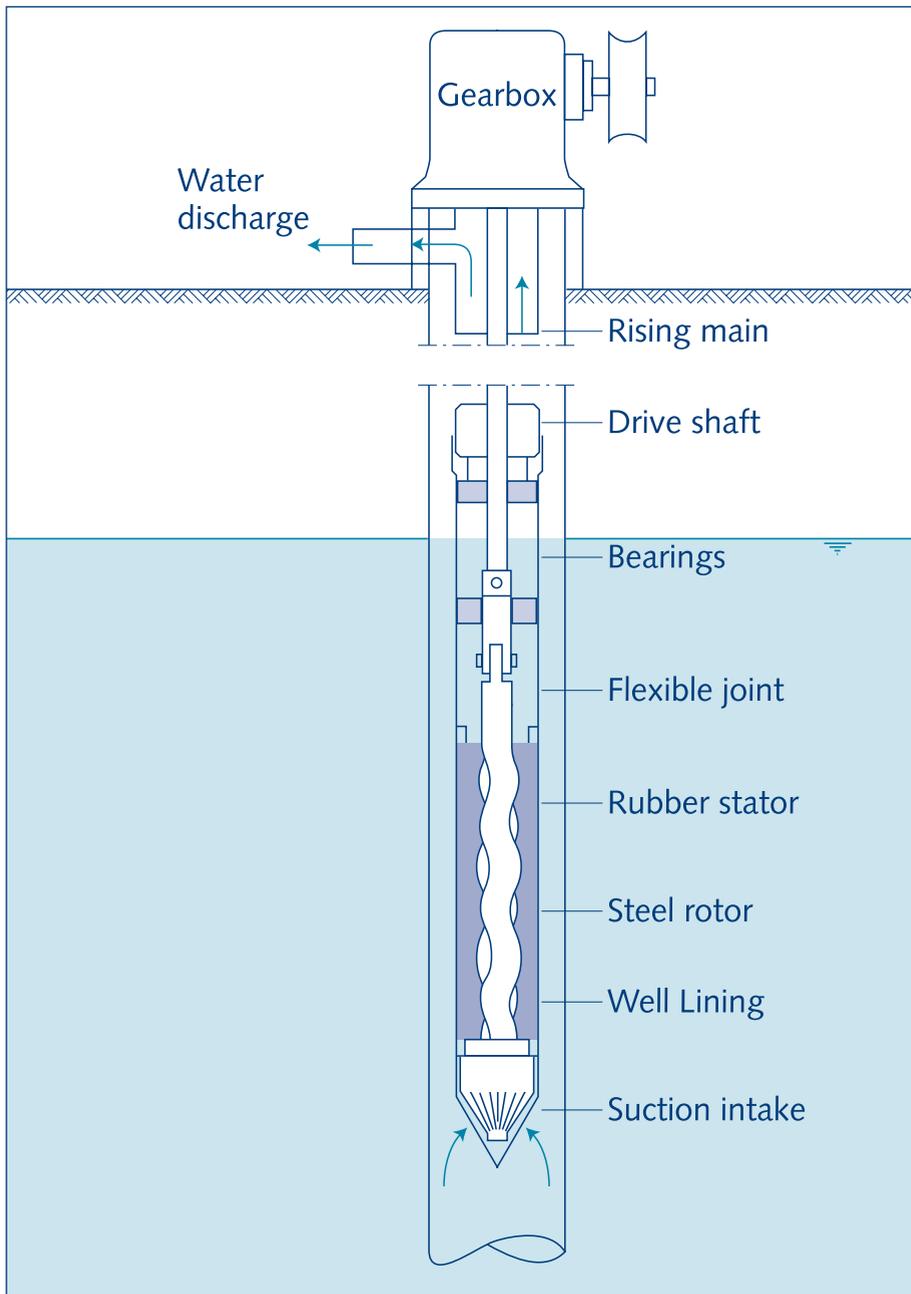


Fig. 9.13. Helical rotor pump

Drive arrangements suitable for helical rotor pumps are manual operation, electric motors, diesel and petrol engines. Different drive heads are available. If there is plenty of space, a standard head with a V-belt drive can be used. Where a compact unit is required, geared heads are installed for diesel engine or electric motor drives.

9.6 Axial-flow pumps

In the axial-flow type of pump, radial fins or blades are mounted on an impeller or wheel, which rotates in a stationary enclosure (called a casing).

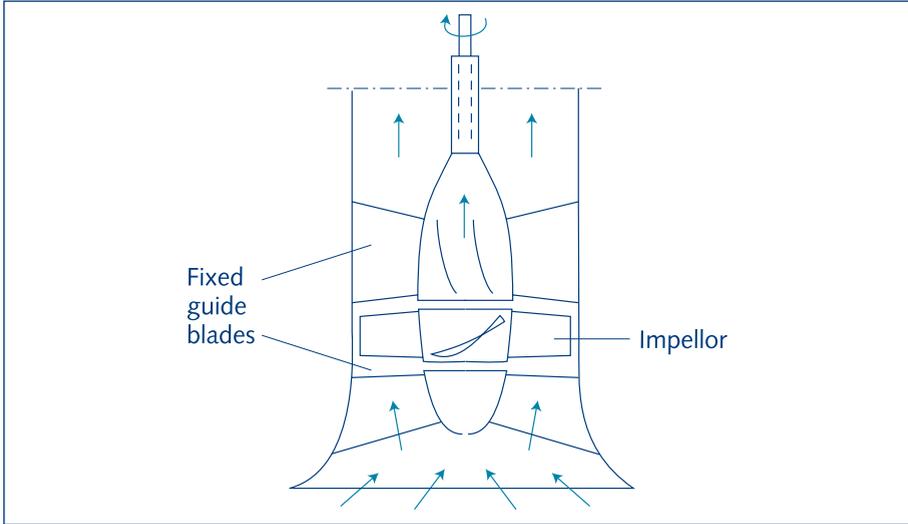


Fig. 9.14. Axial-flow pump

The action of the pump is to lift the water mechanically by the rotating impeller. The fixed guide blades ensure that the water flow has no "whirl" velocity when it enters or leaves the impeller.

9.7 Centrifugal pumps

The essential components of a centrifugal pump are the impeller and the casing (Fig. 9.15).

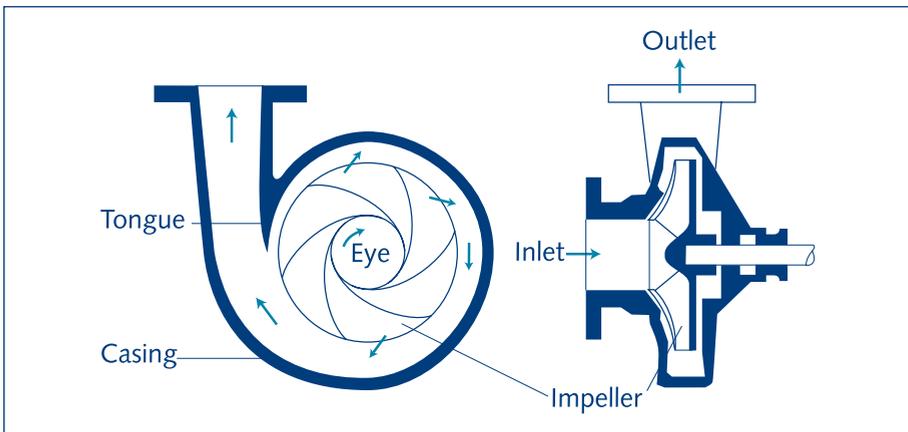


Fig. 9.15. Centrifugal pump (Volute-type casing)

The impeller is a wheel with vanes radiating from the centre to the periphery. When rotated at a sufficiently high speed, the impeller imparts kinetic energy to the water and produces an outward flow due to the centrifugal forces. The casing is so shaped that the kinetic energy of the water leaving the impeller is partly converted to useful pressure. This pressure forces the water into the delivery pipe. The water leaving the eye of the impeller creates suction; it will be replaced by water drawn from the source and forced into the casing under static head.

An impeller and the matching section of the casing are called a stage. If the required water pressure (pumping head) is higher than a single stage can practicably produce, a number of stages may be placed in series (multiple-stage pump). The impellers are attached to a common shaft and therefore rotate at the same speed. The water passes through the successive stages, with an increase in pressure at each stage. Multiple-stage centrifugal pumps are normally used for high pumping heads.

The rotating speed of a centrifugal pump has a considerable effect on its performance. The pumping efficiency tends to improve as the rotating speed increases. Higher speed, however, may lead to more frequent maintenance requirements. A suitable balance has to be aimed at between the initial cost and maintenance cost. A comprehensive study of the pump's characteristics is necessary for final selection.

In centrifugal pumps the angle between the direction of entry and exit of the water flow is 90°. In an axial-flow pump the water flow continues through the pump in the same direction with no deviation (0°). The term *mixed-flow pump* is used for those centrifugal pumps where the change in angle lies between 0° and 90°; they can be single or multiple stage.

9.8 Pump drive arrangements

Two different drive arrangements exist for pumping water from deep wells: shaft-driven and close-coupled submersible electric motor.

Shaft-driven

The crankshaft or motor is placed at the ground surface and powers the pump using a vertical drive shaft or spindle (Fig. 9.16). A long drive shaft will need support at regular intervals along its length and flexible couplings to eliminate any stresses due to misalignment. The advantage of a drive shaft is that the drive mechanism may be set above ground or in a dry pit and thus will be readily accessible for maintenance and repair. An accurate alignment of the shaft is necessary; the shaft-drive arrangement is not possible in crooked tubewells.

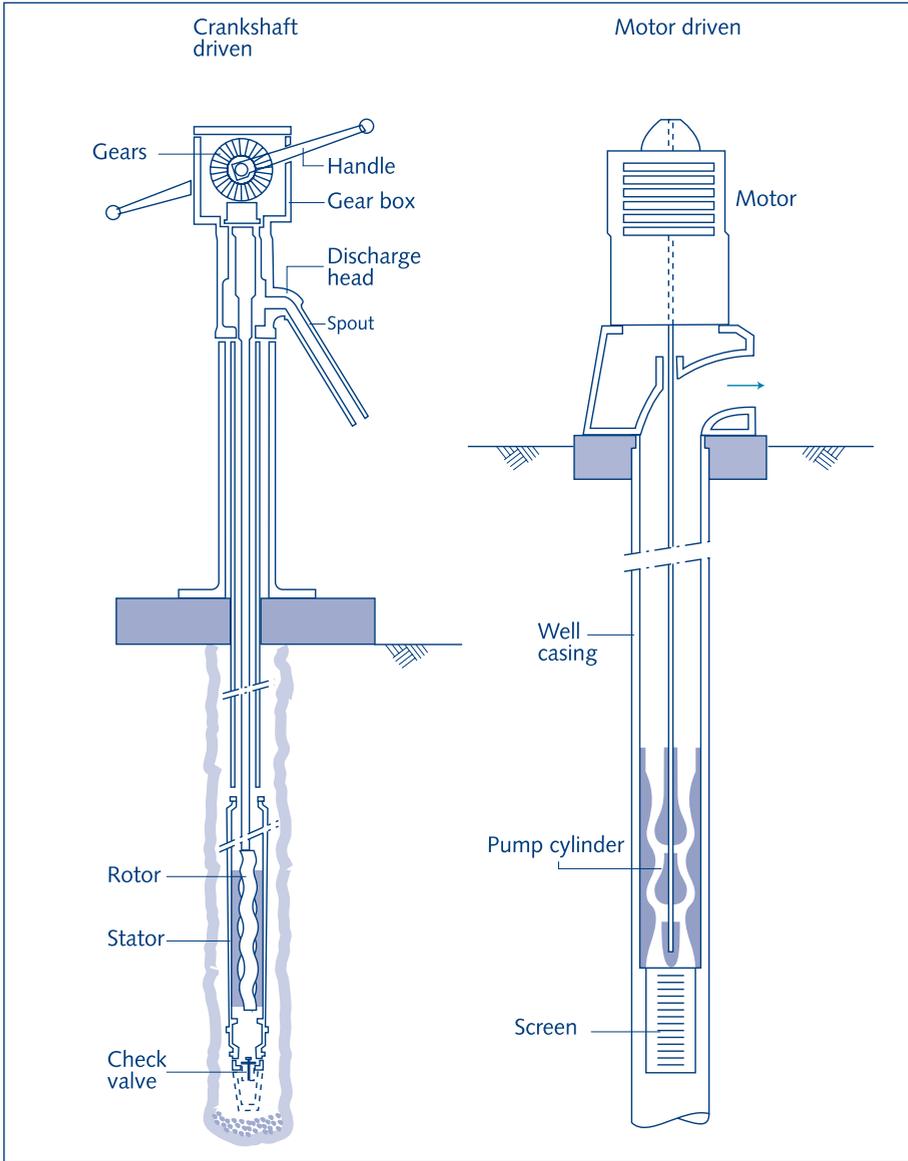


Fig. 9.16. Shaft-driven pumps

Close-coupled submersible electric pump

In this pump drive arrangement, a centrifugal pump is connected directly to an electric motor in a common housing, with the pump and motor as a single unit. This unit is constructed for sub-merged operation in the water to be pumped (Fig.9.17).

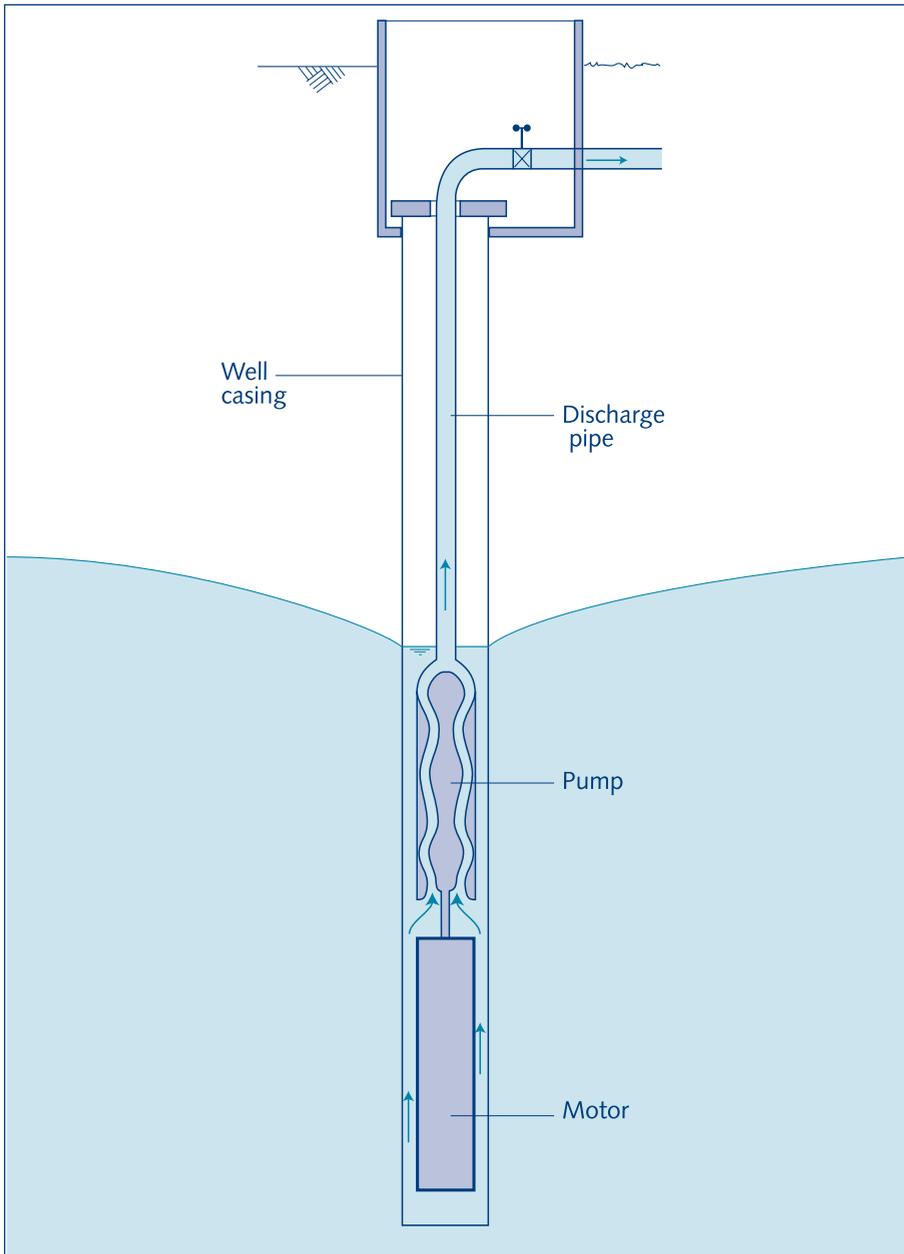


Fig. 9.17. Pump driven by a close-coupled submersible electric motor

The pump-motor unit, usually called the sub-mersible pump, is lowered inside the well casing and set at a suitable depth below the lowest drawdown water level in the well. Submersible pumps are often a tight fit in a tubewell as their outside diameter is usually only 1-2 cm less than the internal bore of the well casing. Consequently, great care is needed during installation or removal of these pumps. A waterproof electric cable connects the motor with the control box housing, the on-off switch and the power connection.

The electrical control should be properly grounded to minimise the risk of shorting and damage to the motor. Figure 9.18 shows a submersible pump in exploded view. The submersible pump-motor unit is usually supported by the discharge pipe, which conveys the pumped water to the connecting pipeline or tank. When sand is found or anticipated in the water source, special precautions should be taken before a submersible pump is used. The abrasive action of sand during pumping would shorten the life of the pump considerably.

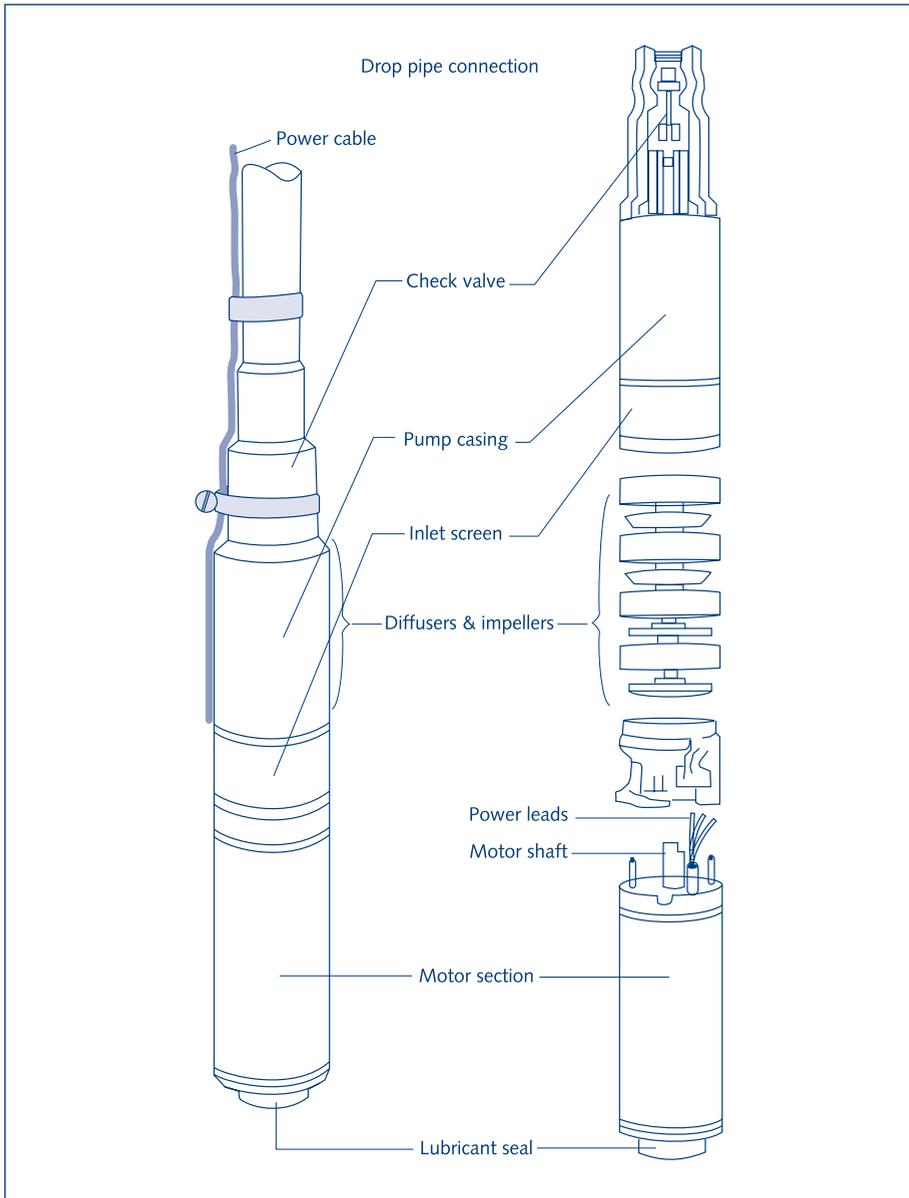


Fig. 9.18. Submersible pump (exploded view)

9.9 Air-lift pumps

An air-lift pump raises water by injecting small, evenly distributed bubbles of compressed air at the foot of a discharge pipe fixed in the well. This requires an air compressor. Because the mixture of air and water is lighter than the water outside the discharge pipe, the water/air mixture is forced upward by the hydrostatic head (Fig. 9.19).

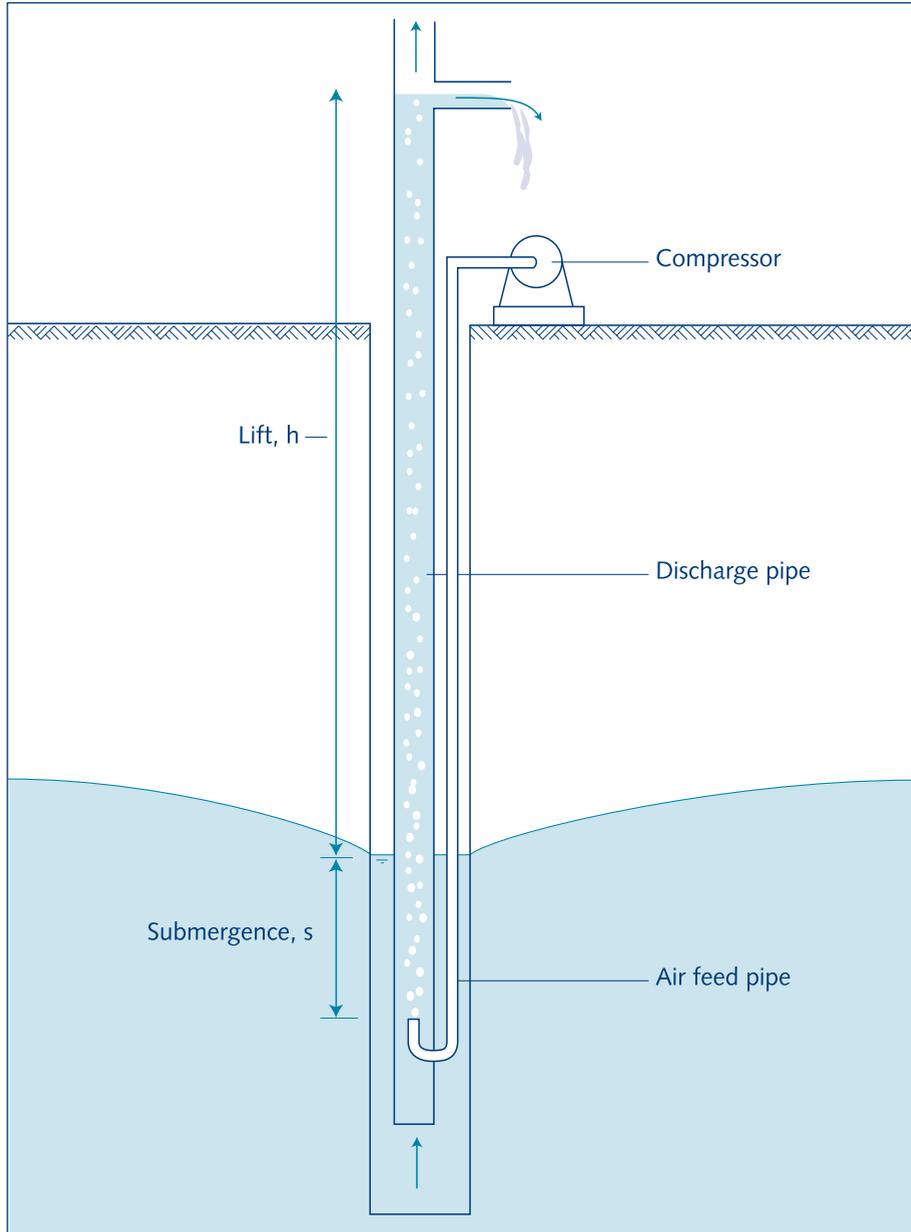


Fig. 9.19. Air lift pump (schematic)

The major drawback of air-lift pumps is their low mechanical efficiency, about 25-40%. The additional energy losses in the compressor give a total efficiency of not more than 15-30%. Air-lift pumps have the important advantages that they are simple to operate and not affected by sand or silt in the pumped water.

9.10 Hydraulic ram

The hydraulic ram needs no external source of power. The ram utilises the energy contained in a flow of water running through it, to lift a small volume of this water to a higher level. The phenomenon involved is that of a pressure surge, which develops when a moving mass of water is suddenly stopped. A steady and reliable supply of water is required with a fall sufficient to operate the hydraulic ram. Favourable conditions are mostly found in hilly and mountainous areas (Fig. 9.20). Hydraulic rams are not suited to pumping water from wells.

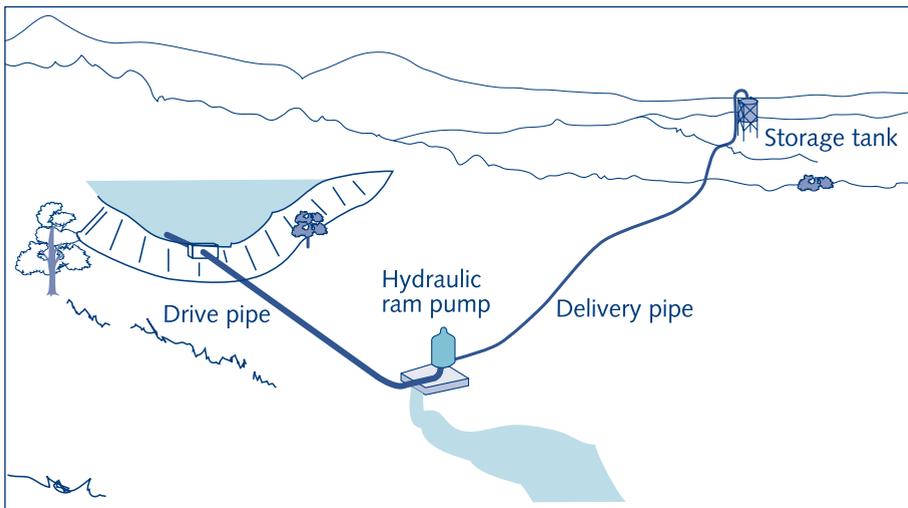


Fig. 9.20. Hydraulic ram installation

The ram operates on a flow of water running from the source down through the drive pipe into the pump chamber. The water escapes through the opened impulse valve (waste valve). When the flow of water through the impulse valve is fast enough, the upward force on the valve will exceed the spring tension of the valve adjustment and the impulse valve is suddenly shut. The moving mass of water is stopped, with its momentum producing a pressure surge along the drive pipe. Due to the pressure surge, water is forced through the non-return (delivery) valve and into the delivery pipe. Water continues to pass the non-return valve until the energy of the pressure surge in the drive pipe is exhausted. The air chamber serves to smooth out the delivery flow of water, as it absorbs part of the pressure surge that is released after the initial pressure wave.

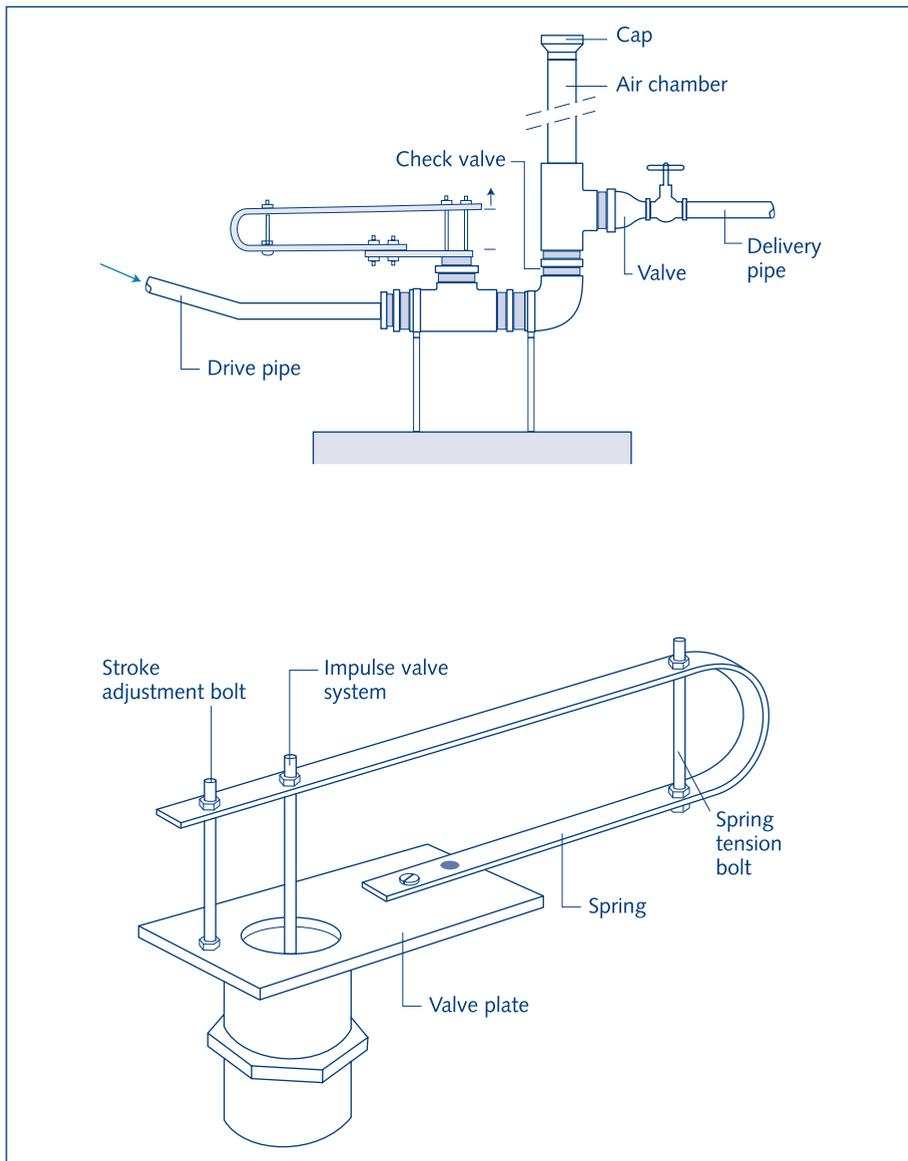


Fig. 9.21. Typical hydraulic ram

When the pressure surge is fully exhausted, a slight suction created by the momentum of the water flow, together with the weight of the water in the delivery pipe, shuts the non-return valve and prevents the water from running back into the pump chamber. The adjustment spring now opens the impulse valve, water begins to escape through it, and a new operating cycle is started.

Once the adjustment of the impulse valve has been set, the hydraulic ram needs no attention provided the water flow from the supply source is continuous at an adequate rate and no foreign matter gets into the pump, blocking the valves.

An air valve is provided to allow a certain amount of air to bleed in and keep the air chamber charged. Water under pressure will absorb air and without a suitable air valve the air chamber would soon be full of water. The hydraulic ram would cease to function.

The advantages of the hydraulic ram are:

- No power sources are needed, and therefore no running costs
- Suitable for local production
- Only two moving parts

Most hydraulic rams will work at their best efficiency if the supply head is about one-third of the delivery head. The higher the pumping head required, the smaller the amount of water delivered. In cases where the required pumping capacity is greater than one hydraulic ram can provide, a battery of several rams may be used, provided the supply source is of sufficient capacity (Fig. 9.22).

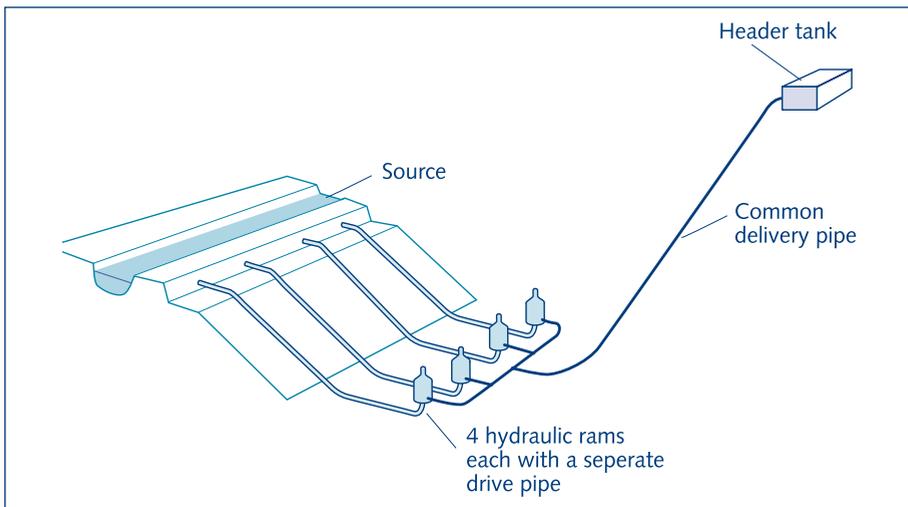


Fig. 9.22. Hydraulic rams placed parallel.
Source: Watt S.B., 1974

The maintenance required for a hydraulic ram is very little and infrequent. It includes activities as replacement of the valve rubbers when they wear out, adjusting the tuning, and tightening bolts if they get loose. Occasionally the hydraulic ram may need dismantling for cleaning. It is essential that as little debris as possible enters the drive pipe. For this reason, it is necessary to provide a grate or strainer to keep back floating leaves and debris.

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Web sites

For technical drawings: <http://www.skat.ch>

HTN Network for cost-effective technologies: <http://www.skat.ch/htn/>

Discussion groups

GARNET-HTN see: <http://www.lboro.ac.uk/departments/cv/wedc/garnet/tnchandp.html>

10 Groundwater withdrawal

*Revised and updated by
Patrick Okuni and John Farr*

10 Groundwater withdrawal

10.1 Introduction

For community water supply systems, groundwater is almost always the preferred source. Surface water sources are very likely to be contaminated and much more subject to seasonal fluctuation. Groundwater withdrawals often can be continued long after drought conditions have depleted the rivers and streams. Use of groundwater for community water supplies is probably still very much below its potential in many countries.

Frequently, available data on groundwater resources are grossly inadequate. Successful development of groundwater supplies may then be promoted by prospecting (exploration) studies. These bring to light the physical and chemical characteristics of the groundwater as well as the potential yield.

Tapping of groundwater resources, both for drinking water supply and for irrigation purposes, dates back to ancient times. In China at least 3000 years ago, wells were being drilled with hand-operated churn drills to depths as great as 100 m and lined with bamboo casings. Hand-dug wells have been sunk since time immemorial, sometimes to a considerable depth, and such wells continue to be constructed in several parts of the world. The technology for tapping groundwater at great depth through boreholes/tubewells¹ is more recent.

The first type of water well drilling that came into general use was the cable-tool (percussion) method. Over a period of several centuries it has developed from crude forms to a number of fairly sophisticated techniques. The need to prevent the collapse of un-stable ground formations and the problems of controlling at depth the heavy tools required for percussion drilling, encouraged the development of other drilling methods. These use rotating cutters or *bits* that bore into the ground while a fluid is passed through them (direct-circulation rotary drilling). For water supply wells, use of a clay-based mud fluid causes problems, as the aquifers to be tapped tend to clog up. This led to development of the reverse-circulation rotary drilling method in which a high-rate flow of clean water is used to carry the cuttings out of the drilled hole. A later logical step was the pneumatic tool placed at the bottom of the drill pipe. In the 1950s, the *down-the-hole hammer* drilling method was introduced. The efficiency of this tool proved remarkable and, even in hard-rock formations, small-diameter holes may now be drilled in a fraction of the time previously required.

1 The words *borehole* and *tubewell* are used interchangeably in different parts of the world. In this publication, *borehole* will be used from now on.

No particular water well drilling technique is applicable under all conditions. Any well construction method can be suitable, depending on the circumstances, though the general trend is towards rotary drilling to reduce time and cost. So, techniques for reaching the groundwater range from ancient methods such as the simple digging of wells with hand tools, and the excavation of the famous ganats (underground galleries extending many kilometres) in Iran and Afghanistan, to the sophisticated drilling machines (*drilling rigs*) capable of making a borehole some hundreds of metres deep even in hard-rock formations.

10.2 Groundwater occurrence and prospecting

Prospecting for water requires a basic knowledge of the various kinds of groundwater-bearing formations that can be found in the earth's crust.

Occurrence

Groundwater occurs in pores, voids or fissures of ground formations. Pores are the spaces between the mineral grains in sedimentary ground layers and in decomposed rocks. The amount of pore space in a ground formation depends upon such factors as grain size, shape, packing and the presence of cementing material. Porosity is the ratio of pore space to total ground volume (Fig. 10.1). A high porosity does not always indicate good permeability (water-bearing potential). Although clays and silts have a high porosity, the size of the pores is too small to allow water to flow easily.

All openings in rocks such as joints, bedding, cleavage planes and random cracks are called fissures in hydrogeological terminology. Igneous² rocks are not generally porous unless they have been decomposed by weathering. Lavas, which contain cavities formed by gas bubbles that escaped during the eruption, can be an exception. Even when a ground formation is highly porous the permeability may be very low because the voids are not always inter-connected. Fissures may also occur in sedimentary³ rocks.

Geologically young and unweathered fissures in all types of ground formation tend to be closed and are likely to contain little or no water. As weathering proceeds, the fissures will open up near the ground surface but remain closed at depth.

Aquifers (water-bearing ground formations) that hold most of their water in large joints and fissures are called pervious, whereas those with the water in pores are called porous. Table 10.1 shows common ground types and the way water usually occurs in them.

² Igneous: originating by solidification of molten or partly molten material (magma)

³ Sedimentary: resulting from the consolidation of layers of loose fragmental material by pressure to form rock

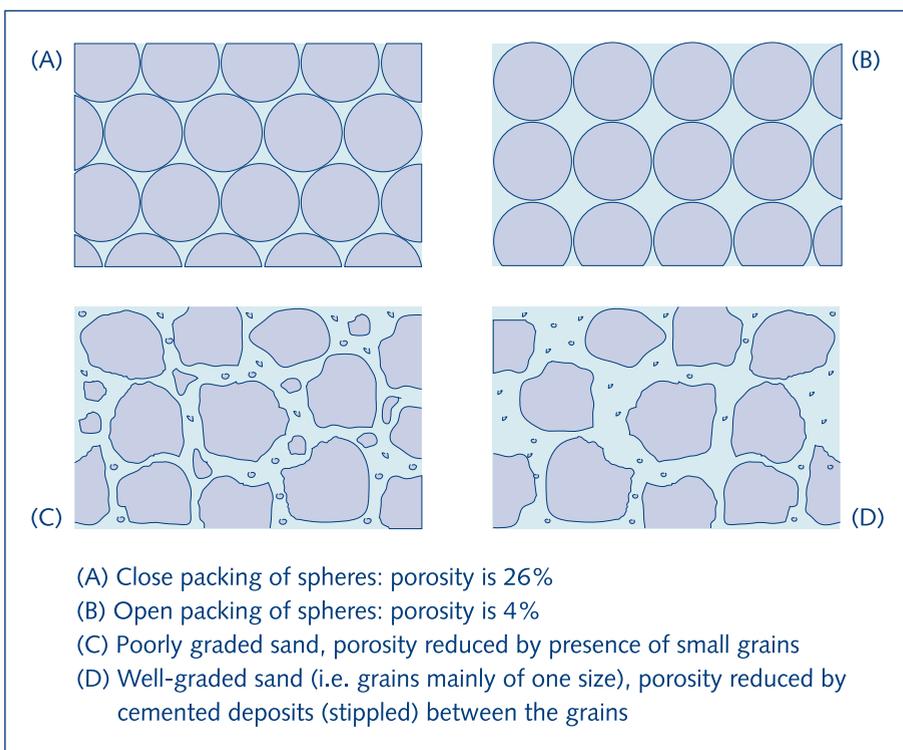


Fig. 10.1. Porosity and texture

Table 10.1 Usual mode of water occurrence

Ground type	Water usually occurring in:
Sand and gravel	Pores
Sandstone	Pores and fissures
Limestone	Fissures often expanding into caves
Chalk	Pores and fissures
Clay	Very small pores
Massive igneous	Fissures with pores in weathered zones
Lava	Fissures with pores in igneous zones
Metamorphic	Fissures with pores in weathered zones

The ease with which water can flow through a ground formation under a hydraulic head is termed the hydraulic permeability coefficient. Hydraulic permeability coefficient is expressed as the velocity of flow of water through the ground per unit of hydraulic gradient, e.g. mm/s or m/d. It depends on the porosity, the average pore size and the distribution of the fissures (see table 10.2).

Ground layers with a very low hydraulic permeability (less than about 10^{-6} mm/sec) are said to be impermeable and those with higher hydraulic permeability are regarded as permeable.

Table 10.2 Porosity and hydraulic permeability for some common ground materials

Material	Porosity (%)	Hydraulic permeability coefficient in mm/s
Clay	45-55	$10^{-3} - 10^{-9}$
Silt	40-50	$10^{-2} - 10^{-6}$
Sand	35-40	$10^{-1} - 10^{-2}$
Clean gravel	40-45	$10^3 - 10^1$
Sandy gravel	25-40	$10^1 - 10^{-2}$
Sandstone	10-20 (pores) (fissures)	$10^{-4} - 10^{-6}$ 10^{-1}
Limestone	1-10 (pores) (fissures)	$10^{-6} - 10^{-8}$ 10^2
Granite (fresh)	1 (pores) (fissures)	10^{-10} 10^2

Figure 10.2 shows the distribution of water in and above an unconfined aquifer.

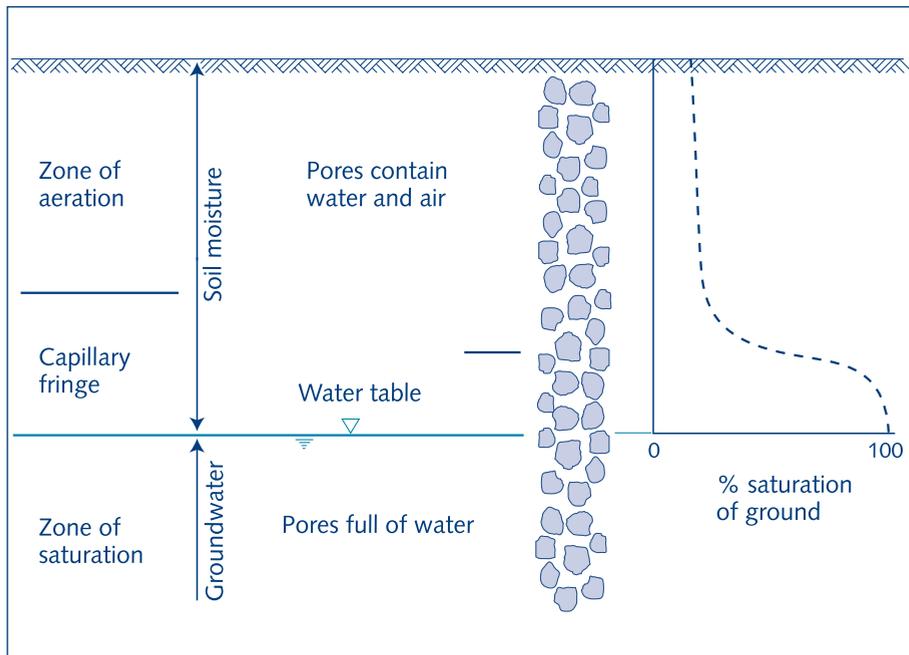


Fig. 10.2. Water distribution above and in a porous unconfined aquifer

An unconfined aquifer is open to infiltration of water directly from the ground surface. This is illustrated in figures 10.3a and 10.3b.

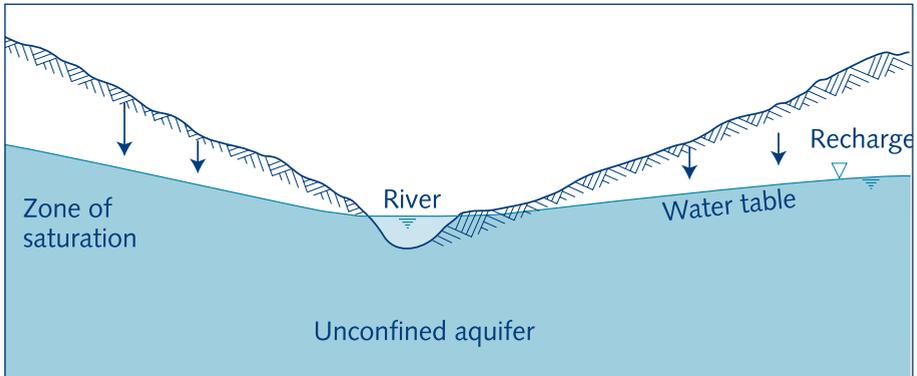


Fig. 10.3a. Infiltration of water into an unconfined aquifer during the wet season

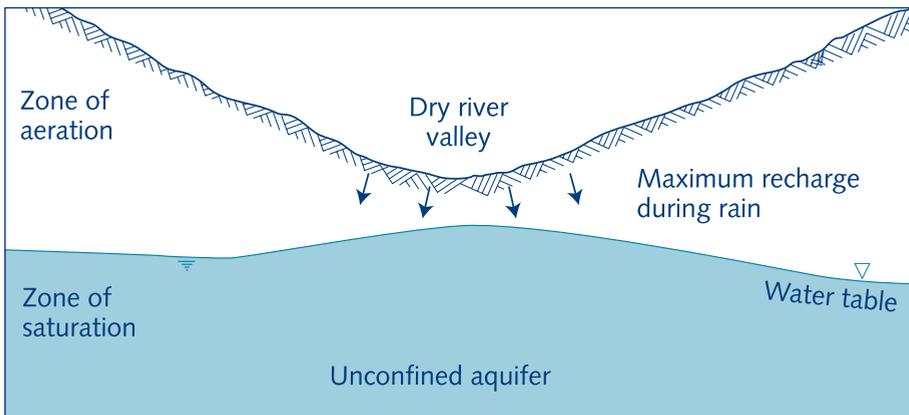


Fig. 10.3b. Infiltration of water into an unconfined aquifer during the dry season

A confined aquifer (Fig. 10.4) is one where the water-bearing ground formation is capped by an impermeable ground layer. The water pressure in a confined aquifer is related to the water level in its recharge area.

The water pressure in a confined aquifer can be measured by drilling into it and observing the level to which the water rises in the borehole. This water level is called the piezometric level. If the piezometric level is above the ground surface, water from the aquifer will naturally overflow from the borehole, which is then called a free-flowing artesian well.

Infiltration of water from the ground surface through permeable ground towards the groundwater table will be halted where a lens of impervious material such as clay is present (fig.10.5). Water will then accumulate in the ground above this lens, forming a perched water table of some distance above the real groundwater table. It is very

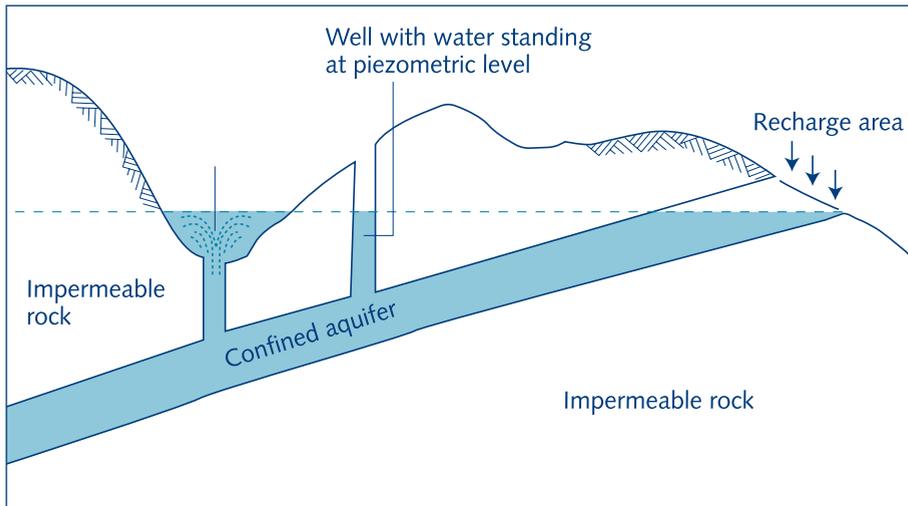


Fig. 10.4. Confined aquifer fed from a recharge area

important to identify a perched water table since the amount of water it contains is often small. Frequently, perched water tables will disappear during dry periods when there is no recharge by infiltration from the ground surface.

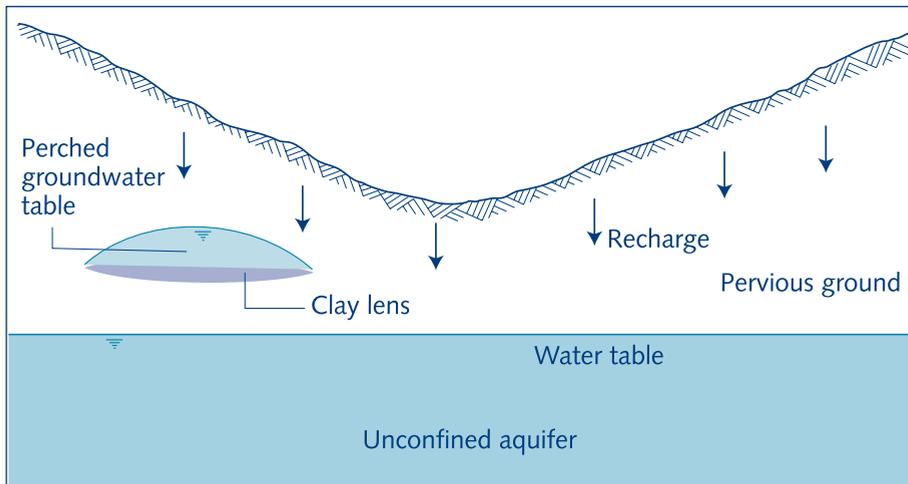


Fig. 10.5. Perched water table

Prospecting

Successful prospecting for groundwater requires knowledge of the manner in which water exists in the water-bearing ground formations. Without this knowledge, effective and efficient water exploration is impossible, and well drilling then becomes rather like a game of roulette. The aim of the prospecting work must be clearly defined. Is it for providing a small local supply or is it to determine aquifer characteristics for the development of the ground-water resources of an entire area?

Available hydrogeological information about the study area should be collected and collated. This may include: geological maps and reports, topographical maps, logs of boreholes, surface geological reconnaissance, meteorological records, and hydrological data.

A survey of the study area should be made, preferably towards the end of the dry season. This survey also taps the indigenous knowledge of local men and women on the history of water sources, water quality and land uses. They also know the flood-prone areas not suitable for well development. Consultation with both sexes is needed because in most cultures men and women have different tasks. Hence their knowledge of water resources also differs. In cultures where male outsiders cannot talk with women, it is often possible to sound out women on their knowledge and needs through a local intermediary such as a female teacher or health worker. In some cases a survey may be all that is needed for an experienced hydrogeologist to define water sources for small community supplies and no further investigation will then be required. If essential data are lacking, some fieldwork is necessary.

The survey should provide sufficient data to form a basis for drawing up a hydrogeological map showing: the distribution of aquifers; any springs or signs of springs present; depth of water tables and piezometric levels; yield of existing groundwater sources; and the quality of the water from them. Sometimes, it is possible to prepare such a map on the basis of an examination of outcrops and existing water supplies. In other cases, it may involve the use of specially drilled boreholes and geophysics. Drilling special test boreholes will usually only be required when an aquifer is to be fully exploited and knowledge is therefore needed of the hydraulic permeability and water storage capacity.

Geophysical investigations, especially electrical resistivity measurements, are very useful in understanding the distribution and quality of groundwater. The value of the electrical resistance of a ground formation depends upon the amount, distribution and conductivity of the water it contains. Resistivity measurements are made by passing an electric current through the ground between two electrodes and measuring the voltage drop between two further electrodes (Fig. 10.6). The depth of penetration of the current is controlled by the spacing of the electrodes. Increasing the electrode spacing makes the current penetrate deeper, and so a complete resistivity depth probe can be carried out. If a depth probe is done near an existing well or borehole of which the water level, water quality and aquifer thickness are known, then the correlation between the resistivity values and the hydrogeological conditions can be established. This provides a basis for interpreting resistivity depth probes in other areas with much the same geology, to establish information on water table depth, water quality and aquifer thickness.

If resistivity measurements are conducted in a grid pattern over an area, the readings can be plotted on a grid map to form patterns of high and low resistivity for each electrode spacing

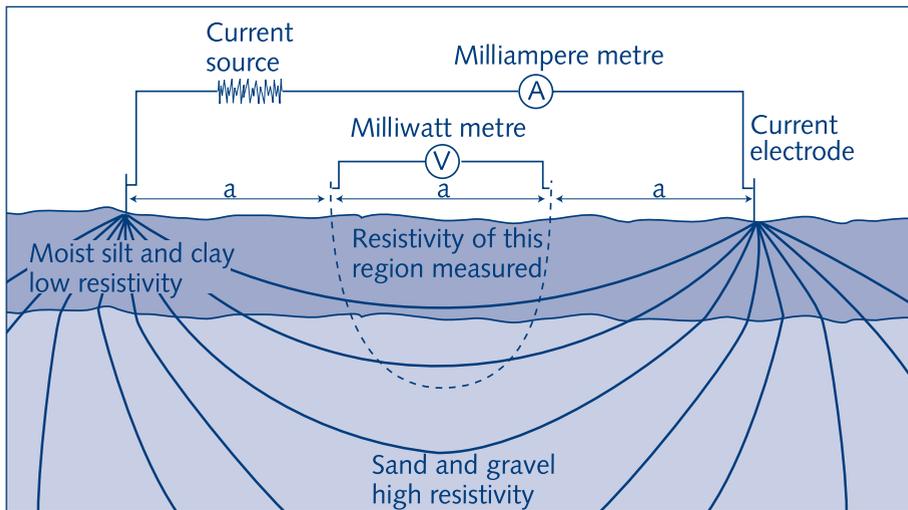


Fig. 10.6. Electrical resistivity measuring arrangement

used. Lines of equal resistivity can then be drawn on the map to highlight areas of low resistivity. These are more likely than high-resistivity areas to be permeable and water-bearing ground formations.

Sometimes it is necessary to drill small boreholes for prospecting purposes to supplement the data obtained from surface geophysical methods. This method is always expensive and often difficult to employ. To obtain the maximum amount of information from a borehole, geophysical logging may be necessary. This involves lowering measuring instruments down the borehole to record ground and water properties. Borehole logging is a complex operation and advice should always be obtained from a geophysicist before it is decided upon. Other, more sophisticated, geophysical techniques for groundwater prospecting are seismic and gravity measurements.

Annex 2 gives a detailed description of the most common methods used in groundwater exploration. Included are:

- Study of any available geological maps and reports
- Study of topographical maps (e.g. 1: 50,000 scale)
- Examination of any existing wells
- Hydrogeological survey
- Surface geophysical investigations (including electrical resistivity, seismic refraction, well logging)
- Aerial photography and satellite imagery
- Airborne geophysical methods (including magnetic, radiometric and electro-magnetic measurements)
- Test wells (drilling, pumping tests, geophysical logging, radioactivity logging, (radio isotopic) tracer investigations, chemical water testing (electric conductivity))

Safe yield

The safe yield of an aquifer is the maximum withdrawal rate that can be permanently obtained without depleting the source. Safe yield is estimated to check whether the planned withdrawal for water supply purposes will be safeguarded in the long run. Basically, the amount of water withdrawn should not exceed the natural recharge. Another limitation is that the groundwater table should not be lowered so much that polluted water from elsewhere would be drawn into the aquifer. Sometimes withdrawal of water from a new well may cause an appreciable reduction of the yield of existing wells nearby. In an area where little is known about the extent and capacity of the aquifer, the new well and any nearby wells should be monitored at least during the early period of operation. Public access to information on the maximum safe yield of wells helps reduce risks of overexploitation by the elite. Without conscious information sharing, the elite often have a monopoly over newly emerging data and may use them for personal interests. In addition, local water management organisations can be encouraged to set rules and control water extraction (see also chapter 5: Integrated Water Resources Management).

10.3 Methods of groundwater withdrawal

The oldest and simplest method of groundwater withdrawal is to dig a hole in the ground to a depth below the groundwater table. Usually the amount of water that can be collected in this manual way is quite limited. When more withdrawal capacity is needed, the aquifer must be tapped over a greater area of contact. This may be done by enlarging the width of the excavation (horizontal), by extending it to greater depth (vertical), or by increasing both the width and depth. Which of these methods can and should be applied in a particular case depends on the thickness of the water-bearing ground formation, the depth of the groundwater table and, in case of community inputs, the balance between the inputs and benefits of each method.

Horizontal means

The horizontal means of groundwater withdrawal are called galleries and may be subdivided into seepage ditches, infiltration drains (Fig 10.7) and tunnels (Fig. 10.8).

Because of the difficulties and costs of excavation, galleries should only be used in cases where the groundwater table is at a shallow depth – not more than 5-8 m below the ground surface. In consolidated ground formation, tunnels may still be economical at greater depths. Galleries offer the only practical solution when groundwater is to be withdrawn from shallow aquifers with a small saturated thickness, as these aquifers have to be tapped over a large contact area. Galleries are also recommended in coastal areas where the fresh water to be withdrawn floats on top of underlying salt water.

Drawdown⁴ of the fresh water table must then be kept as small as possible, otherwise the salt water would rise and mix with the fresh water.

Ditches are easy to construct; they can have a large capacity and a long useful life. However, ditches being open, the water collected in them is unprotected against contamination, which makes them less suited for drinking water supply purposes.

Infiltration drains and **tunnels** are more costly to build, and their design is more complicated. Drains may be subject to clogging. The advantage of drains and tunnels is that these collectors are completely underground, so the collected water is protected against any contamination from the ground surface.

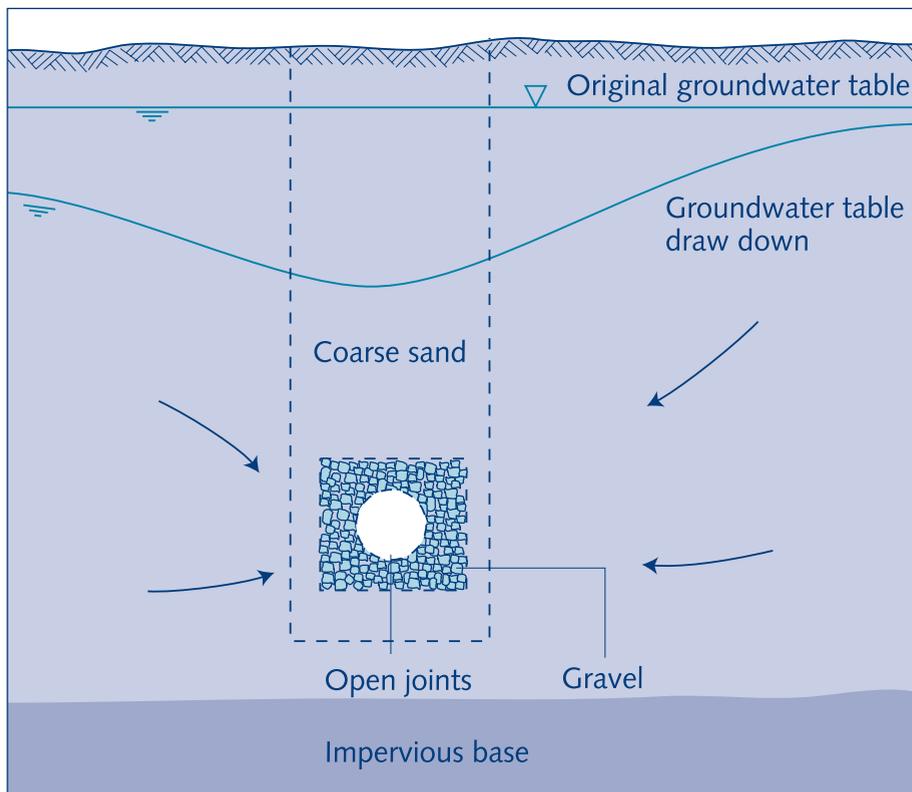


Fig. 10.7. Infiltration drain

4 Drawdown is the lowering of the groundwater table around a groundwater collector, resulting from the withdrawal of the water.

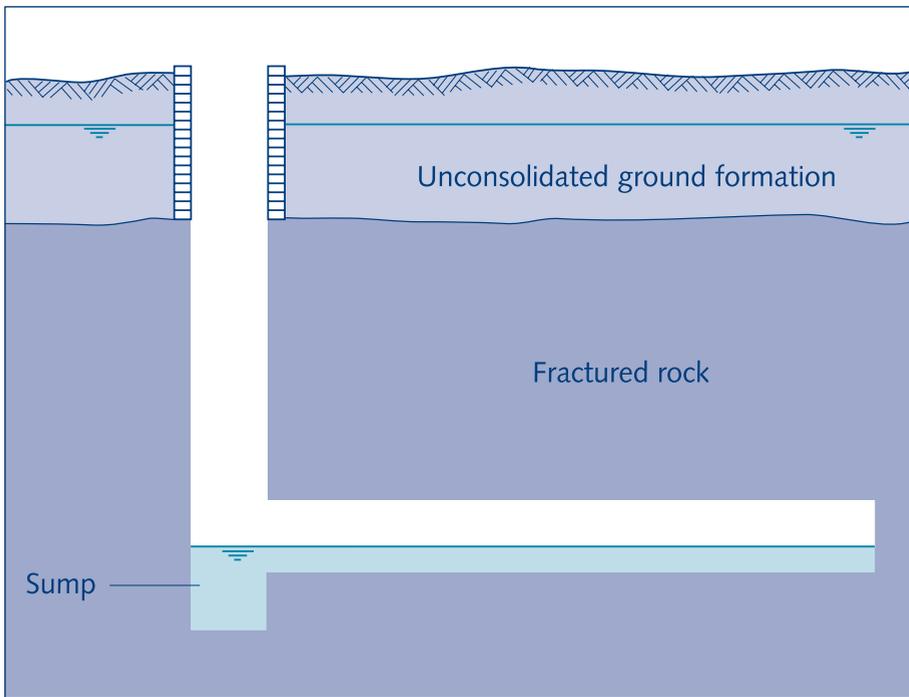


Fig. 10.8. Infiltration tunnel

Vertical means

The vertical means of groundwater withdrawal may be subdivided into large-diameter dug wells (Fig. 10.9) and small-diameter tubewells or boreholes (Fig. 10.10).

Several different well construction techniques exist and the most suitable method for constructing wells in a particular area should be selected carefully. One important factor is the type of geological formation to be penetrated. Table 10.2 provides general guidance on well construction methods. The physical location of a groundwater well is influenced by many factors including hydrogeology, security (to prevent vandalism) and susceptibility to pollution (from pit latrines, flooding, or physical damage). Social requirements, for example easy access for the different user groups and resolution of conflicting demands, also play a significant role in acceptability, use and maintenance.

Dug wells usually have a limited capacity so that their use is restricted to individual household and other small-scale water supplies. The large-diameter shaft acts as a storage reservoir and thus provides for any peak withdrawals. Boreholes should be used when the groundwater table is at a considerable depth below the ground surface, but they are only effective in aquifers of sufficient thickness. The capacity of boreholes varies over a wide range, from less than 1 l/s for small-diameter wells in fine sand aquifers, to over 100 l/s for large-diameter wells in coarse sand or sedimentary rock deposits. Boreholes are very well suited for drinking water supplies because simple precautions will

Table 10.2 Applicability of well construction method

Method	Maximum depth (m)	Diameter (cm)	Geological formation	
			Suitable	Unsuitable
Dug	60	90-500	Clay; silt; sand gravel; soft sandstone; soft, fractured limestone	Igneous rock
Bored	25	5-40	Chalk; gravel; soft sandstone soft; fractured limestone; alluvial formations	Igneous rock
Driven	15-20	3-5	Clay; silt; sand; fine gravel; sandstone (in thin layers)	Any formation with boulders, cemented gravel, limestone, igneous rock
Jetted	80-100	10-30	Clay; silt; sand; pea gravel	Any formation with boulders, cemented gravel, sand stone, limestone, igneous rock
Sludged	50	3-10	Clay; silt; sand; gravel soft sandstone; fractured limestone; alluvial formations	Any igneous rock formation
Percussion-drilled (cable tool)	300	10-60	Clay; silt; sand; gravel; cemented gravel; boulders (in firm bedding); sandstone; limestone; and igneous rock	None
Rotary-drilled (fluid circulation)	250	10-60	Clay; silt; sand (stable); gravel; cemented gravel; sandstone; limestone, and igneous rock	Problems with boulders
Rotary-drilled borehole (down-the-hole air hammer)	250	10-50	Particularly suitable for dolomite; basalts; metamorphic rocks	Loose sand, gravel, clay, silt, sandstone

be adequate to safe-guard the water against contamination. Sometimes, a battery of boreholes in series can be pumped as one unit (Fig. 10.11). Because of the large differences in withdrawal capacity of boreholes and wells, and the fact that withdrawing water from a borehole needs a handpump or other pumping device, the choice between the two options is best made with those who will use and sustain the facility.

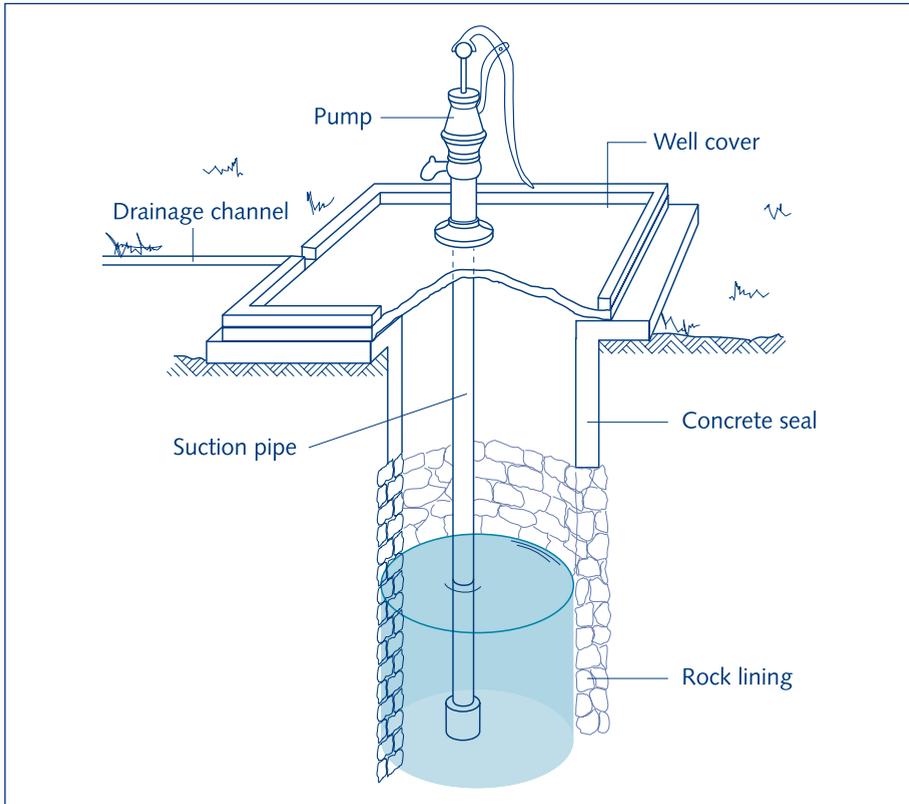


Fig. 10.9. Dug well

In situations where a thick water-bearing ground formation is present at shallow depth, either vertical or horizontal water collectors, or a combination of these, can be appropriate. The technical feasibility will largely depend on the local geological conditions. A much more difficult situation exists when groundwater has to be withdrawn from a thin aquifer situated at a considerable depth. In view of the small saturated area of such an aquifer, boreholes should not be used. Ditches and drains are not appropriate since they would require an excessive amount of excavation work. Sometimes, in consolidated ground, tunnels may be suitable. For unconsolidated ground, radial collector wells may be considered (Fig. 10.10). However, such wells require specialist design and construction and they are, therefore, generally less suited to small-scale water supplies.

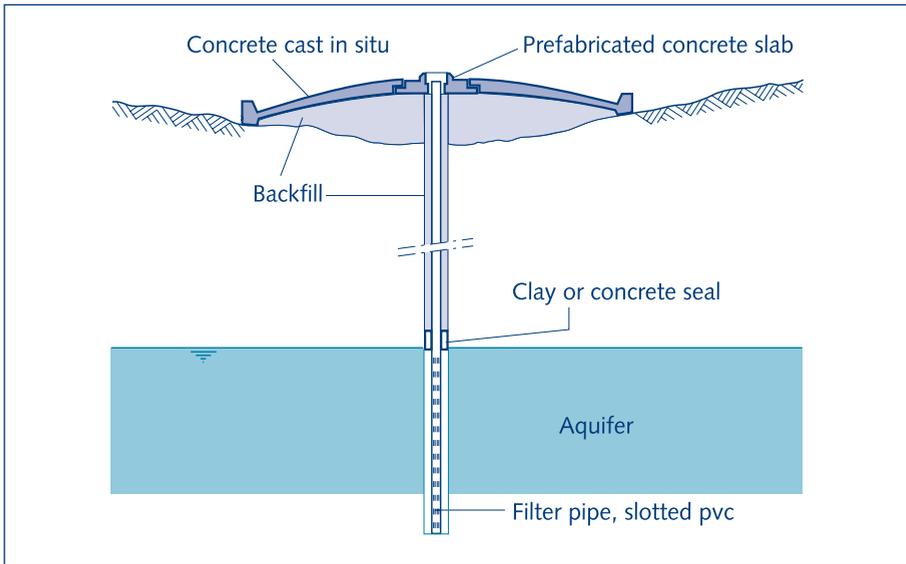


Fig. 10.10. Borehole

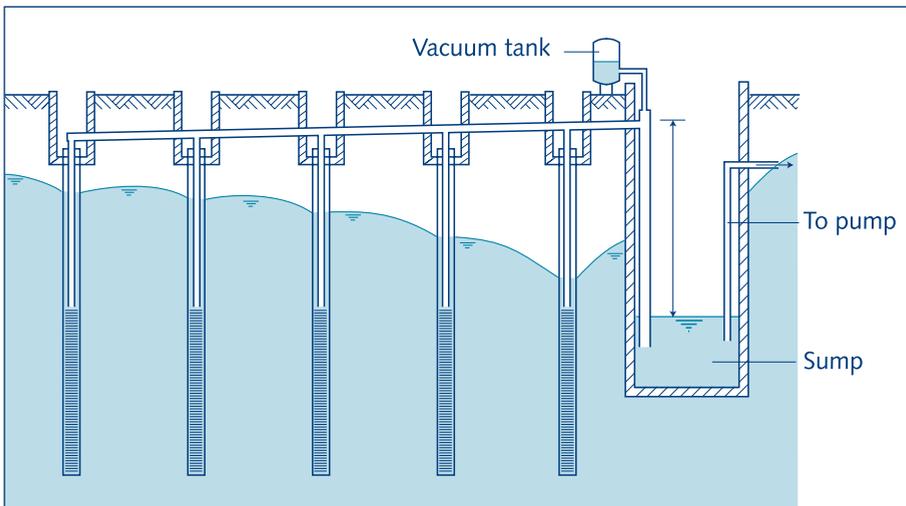


Fig. 10.11. Battery of boreholes

When groundwater is withdrawn there is always a lowering of the groundwater table. In principle, all other withdrawals from the same aquifer are influenced. The effect of groundwater withdrawals for community water supply is usually not great but for the high-rate withdrawals that are frequently made for irrigation purposes, the possible effect of an appreciable lowering of the groundwater table should be carefully investigated. It may be necessary to carry out a test pumping to provide a basis for estimating the future drawdown of the water table (Fig. 10.13). In areas where motor pumps for irrigation are becoming common, safety areas may also need to be planned and managed by the community.

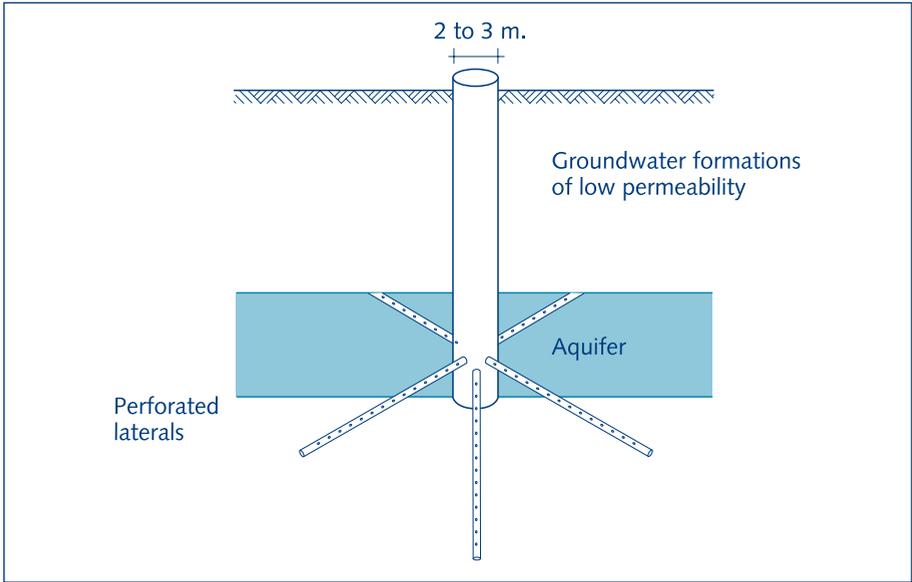


Fig. 10.12. Radial collector well (also called "Ranney Well")

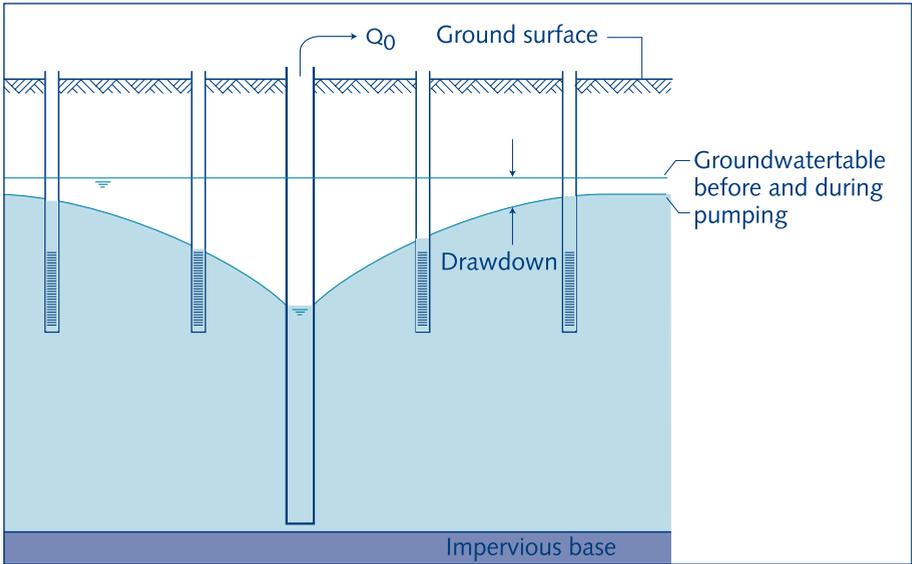


Fig. 10.13. Test pumping

10.4 Infiltration galleries

Ditches for groundwater withdrawal are just a cut in the ground to make the aquifer accessible from the surface. They are easy to construct either manually or with mechanical equipment. The design will also present few problems. The most important requirements are the following (see Fig. 10.14):

- The width and depth should be sufficient to ensure that the collected water flows at a low velocity (usually less than 0.1 m/s) so as to prevent erosion of the ditch sides and to limit the head losses.
- The depth should be greater than 1.0 m and preferably 1.5 m to reduce any penetration of sunlight into the water where it would stimulate plants and algae to grow and cause resistance to the flow of the water.
- The ditch sides should slope gently to provide stability. This is particularly important for the ditch side-water surface contact area.
- For deep ditches, a horizontal embankment about 0.5 m above the normal water level is desirable to facilitate access for cleaning and maintenance work.

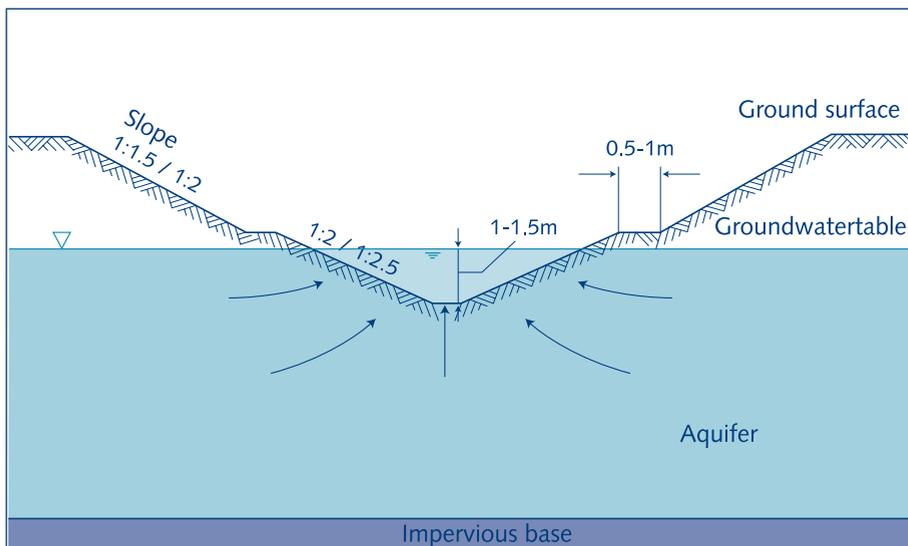


Fig. 10.14. Design of seepage ditch

Seepage ditches being open, the groundwater collected in them is subject to pollution, bacterial contamination and algal growth.

Drains (Fig. 10.15) have pores, perforations or open joints allowing the groundwater to enter. Porous drains may be made of materials such as clay or no-fines concrete (using a pea-size gravel and cement mixture, without sand). Perforated drains are mostly of vitrified clay baked in a kiln, or made of plastic or wood. Drains with open joints are usually made of concrete.

The choice of material for a particular drain construction depends on the required strength, the corrosion resistance needed for the type of groundwater to be collected, and above all on costs and availability. Perforations in the drain need only be made all round it when the drain is placed completely in the aquifer. For drains laid in the upper part of an aquifer, perforations in the underside will be adequate and for drains deep down in the aquifer only upward-facing perforations are needed.

In coarse ground formations such as gravel, the drain openings can easily be made small enough to keep back the ground material. In fine and medium-sized sand, perforated drains and drains with open joints should be packed in one or more layers of gravel or coarse sand, to prevent the fine sand of the aquifer from entering the drains. The outside layer should be fine enough to keep back the aquifer material; the inside layer has to be of a size that is somewhat larger than the drain openings. For an aquifer of sand with an effective size of about 0.2 mm, the gravel pack could consist of two layers, each about 10 cm thick, with grain sizes of 1-2 mm and 4-8 mm. Drain openings about 3 mm wide may then be used. When drains with open joints of 10 mm are used, a third gravel pack layer of 15-30 mm grain size is necessary.

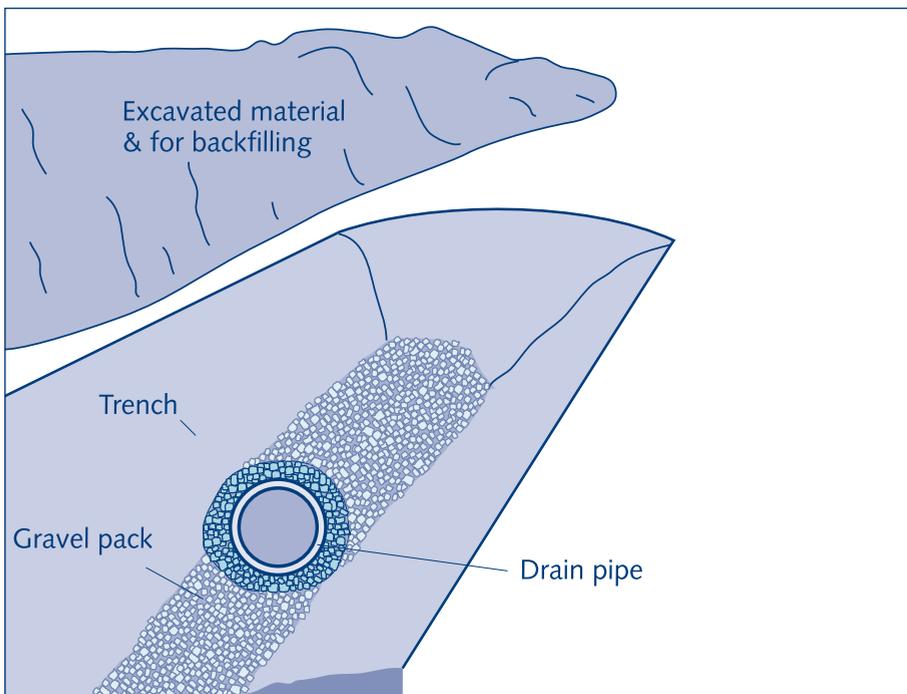


Fig. 10.15. Drain construction

The most important factors in the design of a drain are the internal diameter of the drainpipes, and the depth at which the pipes and gravel pack are placed below the groundwater table.

In spite of the gravel pack, some suspended matter may get into the drain. If this material is allowed to accumulate it will block the drain. To prevent this, the drains should be so sized that the flow velocity is sufficiently high to flush out any silt deposits. For drains to be self-cleaning, the velocity should be higher than 0.5 m/s but it should not be more than 1.0 m/s, or friction losses will be too high. This would cause an uneven drawdown and withdrawal of groundwater along the length of the drain. To accommodate the accumulating quantity of water collected and flowing through the drain, it may be necessary to provide incremental sizes of the drain along its length.

To keep down excavation costs, the drains should be laid no deeper in the ground than necessary. However, the drains must remain fully submerged in the groundwater with the top of the gravel pack at least 0.5 m deep, even at the end of a long dry period when the groundwater table is likely to be at its lowest level. Using the existing groundwater table as a basis, the designer should allow for an operating drawdown of at least 1 m, plus a further drop of the groundwater table of 1 m under dry conditions. The top of the gravel pack thus should be at a depth of 2.5 m or more under the existing water table. If communities contribute voluntary labour, they will need to know about the required minimum depths and the reasons. A simple measuring tool in the shape of an inverted T helps local committee members monitor the proper depth and width of the trench. When iron and manganese is present in the groundwater, there is a serious risk of iron and manganese deposits clogging the drain openings and gravel pack. It is then necessary to lay the drains deeper, some 4-5 m under the existing water table, to prevent oxygen from penetrating to the drains and forming the iron and manganese deposits.

10.5 Dug wells

Dug wells are made simply by digging a hole in the ground. They are widely used in many countries and can be quite satisfactory if conditions are right. Usually no special equipment or skills are required for their construction.

Experience shows that the diameter of a dug well should be at least 1.2 m if two men are to work together at the bottom of the well during the digging. For a well serving a single household or a small community this minimum diameter is usually adequate but when more people are dependent on a dug well, a larger well, 2-3 m in diameter, must be provided. Further increasing the size of a well is seldom useful since the additional water yield so obtained is likely to be very small.

Due to their large diameter and volume, dug wells provide both groundwater withdrawal and storage. Because of the storage capacity, water can be temporarily withdrawn at a higher rate than the recharge inflow into the well. The storage effect is particularly important when the users take the water mostly at peak rates during a few

hours in the morning and the evening. Dug wells have also the advantage that users can still get access to the water through the manhole in case the pump breaks down. These and the next points are important aspects to consider with the representatives of male and female user groups during technology choice.

The depth to which a well can and should be dug largely depends on the type of ground and the fluctuation of the groundwater table. Important factors are the stability of the ground and the costs of digging. Private wells are generally less than 10 m deep. Dug wells for communal use are frequently much deeper; 20-30 m is not unusual and depths of 50 m and more have been achieved.

Most dug wells need a lining. For this, various materials are used such as brick, stone, masonry, concrete cast in a shuttering inside the hole, or pre-cast concrete rings. The lining serves several purposes. During construction, it provides protection against caving and collapse and prevents crumbling ground from filling up the dug hole. After completion of the well it retains the walls. In consolidated ground (e.g. rock) the well may stand unlined but a lining of the upper part is always to be recommended (Fig. 10.16). In unconsolidated ground formations the well should be lined over its entire depth (Fig. 10.17). The section of the well penetrating the aquifer requires a lining with openings or perforations enabling the groundwater to flow into the well.

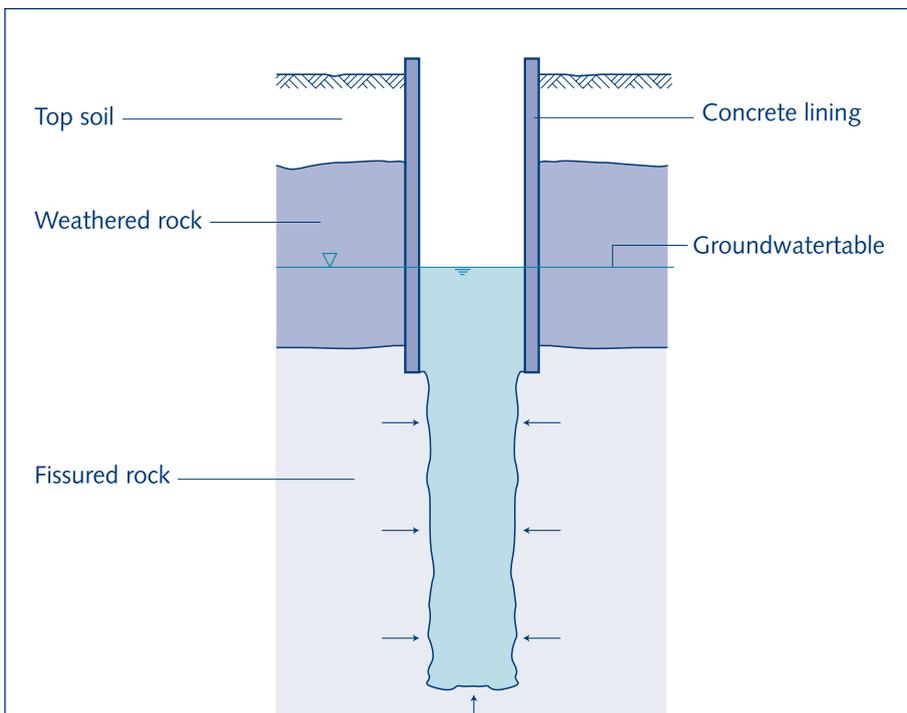


Fig. 10.16. Dug well in rock formation

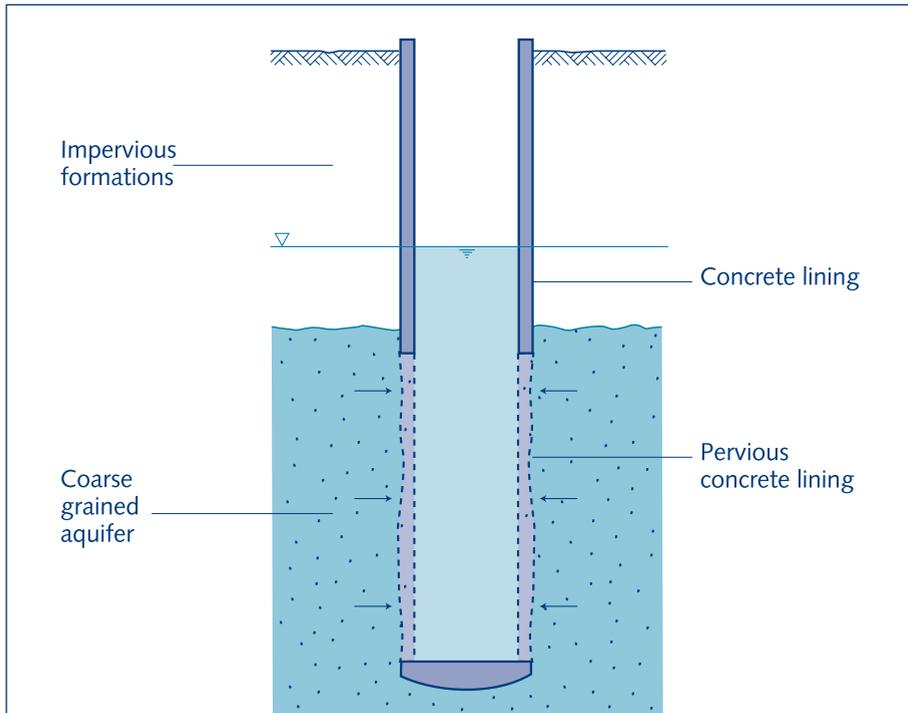


Fig. 10.17. Dug well in coarse granular material

In fine sand aquifers it is impossible to provide a lining with openings or perforations small enough to retain the fine sand and prevent it from passing into the well. In such cases the lining is frequently extended over the entire depth of the well without any openings or perforations. The groundwater enters the well only through the bottom, which is covered with several layers of graded gravel keeping down the fine sand of the water-bearing formation (Fig. 10.17). For example, three layers of graded gravel, each 15 cm thick, may be used with grain sizes of 1-2 mm for the deepest layer, then 4-8 mm, and 20-30 mm effective size at the top.

Lining a dug well will also provide a seal against polluted water seeping from the surface into the well. This is not so effective if the well is open, because the water in it will then be polluted anyway, especially if the water is drawn using buckets and ropes. As a minimal provision, the well lining should be extended at least 0.5 m above the ground to form a head wall around the outer rim of the well. A concrete apron should then be constructed on the ground surface, extending about 2 m all around the well. The concrete apron also seals any fissures between the well lining and the walls of the excavated hole and so prevents polluted surface water from seeping into the well.

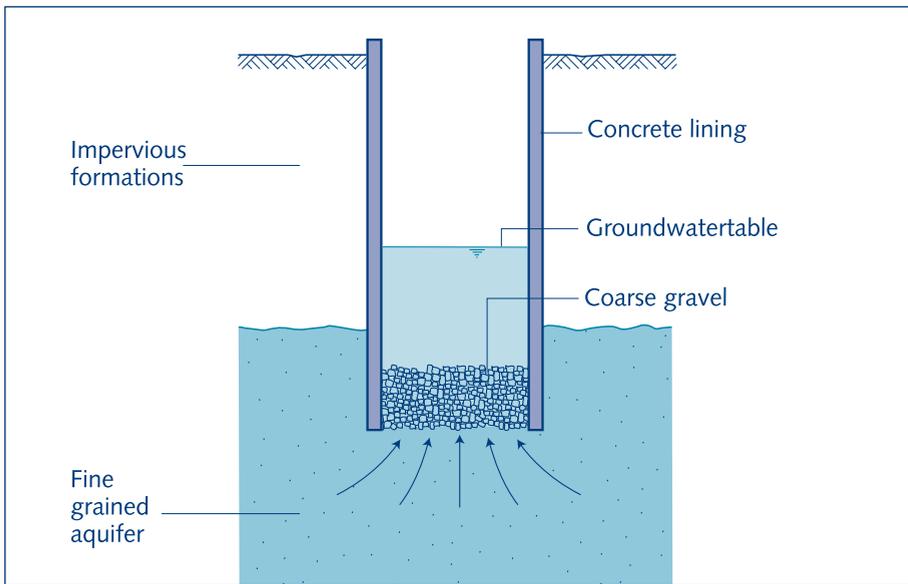


Fig. 10.18. Dug well in fine granular aquifer

All these measures only have a limited effect if the well remains open. A well cover reduces direct contamination by dust and unwanted objects but the bucket and rope will still contaminate the water body. This can be reduced when users decide to clean the well site regularly and even pave it, say with shells, and see to it that the rope and bucket do not trail in the mud but are hung on a bucket stand when not in use.

Full protection of the bacteriological quality of the water from a well can, however, only be obtained if the well top is completely sealed with a watertight slab on which a pump is mounted to draw the water (Fig. 10.19). A manhole that can be tightly and securely locked should be provided to allow disinfection of the water in the well by chlorination. Although simple in itself, the sealing of dug wells is not always feasible; particularly where the standard of pump installation is poor and maintenance requirements cannot be adequately met. Therefore, discussions with local men and women are needed on water quality protection and sanitary measures for the direct well environment against water and soil pollution.

Dug wells are sometimes constructed in a temporary excavation, drained and braced against caving as necessary (Fig. 10.20). Any type of building material may be used. For economy, strength and stability circular walls are to be preferred. Masonry and brickwork are widely used; (reinforced) concrete is also popular, pre-fabricated or cast on the site. When burnt bricks are used, the possible negative impact of the use of local firewood on the environment and on women's work in fuel collection has to be considered. Open joints are used at intervals in masonry and brickwork linings, to enable the groundwater to flow into the well. In concrete linings short pieces of tin tube

or garden hose can be cast to provide openings. To avoid the entrance of polluted water from the ground surface, backfilling should be done in thin layers that are firmly compacted.

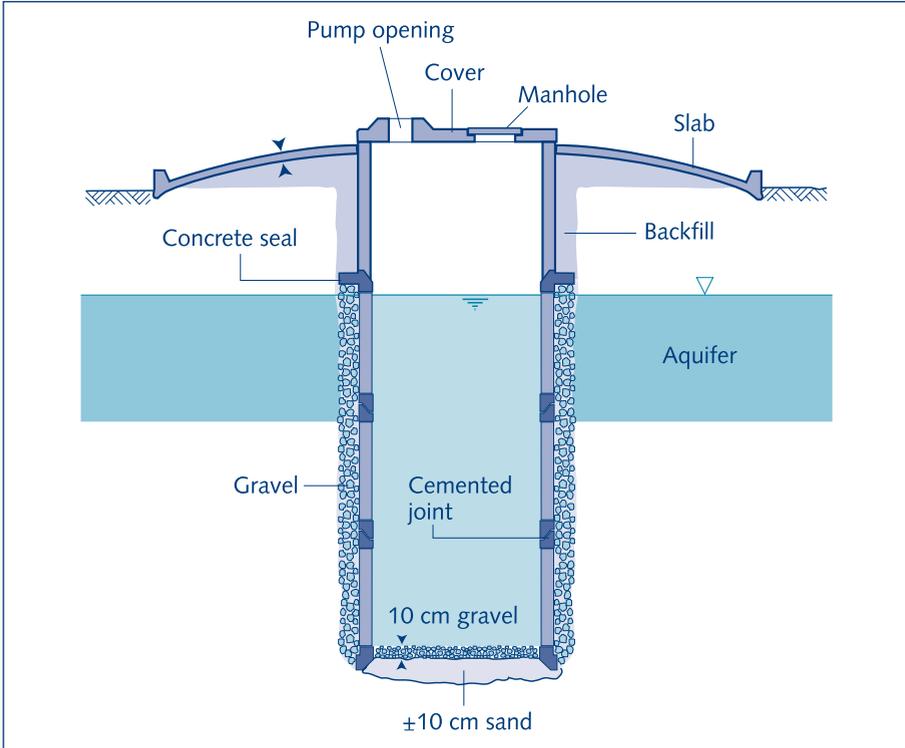


Fig. 10.19 Dug well sealed for sanitary protection

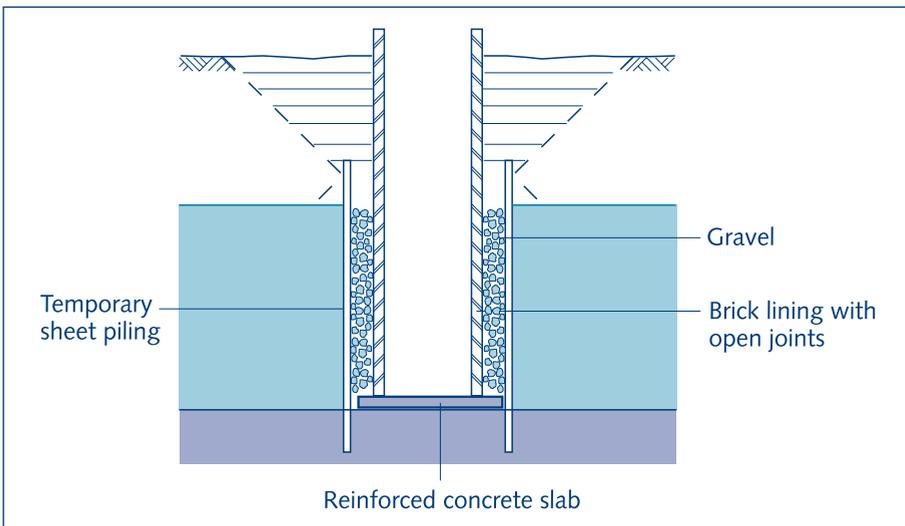


Fig. 10.20. Dug well built in temporary excavation

Stiff consolidated formations requiring no immediate support for stability allow the temporary excavation to be executed as an open hole with unsupported walls. However, it is prudent to carry out the digging section by section as shown in figure 10.21.

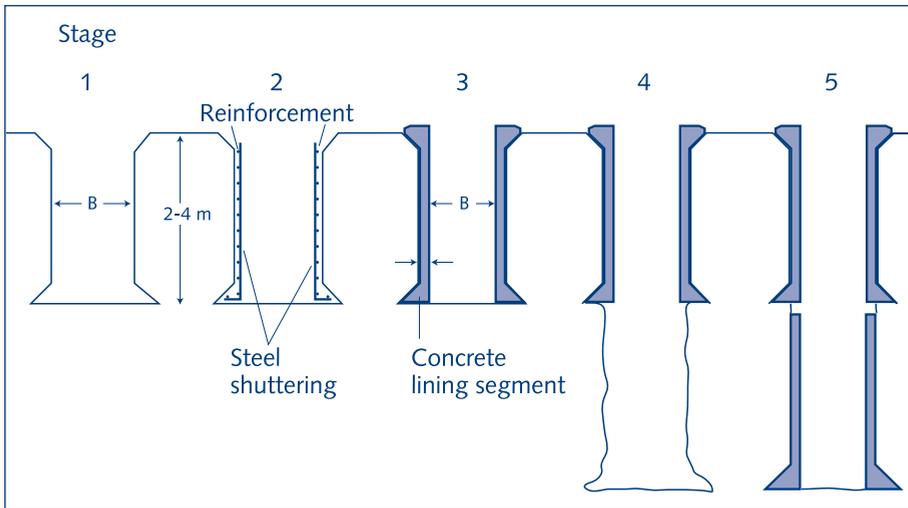


Fig. 10.21. Reinforced concrete lining built on site

Each section should be 2-4 m high and is kept in place by the surrounding ground pressing against it. The most common method of constructing a dug well is by excavation from the inside, removing the ground at the bottom. The lining then sinks down due to its own weight (Fig. 10.22). For wells of a diameter up to 3-4 m the digging frequently is carried out with hand tools. Below the groundwater table de-watering of the well becomes necessary to enable further excavation to be carried out. In this construction method, circular well shapes are mostly used because they settle readily and are not liable to deformation when the well lining sections are subjected to uneven forces.

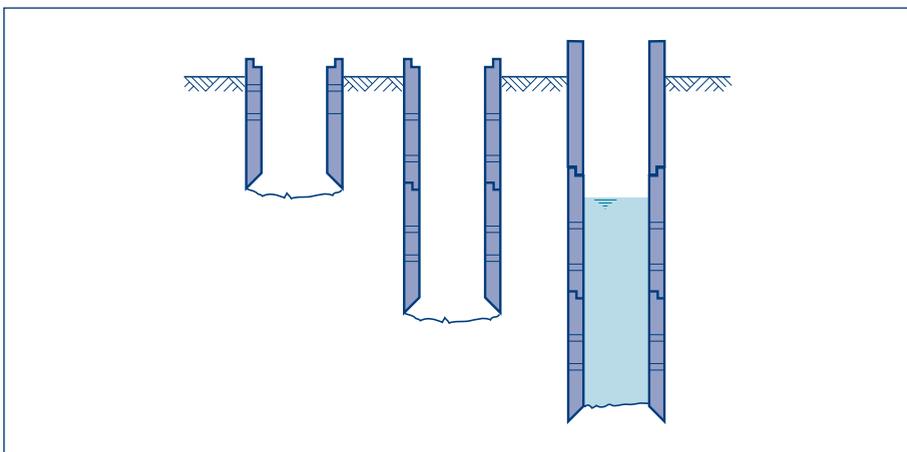


Fig. 10.22. Sinking a dug well by excavation from the inside

Masonry work of stones, bricks or concrete blocks can be used to build the well lining using a strong steel shoe as the base (Fig. 10.23). The shoe prevents the lining from settling unevenly, which could cause deformation and cracks. Reinforced concrete obviously is a more suitable construction material in this respect. It also allows the well lining to be constructed above ground as the well sinking progresses. Large-diameter pipes of concrete or plastic may be used for well lining material. When these are not available or are too expensive or difficult to handle, prefabricated concrete rings may be employed to form the lining (Fig. 10.24). There is no valid reason why prefabricated concrete rings should not be widely used. They require no special materials or skilled masons and need only aggregates and simply trained unskilled workers to cast them. The necessary sand and gravel may usually be obtained in the neighbourhood of the well site.

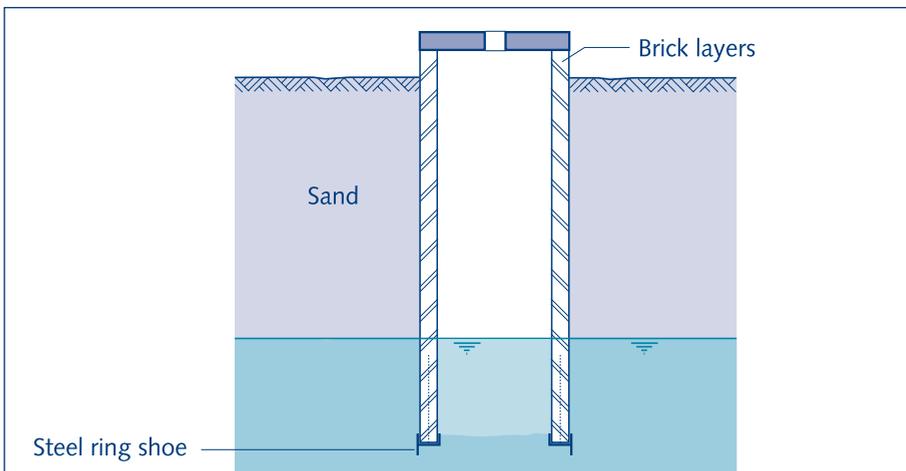


Fig. 10.23. Dug well with brickwork

The lower end of the starter ring is provided with a shoe having an inside cutting edge; the outside diameter is somewhat larger to facilitate the sinking and to reduce ground friction along the outside (Fig. 10.25). The starter ring during sinking leaves a space around the lining. In loose formations this space will be self-sealing but in cohesive formations it must be filled with cement grout or puddled clay as a safe-guard against seepage of polluted water from the ground surface. Over the depth of the aquifer, the rings are made of no-fines concrete (pea-size gravel and cement, without sand) through which the groundwater can enter the well.

Frequently a more economic and technically better construction may be obtained by combining the two methods of construction described above. The construction of figure 10.26 gives an excellent protection against any ingress of polluted seepage water from the surface; it also allows the well to be made deeper when after some time the groundwater may fall to a lower level. The design shown in figure 10.27 does not have this advantage but it costs much less to construct.

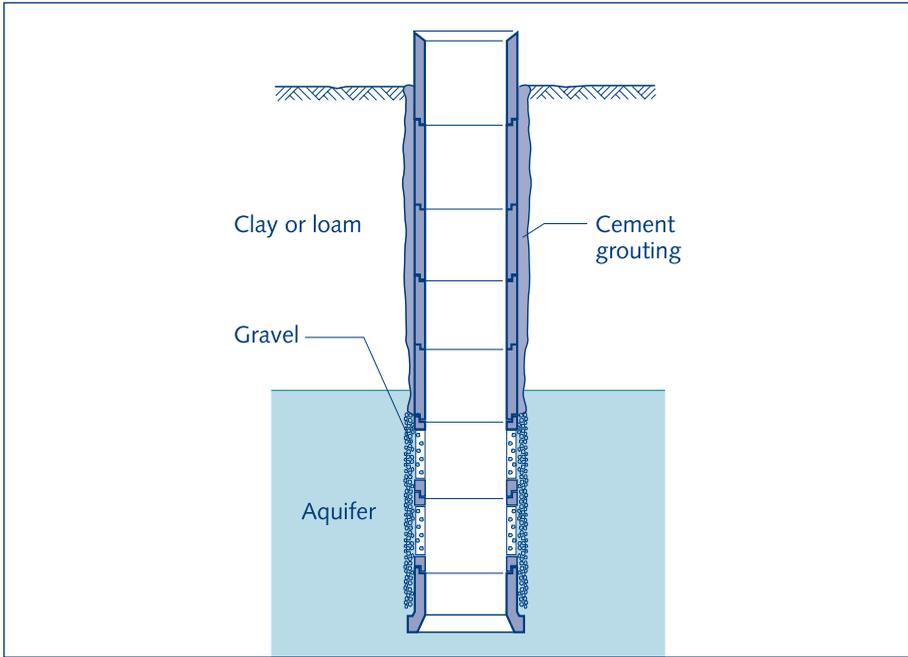


Fig. 10.24. Dug well construction with prefabricated rings

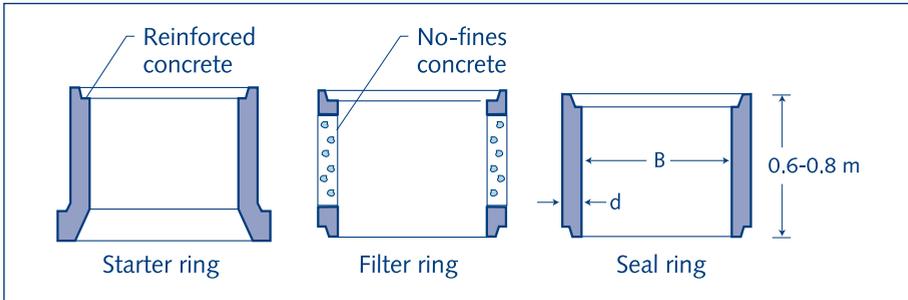


Fig. 10.25. Prefabricated concrete rings

It will be clear that it is not so easy to protect the water of a dug well against bacterial contamination. To sum up, the following precautions are recommended:

- The upper part of the lining should be water-tight preferably to a depth of several metres below the lowest drawdown water level in the well.
- The space between the walls of the dug hole and the lining should be sealed with puddled clay, or better with cement grout.
- The top of the lining should extend some 0.5 m above ground level and should be topped with a watertight cover on which a (hand) pump is to be mounted for drawing the water from the well.
- An apron should be constructed around the raised top of the lining (the head wall) about 2 m wide, sloping outwards and with a gutter draining any spilt water away from the well site.

- The water in the well should be chlorinated for disinfection after the well has been completed. This should be repeated at regular intervals.
- Pros and con's of different options are discussed with women and men and when an open well is decided on, they are encouraged to work out and institutionalise a strategy for maximising and preserving good hygiene.

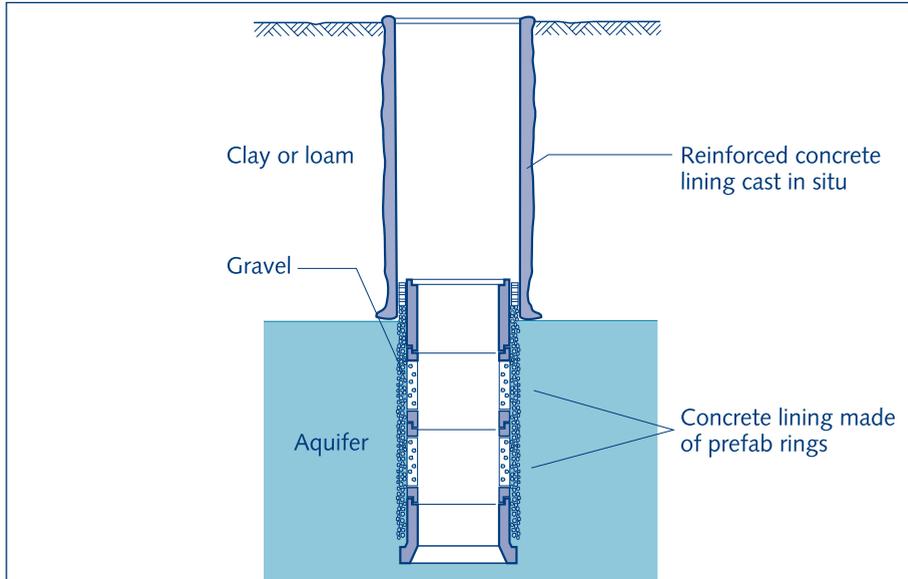


Fig. 10.26. Dug well construction using combination of methods

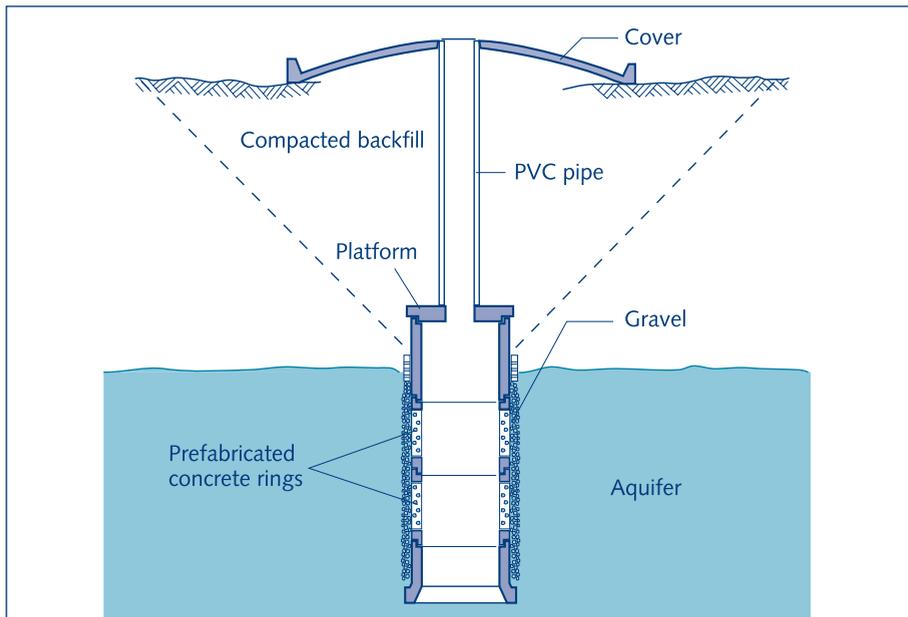


Fig. 10.27. Dug well construction using combination of methods

10.6 Boreholes (Tubewells)

Introduction

A borehole (tubewell) has a casing consisting of pipes (tubes) in the non-water bearing formations, and perforated or slotted screen sections in the aquifer.

Boreholes of small diameter and shallow depth for small capacity water supplies may be constructed by driving, jetting, boring or sludging. Drilling is more versatile and appropriate for larger-diameter boreholes designed for the withdrawal of considerable amounts of water at greater depths, or for tapping aquifers that are overlaid by hard rock or similar ground formations. It does, however, require complicated equipment and specialist drillers with adequate knowledge and experience.

Boreholes can be constructed to 200 m or deeper depending on the method used. Particularly in arid or semi-arid regions, the depth of wells into shallow bedrock and unconsolidated aquifers should accommodate seasonal or annual fluctuations in the water table to avoid drying up in periods of low water table. However, deeper boreholes are more costly to construct and maintain. Drilling norms for particular regions must therefore be established from existing knowledge of drillers and users. There never is a guarantee that water will be found. It is, therefore, important to make full use of any available prospecting and exploration data when choosing the site where a borehole is to be constructed. The assistance of experienced and qualified people is essential, together with the information obtained through hydrogeological surveys.

The completion diameter of a borehole should be optimised taking into account the anticipated yield, the abstraction requirement, the drilling methodology and the type of pumping equipment to be used. For example, a borehole designed for high abstraction urban supply will require a larger drilling and completion diameter (200-300 mm) to accommodate the necessary pumping equipment, to facilitate the higher flow rate, to minimise groundwater inflow velocity and to maximise borehole efficiency. On the other hand a borehole designed for rural handpump supply in which both yield and flow rate are low can be completed at a diameter as small as 75 mm if slim-line pumping equipment is available. In unconsolidated or other unstable formations (boulders, gravel, etc.) several sets of casing of reducing diameter may have to be used to support the borehole as drilling progresses. The requirement to install filter (gravel) pack material alongside screens will also mean increased drilling diameter.

The risk of introducing chemical or biological pollution to the groundwater body via the borehole itself is becoming increasingly important, especially in environments with rapid human development. Installation of an adequate sanitary seal of suitable material between the permanent casing and the natural ground of the borehole wall prevents

direct ingress of surface or near-surface pollutants via this relatively open pathway. Flooding or agricultural pollution protection may require special designs.

Borehole construction is greatly influenced by local factors and relatively unknown underground conditions. Several drilling and construction techniques have been developed for use in these different environments. In many cases, the most modern and expensive drilling rig and tools is not necessarily the best equipment. Careful consideration should therefore be given to the degree of technical sophistication of methods adopted and local knowledge considered. A detailed overview of borehole drilling methods is given in annex 2.

Driving

Driven wells (Fig. 10.29) are made by driving a pointed screen (called a *well point*) into the water-bearing formation. To prevent damage to the well point when driving through pebbles or thin layers of hard material, the point at the lower end of the screen is made of solid steel, usually with a slightly larger diameter than the screen itself. As driving proceeds and the well point sinks into the ground, successive sections of pipe are screwed on top so that the upper end of the casing is always above the ground surface. The well point is driven into the ground using a simple mechanism for hitting the top of the pipe. Many arrangements can be used. Figure 10.30 is indicative. Whichever method is used, it is essential to ensure that the blows are square and vertical; otherwise the pipe will bend and perhaps break. As it is the pipe that transmits the blows to the well point, strong thick-walled piping must be used, particularly when difficult driving in hard formations is expected.

In the well driving method shown in figure 10.31, the drive bar falls free inside the screen. The pipe is pulled into the ground rather than driven so that pipe of normal strength classes can be used. Driven wells are especially suitable for soft sandy formations that are readily penetrated by the well point. Driven wells cannot be made in areas where boulders or other obstacles are encountered in the ground. In all ground formations the resistance against driving increases with depth. The application of driven wells is therefore limited to shallow wells of less than 10-15 m depth. For the same reason the diameter is usually small, varying from as narrow as 3 cm to a maximum of about 10 cm, a diameter of 5-8 cm being the most common. Well pumps cannot be installed inside such small diameters. Driven wells also have the disadvantage that the screen openings may become clogged with clay or similar material during the driving. Removing this clogged material is almost impossible after the completion of the driven well.

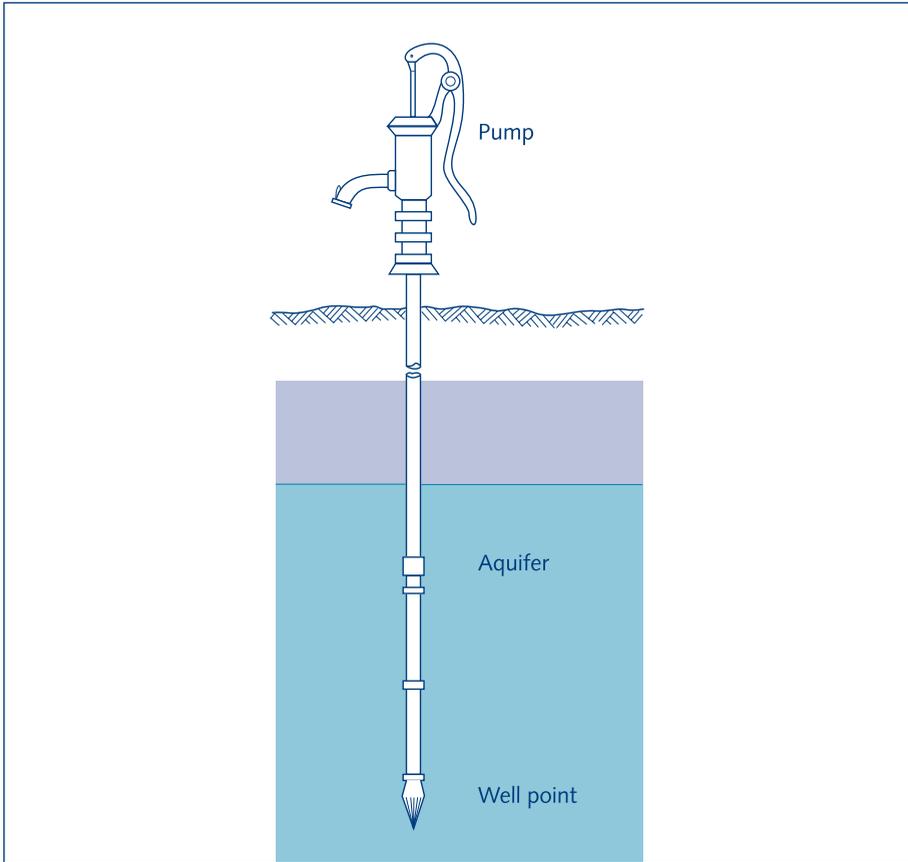


Fig. 10.28. Driven well

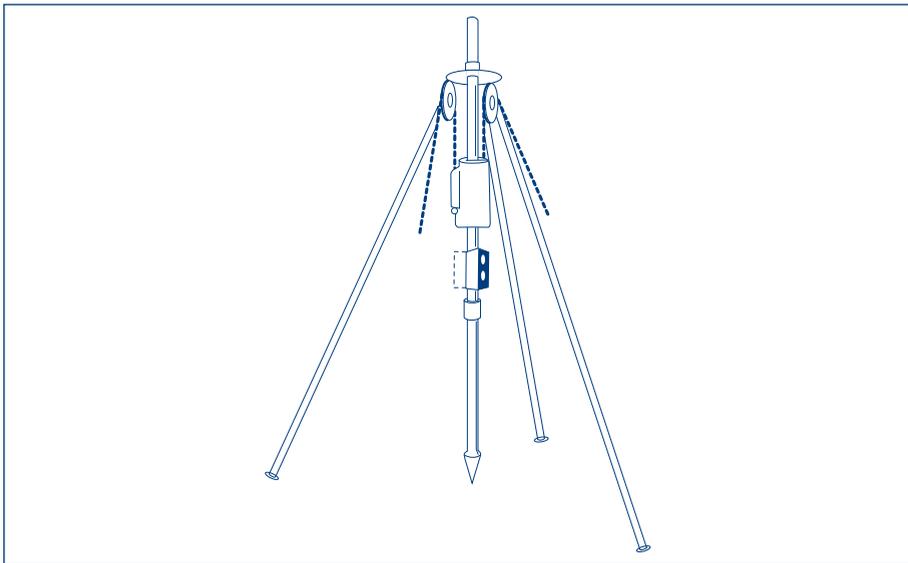


Fig. 10.29. Well driving arrangement

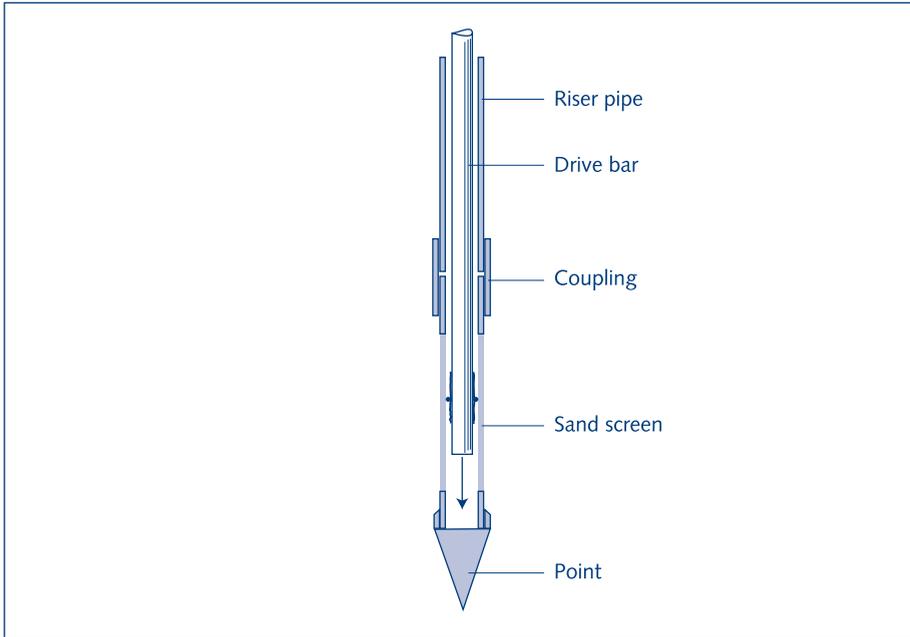


Fig. 10.30. Well driving with inside drive bar

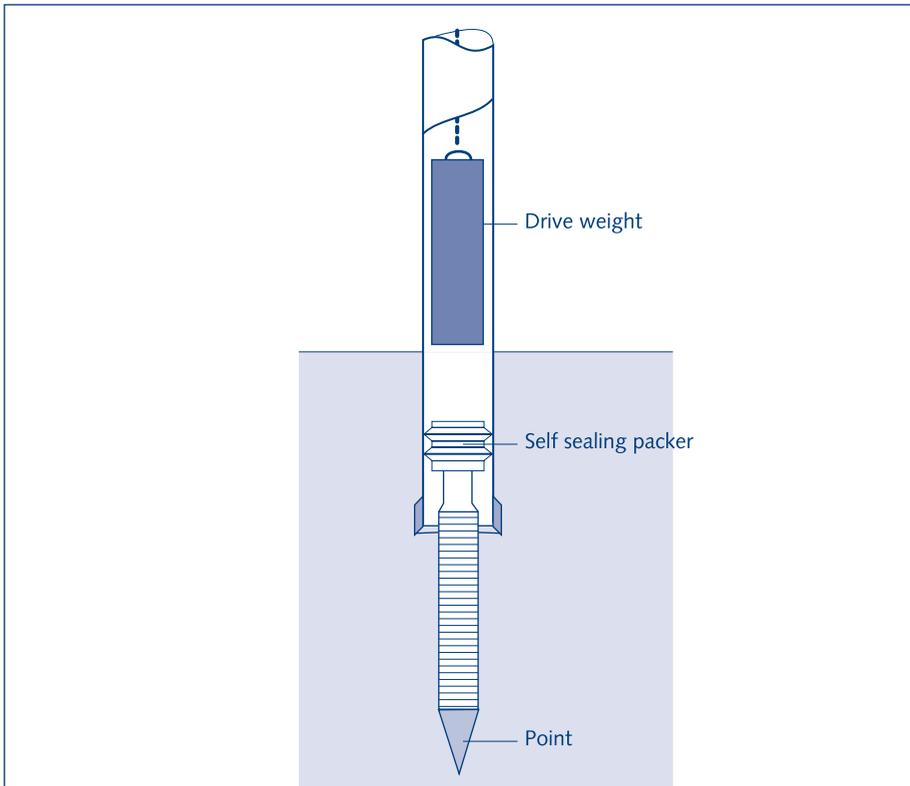


Fig. 10.31. Well drive point with sliding joint

The use of a sliding joint can prevent the clogging of the well screen (Fig. 10.31). During driving the screen is inside the casing and only after reaching the desired depth is it then forced out to intrude the water-bearing formation. When there are hard formations directly below the ground surface, a better solution is to start by boring a hole slightly larger than the well point before driving (Fig. 10.32). When the hole is straight, vertical and deep enough this also helps achieve a plumb well, which otherwise may be difficult to obtain.

If, after completion of the driven well, the inside is thoroughly disinfected, the water from it will be bacteriologically safe and is likely to remain so.

However, the yield from a driven well is usually small, in the range of 0.1 to 1 l/s. This will only be sufficient for private household use or a small community. For a larger supply of water, a number of driven wells may be interconnected with a central suction line and pumped as one unit, but this solution is rather expensive. In rural areas of developing countries driven wells have the advantage of easy and rapid installation with no need for specialised equipment or skills.

Jetting

Jetted wells do not differ much from driven wells but the point at the lower end of the screen is hollow instead of solid, and the well is bored through the erosive action of a stream of water jetting from the point (Fig. 10.32).

Compared with driven wells, jetting of wells is much faster. Mechanical force is not needed so that plastic instead of steel can be used for casing and strainer. Obviously, jetted wells can only be sunk in unconsolidated formations. Sandy aquifers are best suited for this method; clay and hardpan often offer too much resistance to the water jet stream. As with driven wells, boulders cannot be passed but it is a simple process to check the underground formation beforehand by washing the jetting pipe shown in figure 10.33 to the desired depth. Such a jetting pipe is also used in a well jetting technique using a separate jetting pipe to wash the plastic casing and screen into the ground. Compared with driven wells, the depth that can be obtained is somewhat greater, for the same diameter of about 5-8 cm. Clogging of the well screen openings is generally not a problem.

Auger Drilling

Auger drilling (boring) is a totally mechanical method of sinking a borehole. Its primary area of application is for drilling shallow wells in soft and unconsolidated formations such as sand and soft limestone soils, and it is especially appropriate in more clayey overburden that will stand without caving. Mechanical well sinking is an old method of drilling and large-diameter augers were first used over 75 years ago when horses were

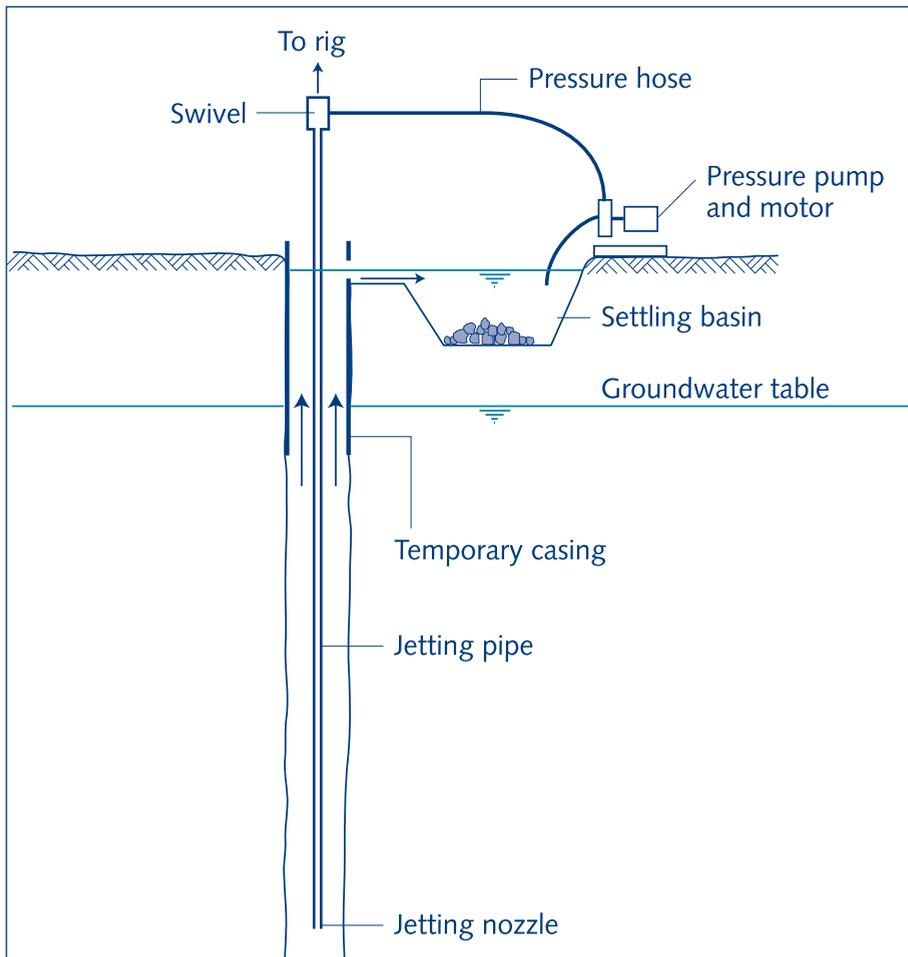


Fig. 10.32. Water jetting

used to provide the motive power. The deepest wells recorded at that time were in the 100-110 m range and were lined with masonry. The auger tools were normally taken down until a caving formation was reached, using an iron or steel *shoe* used to cut a clearance for masonry added at ground level. Auger drilling is not suitable for hard-rock areas, and formations with cobbles or boulders can create difficulties.

The auger is rotated from the surface by means of a drive shaft built up from steel rod sections 3-6 m long and connected by quick-acting couplings. The upper part is called the kelly and has a square cross-section to receive the necessary torque from a rotating table. At the bottom, the auger is provided with a cutting face, which peels the soil from the hole and discharges it into the cylindrical chamber above. When full, the auger is drawn up above ground and the hinged bottom is opened. Each time this is necessary, the drive shaft must be dismantled and coupled together again, a tedious and time-consuming job.

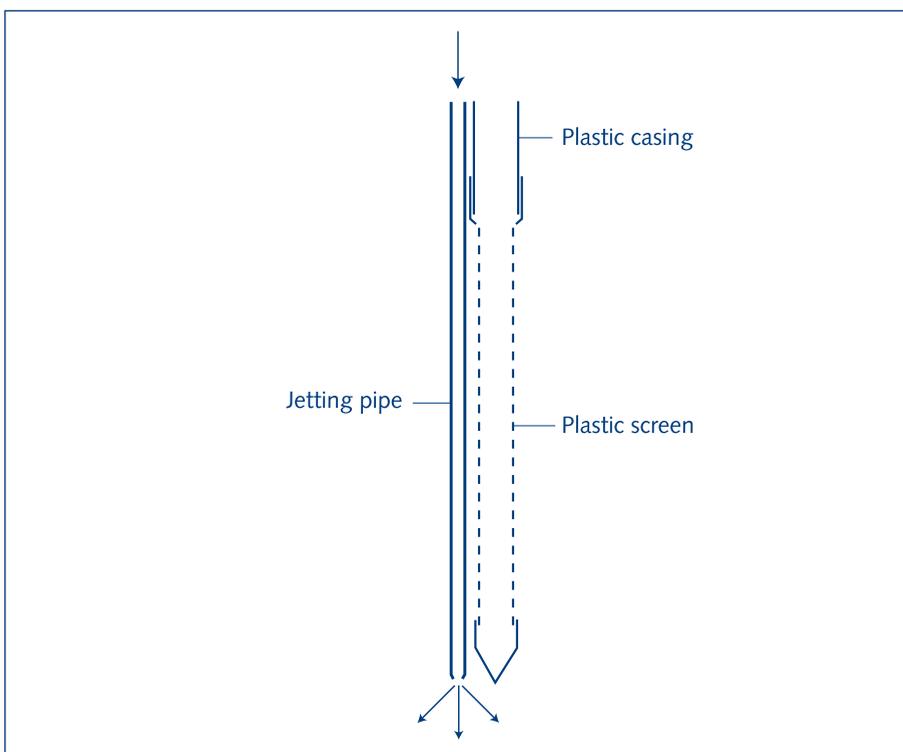


Fig. 10.33. Well jetting with an outside jet pipe

Under the water table, the auger does break up the ground layers but it cannot bring the bored material to the surface as the cuttings escape when the auger is pulled up from the bottom of the hole. A bailer is then lowered in the hole with a cable to collect the cuttings. The bailer is moved up and down near the bottom of the hole; during the down stroke the cuttings are entrapped by a closing valve. The whole operation greatly increases the time required for the boring of a well.

The principal types of auger tools in common use are:

- *Hand auger.* Utilises spiral augers for shallow depth and bucket-type augers for greater depth (Fig. 10.34). Hand auger drilling is suitable for shallow depths at diameters less than 200 mm, and is especially suitable where labour costs are low and low yield (narrow diameter) handpump boreholes are prevalent.
- *Large-diameter bucket auger.* Utilises a cylindrical bucket with auger-type cutting blades on the bottom, rotated into the ground on the end of a long kelly. As the hole progresses the bucket must be withdrawn frequently for emptying. Bucket augers can drill holes up to 1 m in diameter and can excavate below the water table, but have considerable problems with cobbles and boulders that must be removed using other specialist tools.
- *Solid stem 'continuous flight' auger.* Uses a continuous spiral of hard faced flights welded onto a small diameter pipe with individual auger sections joined by solid

hexagonal pins, making the tools non-hollow (hence *solid stem*). A cutter head with hardened teeth or blades some 50 mm larger in diameter than the auger flight cuts ground material, and the cuttings are brought to the surface by the flights that act as a screw conveyor. Continuous flight augers can penetrate rapidly in suitable materials but provide poor formation samples and are unsuitable for use below the water table.

- *Hollow-stem auger.* Appears similar to the solid stem auger, but the flights are welded to a larger diameter pipe such that smaller drill rods can also pass through the centre tube. A plug with attached bit is usually situated in the centre of the cutter head and attached to the internal drill rods that are rotated at the same rate as the outer auger flights. The inner diameter of the hollow stem can be as large as 330 mm and the auger tools can be used as temporary casing whilst a shallow water borehole complete with screens, casing and gravel pack is constructed in the central hollow stem. The depth of penetration of this drilling method is limited to between 15 and 35 m depending on the outside diameter of the augers. This drilling technique is widely used in foundation investigations.

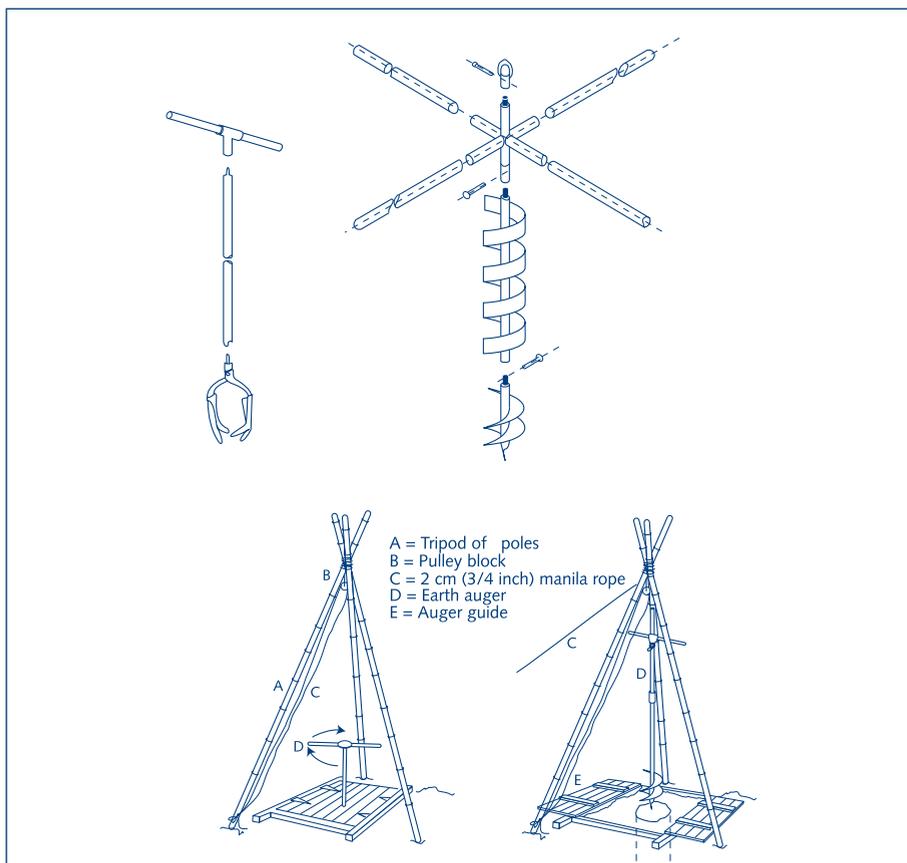


Fig. 10.34. Hand boring equipment

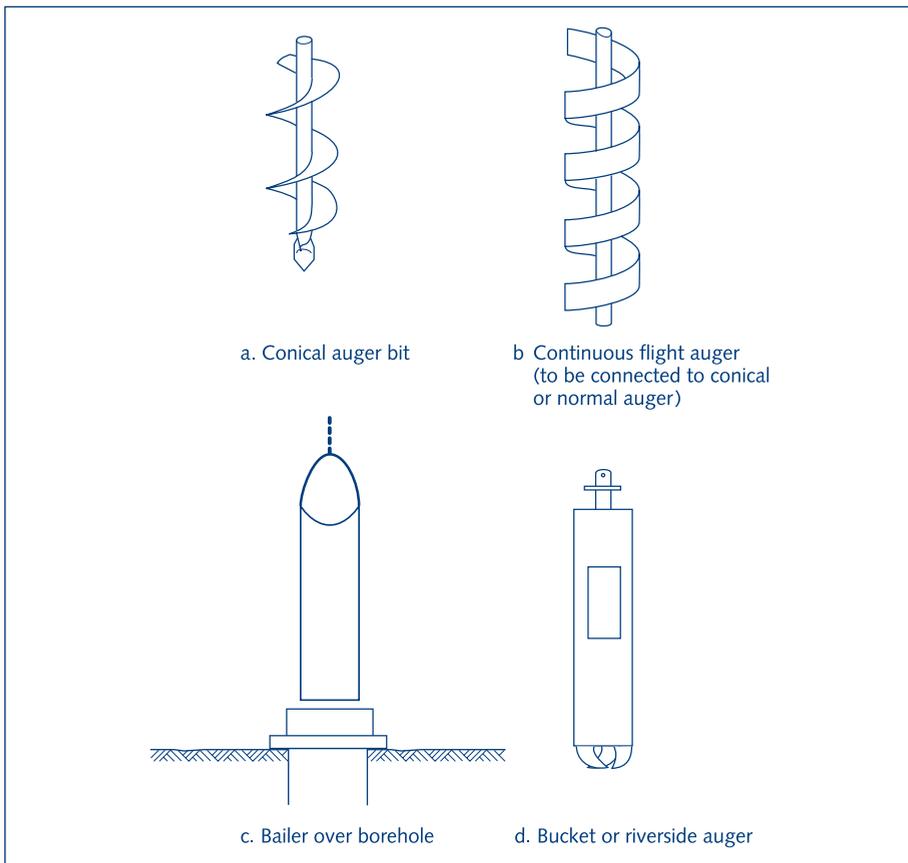


Fig. 10.35. Different auger types

Sludging

The *sludger method* is an indigenous, low-cost, labour-intensive technique for sinking boreholes in unconsolidated alluvial ground formations such as those found in deltaic areas. Boreholes with a depth up to 50 m may be constructed using this method under suitable conditions. In Bangladesh the sludger method has been and continues to be extensively used for sinking numerous boreholes to tap the abundant, shallow groundwater resources present in that deltaic country.

To start the drilling operation, a hole with a diameter of about 0.6 m and 0.5 m deep is made and water is poured into it. Some bamboo staging is erected above the hole. A piece of steel pipe is placed vertically in the soil. The drilling is carried out by moving the pipe up and down with a jerking action while the steel pipe is filled with water. For this, a (bamboo) rafter fastened to the pipe and supported from the staging is operated (Fig. 10.36). At the foot of the drill pipe, soil loosened by the water enters into the pipe, allowing the drill pipe to penetrate into the ground. As a result of the jerking action of the drill pipe, the loosened soil and water is pushed upwards and comes out through the top of the pipe.

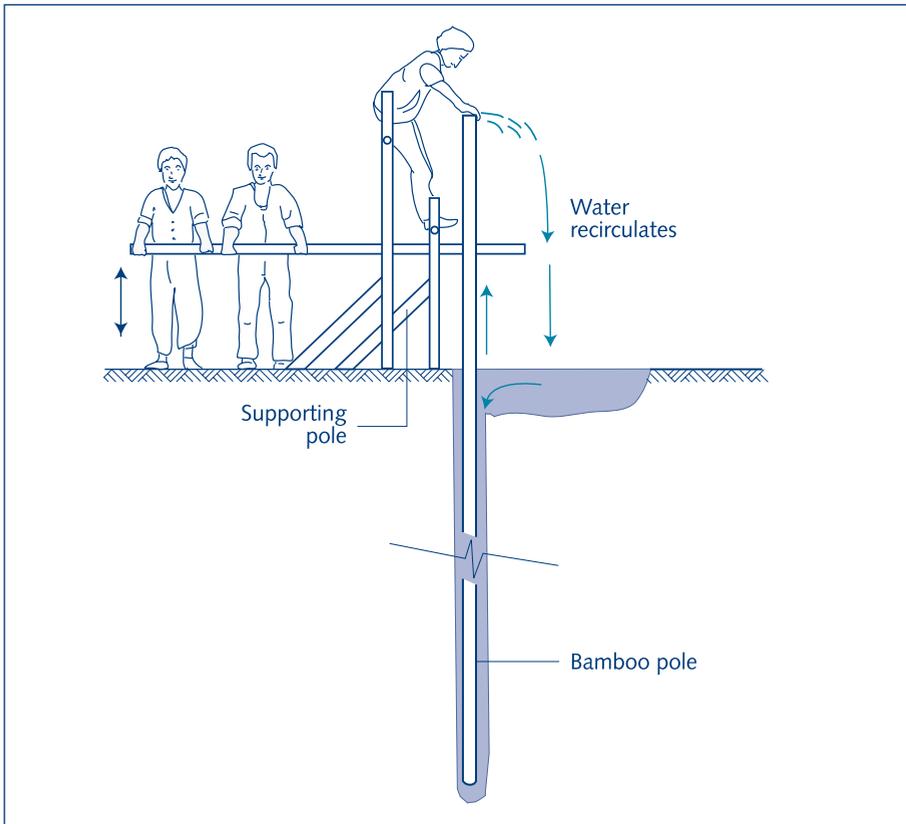


Fig. 10.36. Sludging technique
 Source: *Waterlines, Technical Briefs*, <http://www.lboro.ac.uk/well/resources/technical-briefs/43-simple-drilling-methods.pdf>

During the well sinking, one man sits on top of the staging and takes care that the hole is drilled perfectly vertically. At each upward stroke, he closes the top of the drill pipe off with his hand, which introduces a suction action. This assists the loosening of the soil at the pipe bottom and the forcing up of the drilled soil. More pieces of pipe are added as the string of drill pipe sections penetrates deeper and deeper in the ground.

As the well sinking proceeds, soil samples are collected from the mudflow coming out at the top of the drill pipe. These are taken at each 1.5 m the drill pipe is sunk further, and then examined. The drilling operation is stopped when good water-bearing formations are penetrated sufficiently. The whole length of pipe is withdrawn piece by piece taking care to keep the drilled hole intact. Immediately after withdrawal of the drill tubes, the well casing consisting of plastic pipes complete with strainer sections is fitted and lowered in the hole up to the determined depth.

An improved version of the sludging equipment for wider application, called the *pounder rig*, has recently been developed and tested in Uganda with good results.

Cable tool percussion drilling

Percussion drilling is one of the two principal drilling methods. It uses a percussive force to break the rock via a hardened drill bit (either by means of lifting and dropping, or by means of an air-driven reciprocating piston). Cable tool percussion drilling is a very old method and was already in use more than 1000 years ago in China. The method basically has not changed, but the tools have been vastly improved. It is practicable for drilling both small and large diameter holes to depths as great as 300-500 m. Cable tool percussion drilling uses relatively inexpensive equipment, but its greatest disadvantage is that it is very slow.

The principle of cable tool percussion drilling is that a heavy drill bit is lifted and dropped to crush the rock and thus work its way down into the formation (Fig. 10.37). The string of tools consists of a high-carbon steel drill bit (chisel) surmounted by a drill stem (sinker bar), perhaps drilling jars, and a swivel rope socket into which the wire rope is secured.

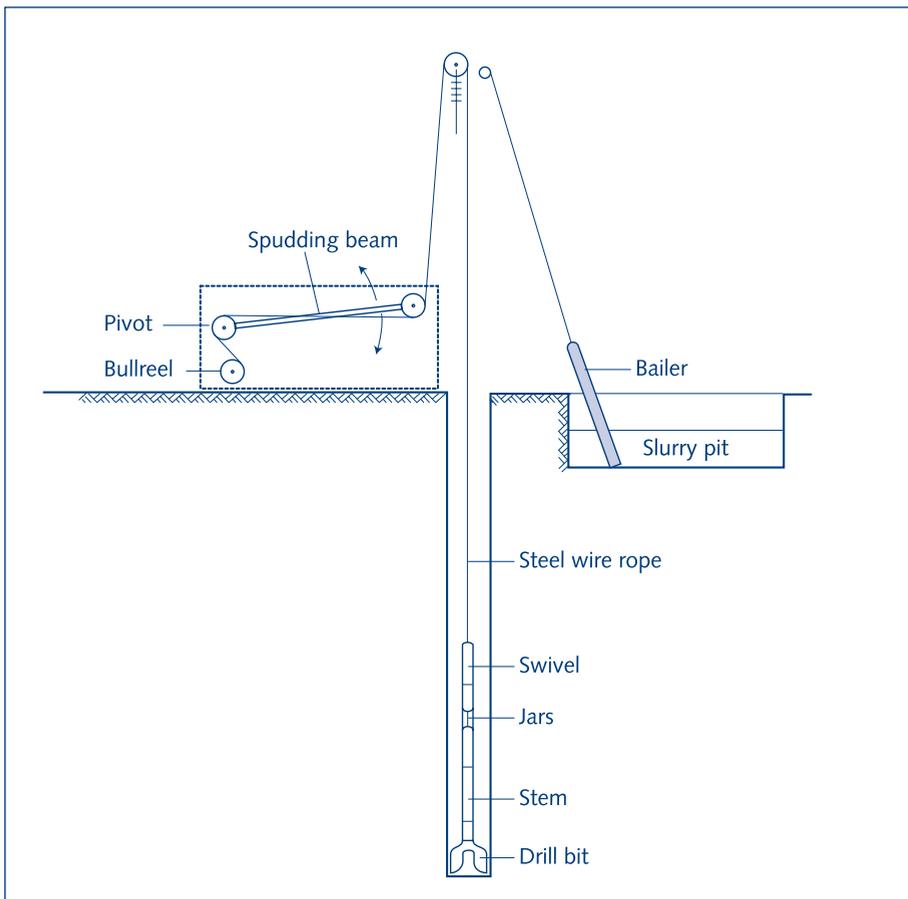


Fig. 10.37. Cable tool or percussion drilling rig

Bits have different shapes depending on the hardness of the formations, and are required to penetrate, crush, mix and ream. The drill stem provides weight and directional stability, and by its pumping action in the borehole it moves the cuttings upward, away from the bit. The tools slowly rotate, moving the bit to a new position on each stroke and ensuring a circular well.

The drilling process, depending on the diameter and depth of the borehole, consists of a column of tools weighing between 300 and 4000 kg reciprocating at 40-80 times per minute at a stroke of between 0.4 and 1.2 m, and slowly rotating. Drilling with a cable tool requires a skilled driller to progress speedily and with the right rotation of the drill bit. Water is fed to the well in small quantities until a natural supply is reached to produce slurry out of the cuttings and to suspend this material above and away from the bit face, but also to cool and lubricate the tool string. The drilling proceeds until the driller feels the thickening slurry retarding the tools, and cleans the hole by lowering a bailer several times. When drilling begins, a short guide tube or conductor pipe is always drilled or hand-sunk in a truly vertical alignment into the ground to stabilise the ground around the working area and to start and maintain a vertical hole. In the past it was common practice to drive casing down to the required depth using a *drive-head* and a *drive-shoe* on the bottom of the casing. However, recovery of temporary (costly) casing was often impossible, even using heavy jacks. The more recent use of flush-jointed temporary casing and less forceful methods of installation has ensured that temporary casing recovery is now the rule rather than the exception.

Down-the-hole (DTH) hammer drilling

The introduction of the air percussion hammer drill marked a significant step forward in the development of drilling tools suitable for constructing boreholes in hard formations. The principal advantages of this method are its speed of penetration and the use of air as a flushing medium to remove rock cuttings from the hole. Typical penetration can be as much as 4-6 m per hour in granite or gneiss, and since no water is required for flushing, this method is especially suited for water-scarce areas.

The air percussion hammer drill works on the same principle as the familiar road drill, with a pneumatic air-actuated single piston hammer operating a cutting bit fitted with hard metal inserts or replaceable 'buttons' attached to the end of a string of drill pipe. Compressed air is circulated down the drill pipe, operating the hammer piston and venting through airways in the bit to carry cuttings to the surface via the borehole annulus.

The bit drills with a frequency of between 500 and 1000 blows per minute. The air is thus driving the piston and is only released through ports in the bit when most of its energy has been expended. The released air now cools the bit, clears the cuttings at the bit face and propels them up the annular space to the ground surface (Fig 10.38).

Hammers with bits of 50-375 mm diameter are widely used, and tools have also been developed to drill up to 750 mm.

A key disadvantage of air percussion methods is that in unconsolidated ground or clays the hammer action is curtailed, the material is simply compressed rather than broken, and penetration is minimal. A minor seepage of groundwater will cause hole-cleaning difficulties since the cuttings will congeal and stick to the wall of the hole, though this can be relieved by the injection of water and surfactant into the air supply.

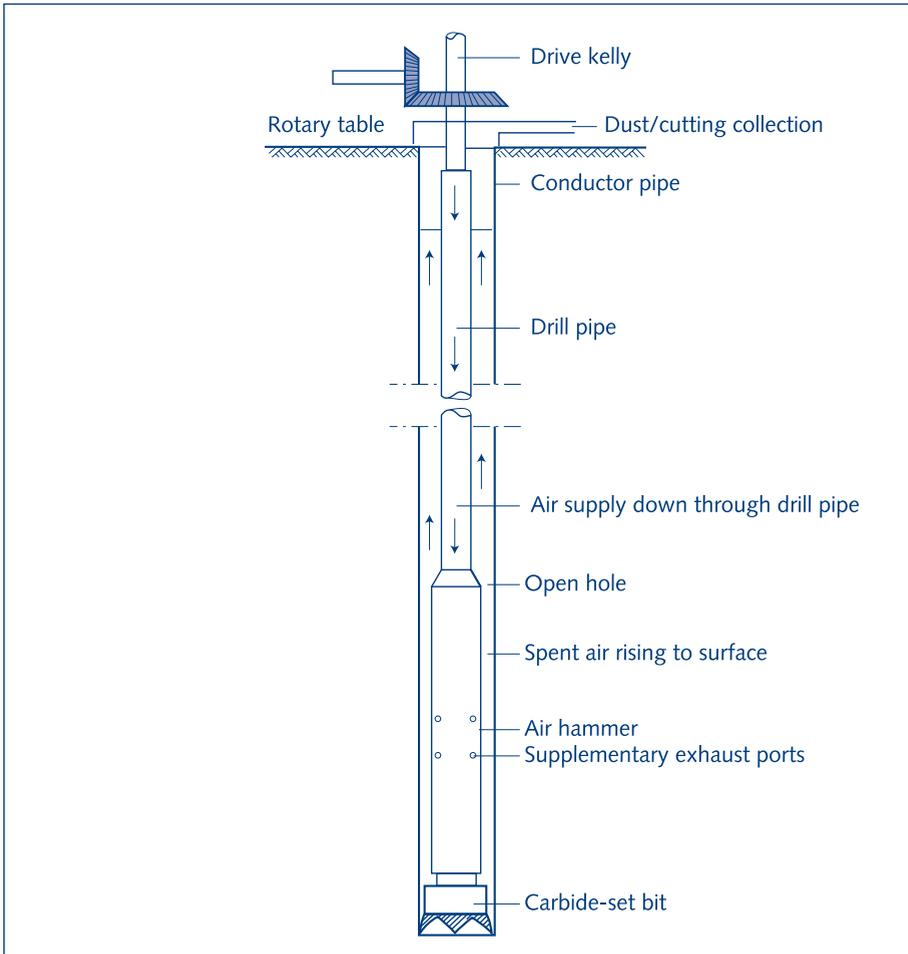


Fig. 10.38. Down-the-hole hammer drill

Direct fluid circulation rotary drilling

Rotary drilling is the second principal drilling method. It relies upon the breaking up of rock material by abrasion and crushing action rather than percussion. The drilling bit is rotated and applies considerable downward force on the rock that grinds down and breaks up the formation. Continuous circulation of a flushing fluid then removes the

cuttings and the loosened ground from the hole. Rotary drilling is particularly suitable in loose ground formations and soft rock, and can be used to drill large-diameter holes to considerable depths. The flushing medium is extremely important in this process. It may be air or water with or without additives to increase viscosity.

The principal components of a rotary drilling rig are similar to those of an air percussion rig, with a mast (or “draw-works”) used to raise and lower the drill pipes and bit, a rotation mechanism to rotate the drill string and a pumping system to circulate the flushing fluid (Fig. 10.39).

The rotation mechanism may either be a fixed *rotary table* or a *top head drive*, with the latter being more commonly applied in water borehole drilling. In the case of a rotary table rig the uppermost section of the drill string is called the *kelly*. The kelly is of special heavy-duty construction that transmits rotary drive from the rotary table to the drill string.

The drilling fluid, previously clay based or nowadays more usually degradable polymer based, is mixed in a *mud pit* or tank and pumped at high pressure through a flexible hose to the top of the rotating drill string via the kelly. It then flows through the centre of the string to the bottom of the borehole, out through the ports in the bit, and returns to the surface and back into the mud pit via the annular space.

At the bottom of the drill string is the drill bit. The type of bit used depends on the nature of the ground, with drag bits commonly used in unconsolidated or soft formations and roller cutter bits applied in more competent strata. The roller bit is rotated at speeds of 3–30 rpm, depending on borehole diameter and strata. Drag bits carry no rollers but have three or four hard-faced blades and are used to cut soft strata in a manner similar to a wood auger. They penetrate rapidly in soft, unconsolidated materials but tend to stress the drill pipe, over-tighten the tool joints and penetrate very slowly if used in harder strata.

Apart from the common verticality and straightness constraints, rotary drilling requires a continuous weight to be placed on the bit in order to effect penetration. Typically this downward force (*weight*) is within a range of 250–2750 kg per 25 mm of diameter, and is normally applied by the use of *drill collars*. A drill collar is a heavy-walled length of drill pipe that is added to the drill string immediately above the bit, this concentration of weight helping to keep the hole straight and to maintain penetration. The main drill string consists of externally flush drill pipes manufactured of seamless tubing in lengths of between 3 and 10 m. High fluid circulation rates require that the internal diameter through the pipe and tool joints create only minimum friction loss with respect to the descending fluid.

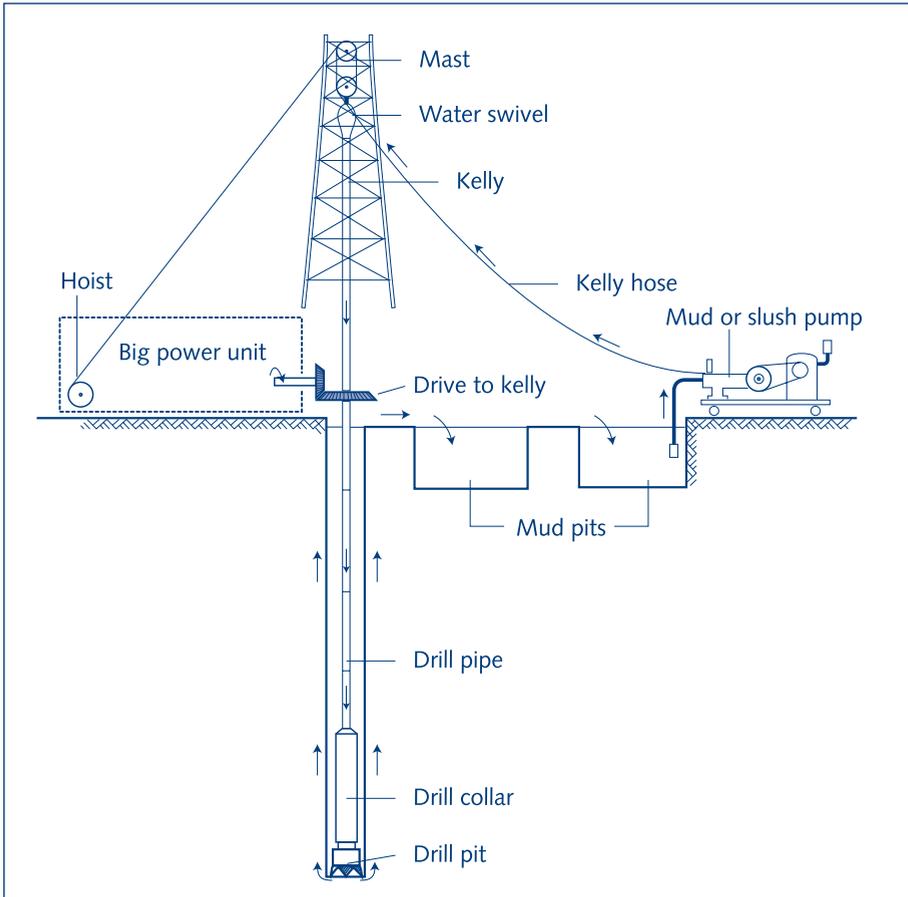


Fig. 10.39. Normal direct circulation rotary drill

Reverse fluid circulation rotary drilling

This method differs from the more common direct circulation system in that the drilling fluid is circulated in the reverse direction. Basically the equipment is similar in general arrangement but considerably larger, and the water way through the tools, drill pipe, swivel, and the kelly is rarely less than 150 mm in diameter (Fig. 10.40). Reverse circulation operates on the basis of “high volume – low pressure”, with a relatively high uphole velocity carrying cuttings up the drill pipe and a low downhole velocity generated by flow under gravity in the annular space. The technique is especially applicable to larger-diameter boreholes in unconsolidated materials, with fast penetration, minimal erosion of the borehole wall, representative formation samples and good borehole stability. As a result of the relatively large diameter drill pipe requirements the minimum practical drilling diameter is of the order of 350 mm, and sizes in excess of 1.8 m are not unknown.

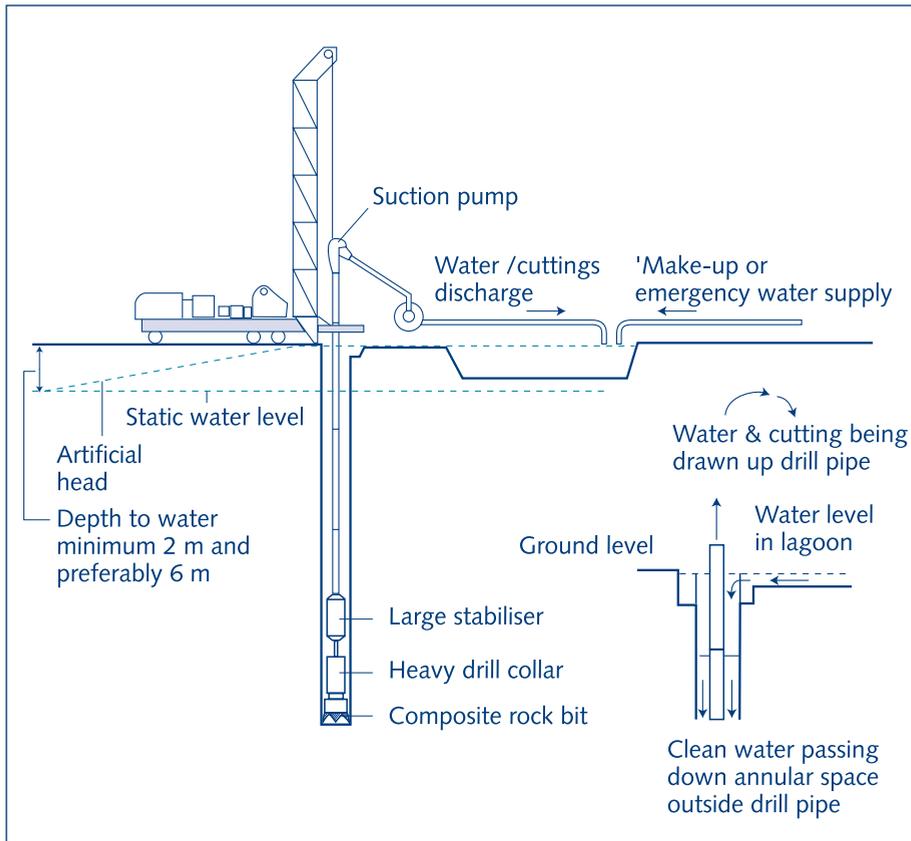


Fig.10.40. Reverse circulation rotary drill

One advantage of the reverse circulation method is that drilling mud is seldom used, with relatively clean water imposed upon the aquifer and therefore no invasion of the formation. Occasionally, low concentrations of polymeric fluid additives are used to reduce friction, swelling of water-sensitive clays, and water loss. In the event of significant clays being drilled in the upper section of the hole, the lagoon should be cleaned out and refilled with fresh water before proceeding into the aquifer, to prevent aquifer invasion.

The main advantage of the reverse circulation method is the very rapid rate of drilling at large diameters, especially in unconsolidated sands and gravel – boreholes are sometimes drilled and lined within 24 hours – as no hole cleaning is necessary. This minimises chances of borehole collapse and possible loss of tools, especially where the head surcharge in the hole to stabilise the borehole walls is small (1-2 m). Under ideal conditions penetration rates of 0.6 m/min have been recorded and average rates of 12 m/hour are quite common. If flanged-and-bolted drill pipe connections are used, they require time-consuming handling and may slow drilling progress.

Return water will infiltrate permeable formations, particularly coarser unconsolidated materials, and will result in water losses from the system. In view of this, one of the prerequisites of reverse circulation drilling is the ready and close availability of a substantial supply of water for make-up purposes. This is often quoted as 45 m³/h and in practice can amount to between 9 and 70 m³/h. Fine particles in the return flow that filter out on the walls of the borehole will help minimise these losses, but a considerable quantity of “make-up” water must be available at all times. If water losses are sudden and cannot be immediately made up, the water level in the borehole will drop and caving of the hole walls usually results.

As noted above, the normal reverse circulation system utilises suction as the motive power behind the flow circuit. However, there are circumstances such as pipe friction at greater depths, or a low water table, when suction is insufficient. For this reason most drilling rigs have provisions for introducing an airlift into the system. Providing that drilling has reached a sufficient depth for proper operation of an airlift, this will then induce water within the drill pipe, and the mud pump can be bypassed.

Air circulation rotary drilling

Air circulation has advantages such as longer bit life, faster penetration, and rapid delivery of cuttings to the surface. Air drilling can, however, only be successful in semi-consolidated or consolidated formations in which no assistance is required from the circulating fluid to support the borehole walls.

The drilling equipment is basically similar to that used for conventional fluid flush drilling; one difference is the design of the drilling bit that has air passages to cool the bearings of the bits (Fig. 10.41).

One of the limiting factors when drilling with air, especially as the borehole diameter increases, is the need to produce an adequate uphole air velocity to ensure cuttings removal. To overcome some limitations, several variations on the reverse circulation system have been developed such as dual-wall drill pipes or pipes with built-in air channels. The dual wall method can be used either with DTH hammer and button bits, or with rotary tricone bits.

Dual wall drilling has an advantage of the formation and groundwater samples originating over a very short vertical section before being flushed to the surface without any risk of further contamination. In addition, problems of lost circulation are largely eliminated. The principal disadvantages of this method are its depth and diameter limitations (diameter generally less than 254 mm; depth generally less than 200 m in unconsolidated materials) and the high capital cost of the equipment.

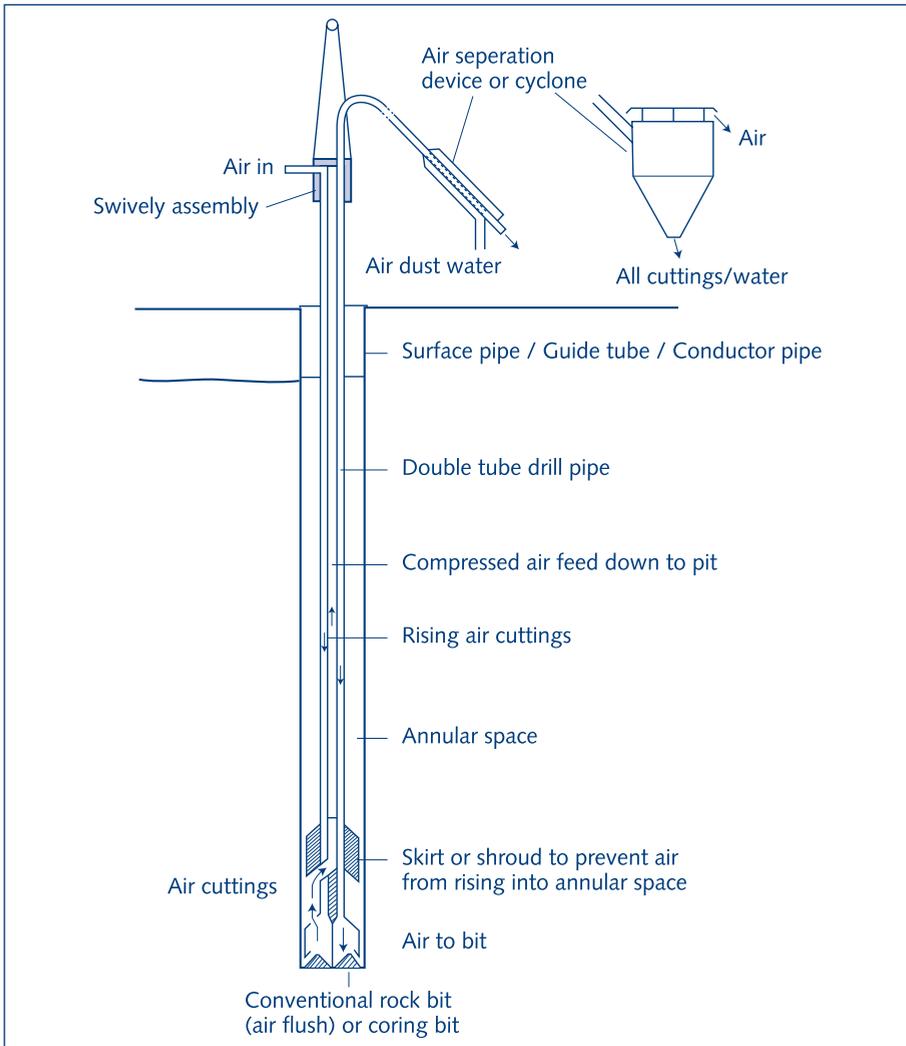


Fig.10.41. Reverse circulation air drilling

Other drilling techniques

Hydraulic tube racking may be used in conjunction with a rig or crane for drilling relatively shallow wells of large diameters in loose gravel, sand, boulders, or similar ground formations. A short guide tube is hand-sunk into the ground and the first of a column of permanent tubes lowered within it. The bottom edge of the column is serrated and the tubes drilled, perforated, or slotted as required. A hydraulically clamped spider with long horizontal rams oscillates the tube column downwards slowly but steadily. When the first tube is close to the spider table, the next tube is placed upon it and the joint welded. Tubes of 450 mm - 1.2 m can be worked down to 30 m or so under the right conditions, the advantage of this system being that there is no need for temporary tubes or large lagoons. There also is no contamination of the aquifer by drilling fluids.

Scow (or California Stovepipe) drilling is applied in cable-tool drilling, and combines the cutting edge of a chisel with the material handling ability of a bailer. The *scow* consists of a heavy, thick-walled bailer with a cutting shoe and a flap valve, suspended by a *scow sub* by means of a pair of reins and a heavy pin from the drilling jar and swivel socket assembly. In operation, it is run into the hole and the normal reciprocating action is applied inside the casing shoe. Water for drilling is added if it is not present naturally. Material is dislodged by the cutting shoe and swept into the body of the scow. The scow is then removed for emptying periodically. Scow drilling can be successfully used in loose, troublesome strata, especially where coarse gravel and boulders occur, and has the advantage of dislodging and lifting the material rather than expending time and power on crushing.

Shell and auger drilling is a percussion technique that uses a string of either solid or hollow rods that are lifted and dropped to provide the cutting action to a chisel bit with a check valve incorporated in the tool. The auger tool is used to cut and remove clay, and the *shell* is similar to a bailer and removes soft strata already loosened by the chisel bit. If hollow rods are used, drilling fluid to assist in removing cuttings can be incorporated in a reverse circulation circuit, with flow down the borehole annulus and return flow through the centre of the rods. This drilling method requires only a lightweight rig and is often used for site-investigation work, since hole diameters are limited to 50-100 mm to very shallow depths (< 20 m).

10.7 Construction materials

During the design of a borehole the selection of the correct materials required for its construction is of extreme importance. In defining the technical specifications of the materials considerations must be given to the expected dimensions of the borehole (depth, diameter) and its ultimate usage, as well as the cost and availability of the materials. Ideally all materials for a borehole, or a complete drilling programme, should be with the contractor prior to commencing the work. The principal borehole construction materials, including those used in drilling as well as those incorporated in the completed structure, are briefly discussed below.

Drilling fluids

Virtually all rotary drilling systems require drilling fluids. The addition of some natural clay to increase viscosity in order to aid the lifting of rock cuttings and gravel subsequently led to the use of the term "mud" for this circulating medium. Nowadays the term "drilling fluid" is more commonly used, and refers variously to clean water, dry air, a suspension of solids or a mixture of liquid additives in water or water, surfactants and colloids dispersed in air.

The primary functions of a drilling fluid are:

- To remove cuttings from the face of the borehole and transport them to the surface
- To lubricate and cool the bit and tool string
- To stabilise the borehole whilst drilling proceeds
- To facilitate suspension of cuttings in the borehole when the fluid is not circulating (e.g. whilst adding additional drill pipe)
- To allow settlement of fine cuttings to the bottom of the settling pits
- To control fluid loss in permeable formations by building up a wall-cake to consolidate the formation and to reduce the loss of fluid into the formation
- To help control sub-surface hydrostatic pressures
- To help provide some buoyancy to long strings of tools or casing in deep boreholes

The most widely used drilling fluid additives are:

- **Clay or bentonite**

Natural clay or *bentonite* is widely used in water well drilling. Different types of clay may be used, but the only common commercially available clay is montmorillonite or *bentonite*.

- **Organic polymers**

To overcome the possible aquifer degradation inherent with clay-based drilling fluid additives, organic polymers are now widely used as a clay substitute.

- **Air**

The ideal drilling strata in which to apply this flushing medium are hard, stable igneous or metamorphic formations when penetration rates can be high. Fissured zones can often be penetrated without the fear of lost circulation associated with water-based fluids.

- **Foam**

Drilling foam is created when a small volume of water mixed with a surfactant is injected into the air stream. Foaming agents prevent cuttings from aggregating and also reduce the surface tension of water droplets so that both can be lifted more easily to the surface, thus helping to overcome large groundwater inflows. Foam also reduces the uphole velocity requirements, thereby reducing the volume and pressure of air required. Loss of air into the formation and hole wall erosion are thus minimised.

Casing and Screens

Casing used in water boreholes is tubular material that provides support to the walls of the borehole and may be either temporary (used by the driller during drilling of the borehole and subsequently withdrawn) or permanent (forming part of the final construction of the completed borehole). Steel casing is most commonly used, but thermoplastic casing is now increasingly applied in areas of potentially corrosive groundwater and where boreholes are generally less than 300 m deep. Other materials

such as glass fibre are also used but are much less common. Selection of casing material should be based upon water quality, borehole depth and diameter, drilling methods, local regulations and cost.

A borehole screen is a filtering device that allows groundwater to enter the borehole. It provides structural support but prevents sediment from entering, especially in unconsolidated strata. Proper screen design and selection is important for the hydraulic efficiency of the borehole, as well as for the longevity and long-term cost of the structure. Screens should have maximum open area to create minimum resistance to flow into the hole, but at the same time must minimise sediment ingress and provide adequate structural strength. Optimising each of these criteria for a particular borehole is the essence of screen design.

Gravel Packs

Gravel pack (or filter pack) is graded granular material placed in a thin zone around the screens in order to increase the effective hydraulic diameter of the borehole and to help inhibit the movement of sediments into the borehole. Clearly, the borehole diameter relative to the final cased completion diameter must be large enough to allow installation of such material, which in turn implies that larger drilling diameter must be employed in holes that are scheduled to be gravel packed. Gravel pack material should be chosen to retain most of the unconsolidated formation material, and screen openings are then selected so as to retain about 90% of the gravel pack. Gravel packs are 100-200 mm thick.

Sanitary seals

In order to prevent ingress to the aquifer of any potential contaminants from the surface via the borehole itself, it is necessary to seal the uppermost section of the annulus between the outermost casing string and the wall of the borehole. The sealing material commonly used is cement grout or a bentonite mixture. It is also prudent to ensure that no potential contaminants can enter the borehole from the surface via the space between any other internal casing strings.

A sanitary seal should preferably be installed in continuity with a cement or concrete well head protection slab to minimise cracking and joints. It is common practice for this slab to be at least 1 m square, with at least 500 mm of its thickness below natural ground level and 300 mm above (Fig. 10.42). The permanent casing protruding through this slab should extend at least 300 mm above the slab, which should slope in all directions away from the casing.

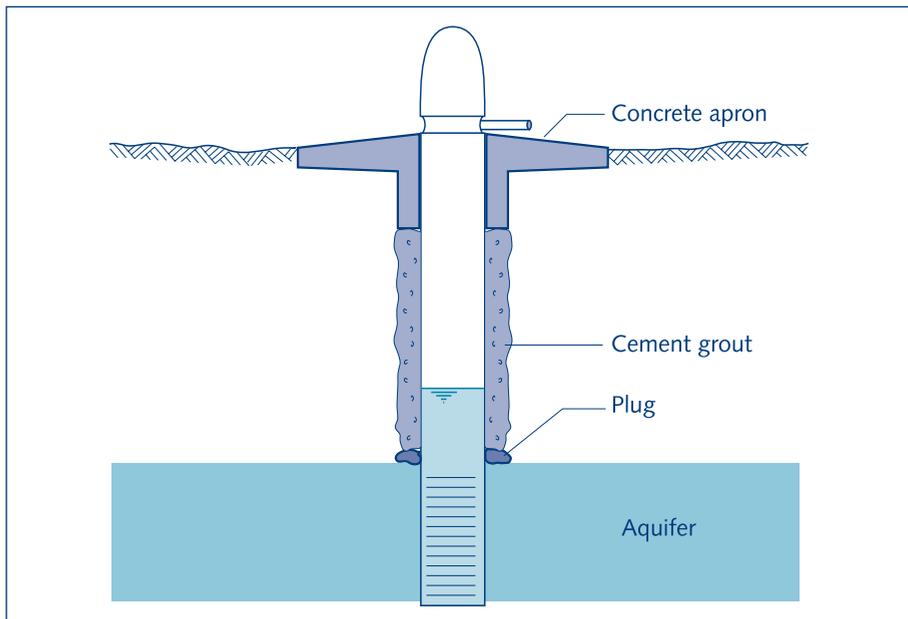


Fig. 10.42 Sanitary seal of borehole

10.8 Borehole development

All drilling methods alter the hydraulic characteristics of formation materials in the vicinity of the borehole. Development procedures are designed to restore or improve these characteristics so as to maximise the performance of the borehole by improving specific capacity and hydraulic efficiency. Other forms of development, often termed *aquifer stimulation*, may be applied after completion of the borehole in order to improve the transmitting properties of the formation in semi-consolidated and consolidated strata. The more commonly used development techniques are outlined below.

Development after completion of drilling

Development of a borehole after completion of drilling forms part of the normal drilling procedure. A number of factors influence the type and potential success of such development, including the well completion configuration (screen slot size, open area, gravel pack thickness and whether a graded or a natural pack is present), the type of drilling fluid used (air or water-based fluids using clays or polymers) and the nature of the formation itself. Development methods include:

Overpumping. This method entails pumping the borehole at a rate greater than that at which it will be pumped in production. This is the least effective method, as it generally only develops the more permeable sections of the aquifer and, since water only flows inwards toward the borehole, it can draw excessive material against the screen openings, creating a condition termed *bridging* in which the formation is only partially

stabilised. Formation material may subsequently enter the hole if the formation is agitated and the *bridges* collapse.

Surging. This method flushes water backwards and forwards through the screen, so preventing any bridging behind the screen and moving fines through into the hole. The surge effect can be generated by intermittent pumping and repeatedly allowing the water column to fall back into the hole, or preferably by mechanical means using a close-fitting plunger (*surge block*) moved up and down on the hole by the drilling rig.

Air surging and pumping. This technique uses an airlift pumping action in combination with the surging effect described above. Air is injected into the hole to lift the water column, and then shut off such that the column falls back into the hole.

Jetting. This development method uses the injection of high pressure air or water through the screens to remove fines and drilling fluids. It uses a special jetting tool that directs horizontal jets onto the screens to break up any filter cake and agitate and flush the adjacent gravel pack or formation. This method is most appropriate with rotary rigs.

Jetting and simultaneous pumping. This method combines high pressure water jetting with pumping (usually using an airlift system) and is particularly applicable in unconsolidated sands and gravel. Essentially the jetting process loosens the fine material and the pumping action draws it through the screen and directly to the surface.

In all the development methods noted above the addition of a small quantity of a polyphosphate compound before or during development will assist in breaking up and removing any clay material that may clog the screens or the gravel pack.

Hydro-fracturing

In bedrock aquifers groundwater is contained in fractures and borehole yields are often very low. In such cases the yield may be enhanced by the use of an aquifer stimulation method such as hydro-fracturing. This is a second level of development (of the aquifer) in which high pressure pumps are used to inject fluid to overcome the overlying rock pressure and open up existing and new fractures that will enable water movement into the borehole. Considerable pumping pressure is required.

Supplementary development methods

Other development or stimulation methods include the use of acid injection in carbonate aquifers to enlarge fissures by dissolution of aquifer material, and “shooting” in hard-rock terrain using explosive charges placed in the borehole to increase the number of fractures around the hole. Such methods are very specialised techniques, and are not commonly used in the majority of water borehole drilling work.

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Web sites

HTN Network for Cost-Effective Water and Sanitation Technologies:

<http://www.skat.ch/htn/>

Water and Sanitation Program: <http://www.wsp.org/english/index.html>

WatSanWeb: http://www.skat.ch/ws/link/watsan/WW_index.htm

Practica Foundation on low cost drilling techniques: <http://www.practicafoundation.nl>

Discussion groups

Handpump Technology Network: <http://www.jiscmail.ac.uk/lists/htn.html>

Water Forum: <http://groups.yahoo.com/group/waterforum/>

11 Surface water intake and small dams

Revised by Nhamo Masanganise

11 Surface water intake and small dams

11.1 River water intake

Groundwater may be the preferred source (chapter 10), but the most convenient source of water for small communities is frequently a natural stream or river close by. The two most important criteria in judging the suitability of the surface water source are the quality of the water and the reliability of the flow.

In tropical countries, rivers and streams often have a wide seasonal fluctuation in flow. This also affects the quality of the water. In wet periods the water may be low in dissolved solids concentration but often of a high turbidity. In dry periods river flows are low and the load of dissolved solids is more concentrated. Mountain streams sometimes carry a high silt load but the mineral content is mostly low and human pollution is generally absent. In plains and estuaries, rivers usually flow slowly except when there is a flood. The water may be relatively clear but it is almost always polluted, and extensive treatment is necessary to render it fit for drinking and domestic purposes. The quality of river water does not usually differ much across the width and depth of the riverbed.

Whenever practicable a river intake should be sited

- where there is adequate flow;
- at a level that allows gravity supply to minimise pumping costs;
- upstream of densely populated and farming areas to reduce silt inflow;
- upstream of cattle watering places, washing places and sewer outlets (to eliminate pollution of the water);
- upstream of bridges (to reduce velocity/turbulence).

Intake designs aim to avoid clogging and scouring and to ensure the stability of the structure even under flood conditions. Where the river transports no boulders or rolling stones, an unprotected intake may be adequate (fig. 11.1).

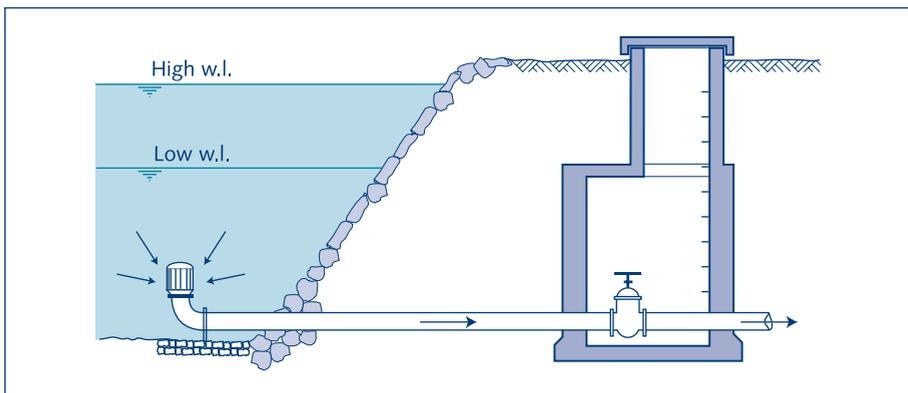


Fig. 11.1. Unprotected river intake

Where protection of the intake is deemed necessary, intake structures of the type shown in figure 11.2 may be suitable. They can be constructed of concrete, stone (masonry) or brick.

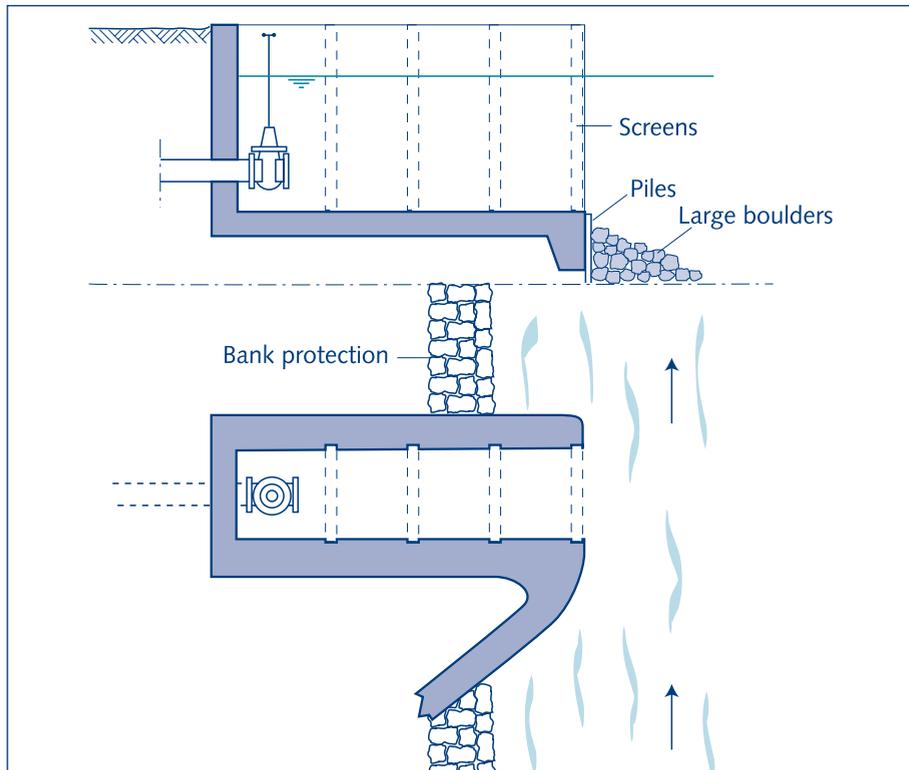


Fig. 11.2. River intake structure

The bottom of the intake structure should be at least 1 m above the riverbed to prevent any boulders or rolling stones from entering. The intake structure must always include one or more baffles or screens to keep out debris and floating matter such as tree trunks and branches. It is advisable to use "passive" screening that does not create turbulent flow conditions. To reduce the drawing in of silt and suspended matter, the velocity of flow through the intake should be low, preferably less than 0.1 m/s. To make use of the natural current to help cleaning of screens, the following tips are suggested:

- The screen axis must be parallel to the current flow
- Dead-end approach channels should be avoided as they collect debris
- Adequate water cover must be provided all around the screen

More information about different types of screens is given in section 11.6.

A river intake always requires a sufficient depth of water in the river. A submerged weir may have to be constructed downstream of the intake to ensure that the necessary

depth of water will be available even in dry periods. This type of weir is only a small structure and cannot be expected to provide any storage or flow balancing. Ideally the weir should be founded on rock, to provide the best conditions with regard to bearing capacity, seepage and safety against sliding. Soils with a clay/silt content that reduces permeability and increases cohesion, but with low plasticity are also acceptable as foundation materials. A key at the upstream toe of the weir will improve stability and an upstream apron will extend the seepage path and thus reduce seepage. A weir stability calculation chart is given in table 11.1.

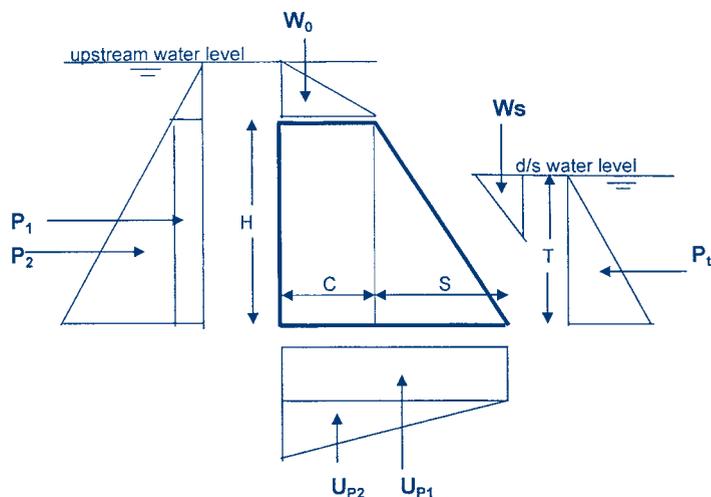
Table 11.1 Weir Stability Calculation Chart

Item	Description	Force	Lever arm	Moments		H →	V ↓
				Overtuning	Restoring		
P_1	$\rho_w \times D \times H$		$H/2$			+	Nil
P_2	$\rho_w \times H^2 \times 0.5$		$H/3$			+	Nil
P_t	$\rho_w \times T^2 \times 0.5$		$T/3$			-	Nil
W_0	$\rho_w \times D \times C \times 0.5$		$S+2C/3$			Nil	+
W_1	$\rho_w \times C \times H$		$B-C/2$			Nil	+
W_2	$\rho_w \times H \times S \times 0.5$		$2S/3$			Nil	+
W_s	$\rho_w \times T \times DS \times 0.5$					Nil	+
U_{p1}	$\rho_w \times T \times B \times \text{uplift factor}$		$B/2$			Nil	-
U_{p2}	$\rho_w \times (H+D-T) \times B \times 0.5 \times \text{uplift factor}$		$B/3$			Nil	-

TOTALS Σ

DIM	
C =	
D =	
H =	
T =	
S =	
B =	

NB: $B = C + S$



Density of water, $\rho_w =$

Density of construction material $\rho_m =$

Uplift Factor =

Downstream Slope =

$M =$ Restoring moment – Overturning moment

$\tan \Theta = \Sigma H / \Sigma V$ (must be less than 0,75)

$\Theta =$ $\cos \Theta =$

$\Sigma H^2 =$ $\Sigma V^2 =$

$R = \sqrt{\Sigma H^2 + \Sigma V^2} =$

$x = \frac{\bar{x}}{\cos \Theta} =$ =

$\bar{x} = \frac{M}{R} =$ =

Mid third = from to

Resultant passes through mid third = Stable

Resultant does not pass through mid third = Unstable

Safety against sliding, Q is given by:

$$Q = (C_0 \times B + V_+ - V_-) \times f / (H_+ - H_-)$$

Where:

$Q > 4$ for safety

f = friction factor or foundation material

$C_0 =$ cohesion of foundation material

Frequently, pumping is needed to draw water from surface water sources. If the variation between the high and low water level in the river (or lake) means that the pumping head will not be more than 3.5-4 m, a suction pump placed on the bank may be used (Fig. 11.3).

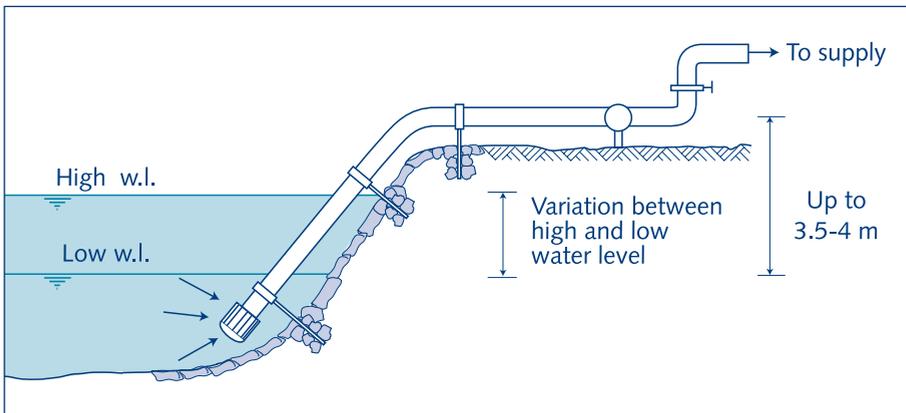


Fig. 11.3. Pumped river (or lake) water intake

A different intake arrangement is needed if the required pumping head exceeds 3.5-4 m. One arrangement worth considering uses a sump constructed in the bank of the river (or lake). The river (or lake) water is collected with infiltration drains laid under the riverbed and flows under gravity into the sump. As the lowest water level in the sump will probably be too deep for a suction pump placed above ground, the water is usually drawn with a submersible pump, or a spindle-driven pump, positioned down in the sump.

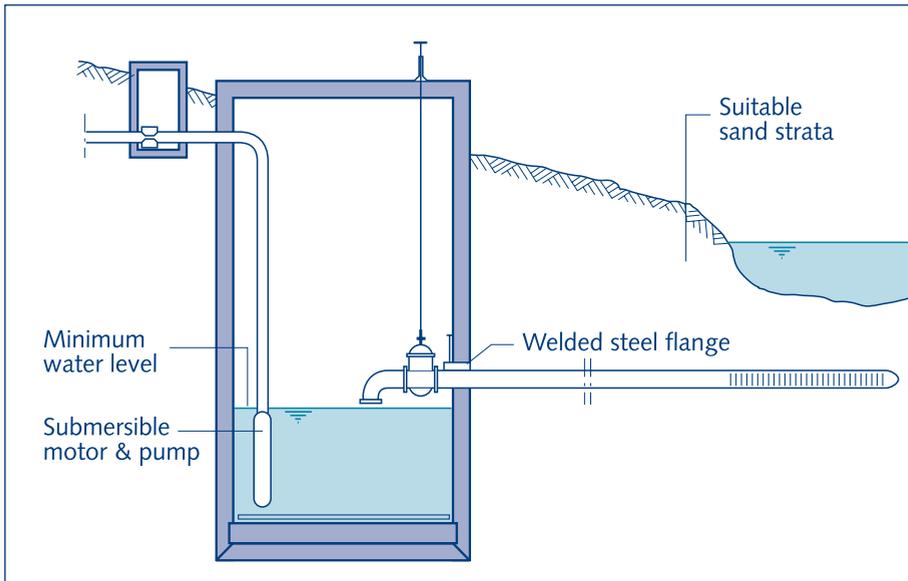


Fig. 11.4. Bank river intake using infiltration drains

11.2 Lake water intake

The quality of lake water is influenced by self-purification through aeration, bio-chemical processes, and settling of suspended solids. The water can be very clear, of low organic content and with high oxygen saturation. Usually, human and animal pollution only present a health hazard near the lake shores. At some distance from the shore, the lake water has generally a low density of pathogenic bacteria and viruses. However, algae may be present, particularly in the upper layers of lakes.

In deep lakes, wave action and turbulence caused by the wind striking the surface will not affect the deeper strata. As there is no mixing, a thermal stratification will develop, with the warmer upper water layers floating on top of the cooler deeper ones. As a result the deeper water layers may differ in quality from the upper water. The thermal stratification can be fairly stable, especially under tropical conditions. Figure 11.5 gives an example. In colder climates the winter temperature of the upper water can drop below that of the lower layers and inversion of water layers will occur.

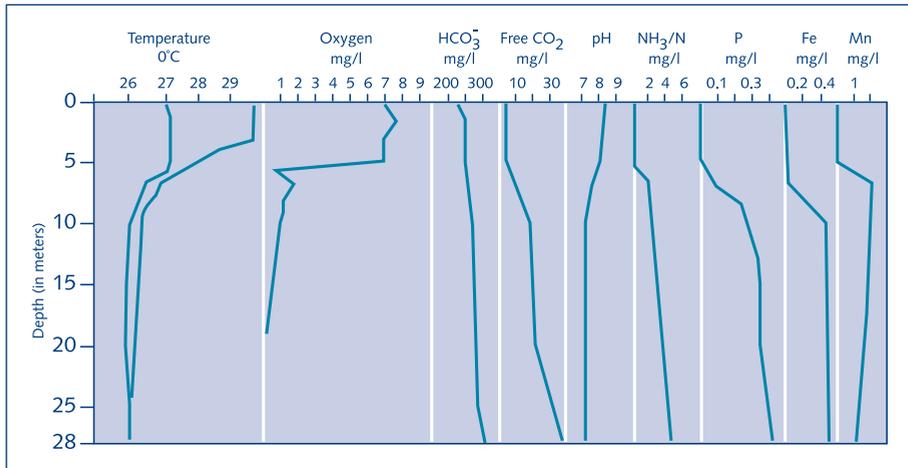


Fig. 11.5. Water quality variation with depth in a deep lake (Indonesia)

Thermal stratification should be taken into account when choosing the location and depth of a lake water intake for water supply purposes. The presence of algae in the upper water layers is another relevant factor.

In deep lakes with water of a low nutrient content (nitrates, phosphates, etc.), the chemical quality of the water will be much the same throughout the full depth. For water supply purposes, water from deeper strata will have the advantage of a practically constant temperature. Provision should be made to withdraw the water at some depth below the surface (fig 11.6). In some instances a multi-level intake structure can be constructed to take advantage of the aerated water on the surface (Fig. 11.7).

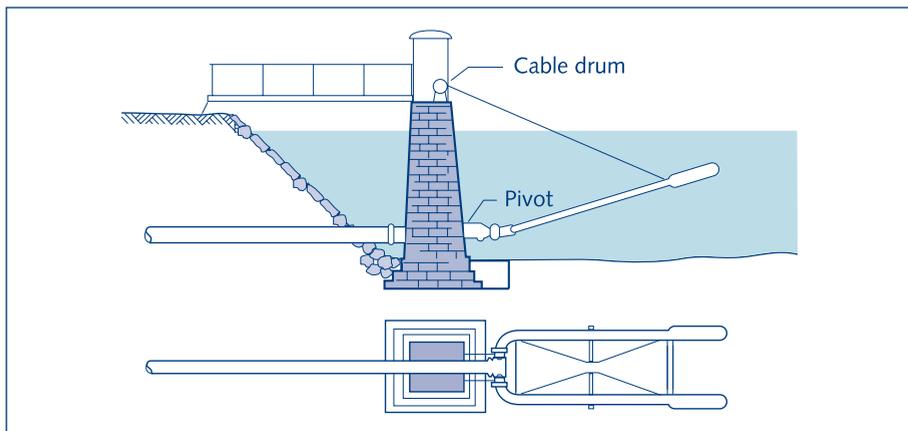


Fig. 11.6. Variable depth lake water intake

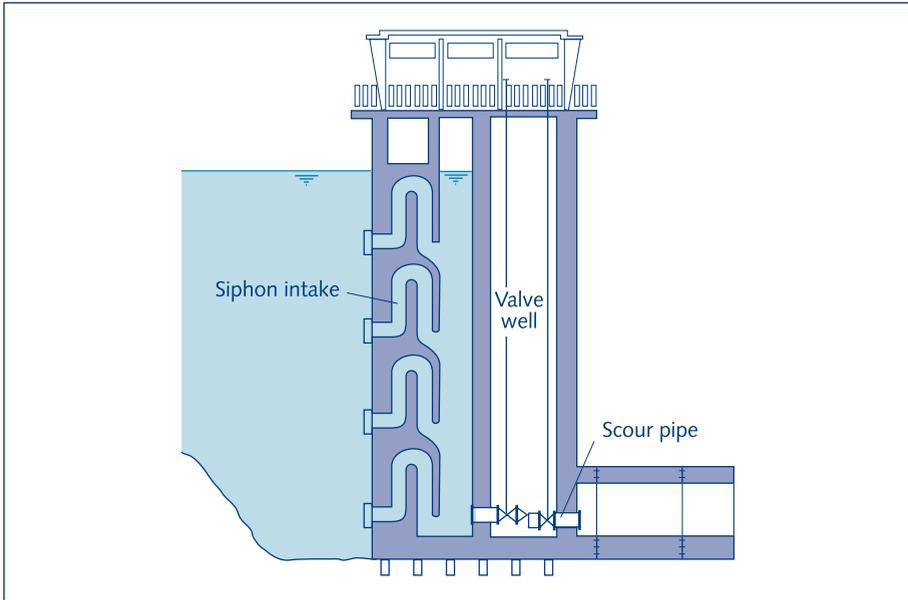


Fig.11.7. Multi-level intake (Adapted from Twort et al, 1994)

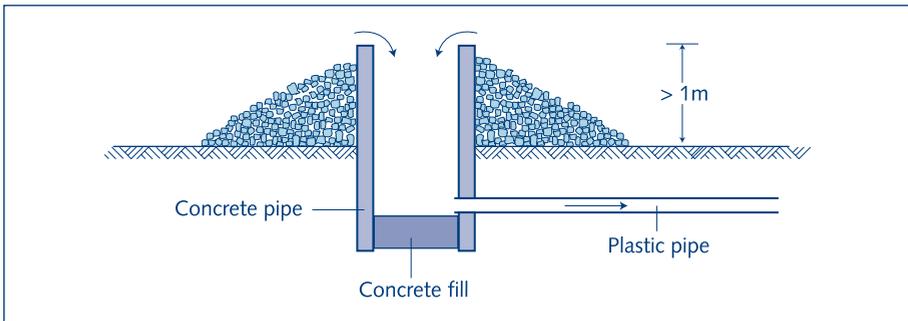


Fig.11.8. Intake structure at bottom of shallow lake

11.3 Typical intake constructions

For small community water supplies only small quantities of water are needed and often very simple intake structures can be used. With a per capita water use of 30 litres/day and the peak intake 4 times the average water demand, 1000 people would require an intake capacity of only 1.4 l/s. A 150 mm diameter intake pipe would be sufficient to keep the entrance velocity 0.1 m/s. If an entrance velocity of 0.5 m/s were allowed, a pipe as small as 60 mm would be adequate.

For small capacity intakes, simple arrangements using flexible plastic pipe can be used (fig. 11.9). Another intake construction using a floating barrel to support the intake pipe is shown in figure 11.10. The water is pumped from the well sump.

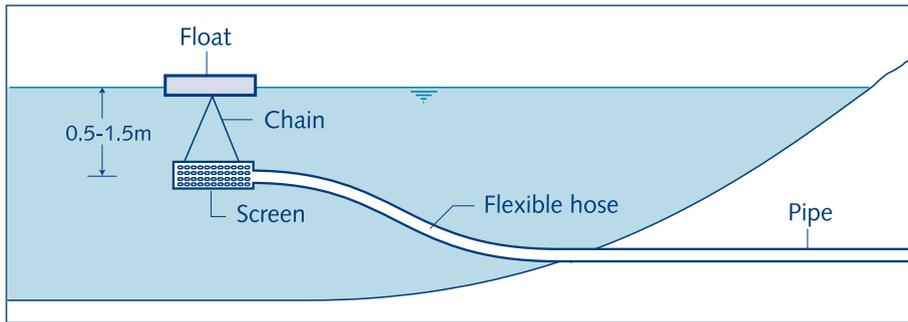


Fig.11.9. Simple water intake structure

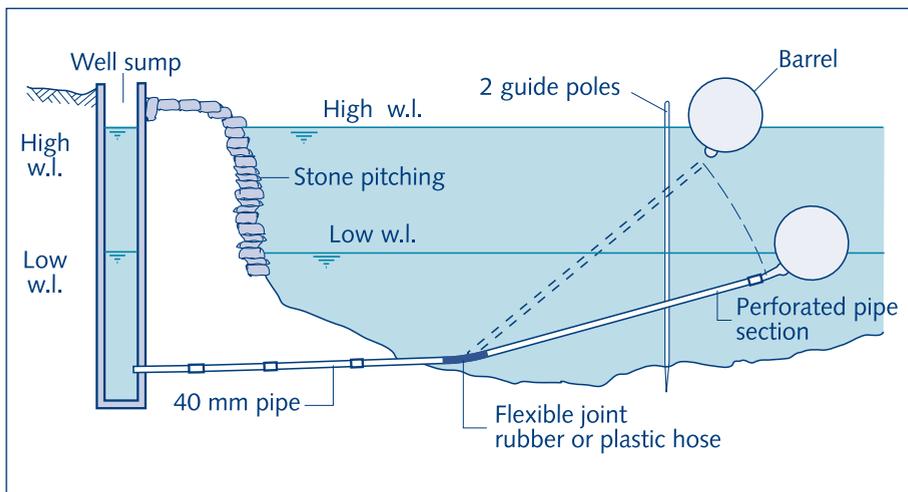


Fig. 11.10. Float intake

11.4 Small dams and village ponds

There are many small communities whose only source of water supply is a small dam or pond. These are called "hafir" (Sudan), "tapkis" (Nigeria), "represa" (South America), "tank" (India), "pond" (Eastern Africa), or other local names. In this handbook they are all referred to as small dams or village ponds.

Small dams and village ponds vary in area from small to large ones of several hectares. They may be entirely natural in origin - depressions in flat areas - or specially constructed for the purpose. If man-made, they may have been excavated for the express purpose of holding water or they may be pits from which clay for building has been taken. Many are a combination of natural and man-made pools, which have been deepened or enlarged over the years to increase their holding capacity. Some are to be found inside villages or small towns. More often, they are located just outside the village area. In India they are often stocked with fish, which provides a valuable source of food for the villagers.

Unfortunately they are also used for washing and bathing. More often than not they are polluted, hazardous at best, and potential spreaders of epidemics. Too often they harbour high densities of pathogenic bacteria, viruses and parasites that account for incalculable numbers of illnesses and deaths, particularly amongst children.

The water may be full of silt or colloidal matter, especially immediately after the rains. Some village ponds have been in existence for centuries; they may be full of aquatic vegetation. Some are the recipients of refuse of all sorts. Others are relatively well kept, beautiful in appearance and a distinctive feature of the landscape.

In practice, it is impossible to prevent the pollution of these small dams and village ponds. By their very nature they are at the lowest point of the surrounding area and the entire village drainage finds its way into them.

In a newly excavated pond, construction of an intake presents no difficulty if carried out before the tank fills. In an established pond, work must be done while the water is in use. Probably all the work will be done by hand and no special tools will be available. In ponds where the water has high turbidity the water is best drawn from just below the surface. A floating intake device may be suitable. Plastic pipe could be used instead of galvanised iron for the collecting pipe. The floating support may be made of bamboo or other locally available materials.

The presence of algae and other aquatic vegetation, as well as fish, in the pond will make it necessary to fix a strainer around the intake. The level of the intake opening must be below the lowest draw-off point if siphoning is to be avoided. A well may be dug close to the bank and the intake pipe either thrust (using a heavy jack) or driven from the well into the pond. The pipe is then capped while the well is lined with masonry or concrete, and the floating intake fixed. The well should be deepened to form a sump, which will allow some settlement of suspended matter.

For obvious reasons, the water from these ponds is not suitable for drinking. Unless reliable and effective water treatment (central or home-based) is applied, villagers must be discouraged to drink it; a health campaign should highlight the serious health risks from this water.

11.5 Operational and protectional issues

River and lake intakes are usually located at a distance from the community. Therefore the caretaker needs a good schedule and routine to visit the intake as they will not be permanently there. During high river discharge daily visits are advisable to remove floating material and debris from the screens and weir. Checks for any damage of intake, bank protection and weir from heavy material (e.g. trees) or from heavy flow

from debris need to be made. During low river discharge the visits can be less frequent. Then the checks are more related to the surface water level and – in case of the variable intake point – to adjust the intake point.

Siltation of weirs is common for most rivers. This is due to the settlement of sediment present in the river water. Several times per year during high river discharges, the weir sluice gates are opened to flush out the accumulated sediment behind the weir. Monitoring of these operational issues by the water agency is needed.

The construction of surface water intakes are sturdy and easy damage is not expected. Because usually there will not be a permanent supervision of the site, special protection of the site is recommended. When pumps are involved special arrangements must be made to prevent theft of engines/motors and pumps. The area around the intake needs to be fenced to keep animals out and also to discourage people of entering; this will reduce the risk of accidents.

11.6 Screens

In water supply engineering, screens are used for various purposes:

- Removal of floating and suspended matter of large size which otherwise might clog pipelines, damage pumps and other mechanical equipment, or interfere with the satisfactory operation of the treatment processes. Fixed screens are used for this purpose and they are cleaned on site by hand or mechanically.
- Clarification of the water by removal of suspended matter even of small size, to lighten the load on the subsequent treatment processes. In particular screens are used to prevent filters from becoming clogged too rapidly.

Screening is done by passing the water through closely spaced bars, gratings or perforated plates. It does not change the chemical or bacteriological quality of the water. It serves to retain coarse material and suspended matter larger than the screen openings. Even when screened-out material forms a filtering mat of deposits, the screening still is purely of a mechanical nature.

Bar screens usually consist of steel strips or bars spaced at 0.5-5 cm. If the amount of material expected to be screened out is small, the bars are set quite steeply, at an angle of 60-75° to the horizontal, and cleaning is done by hand using rakes. If larger amounts will be retained, cleaning by hand should still be feasible; to facilitate the cleaning work, the bars should be placed at an angle of 30-45° to the horizontal (Fig. 11. 11).

The water should flow towards the bar screen at a quite low velocity, 0.1-0.2 m/s. Once the water has passed the screen, the flow velocity should be at least 0.3-0.5 m/s in order to prevent the settling out of suspended matter.

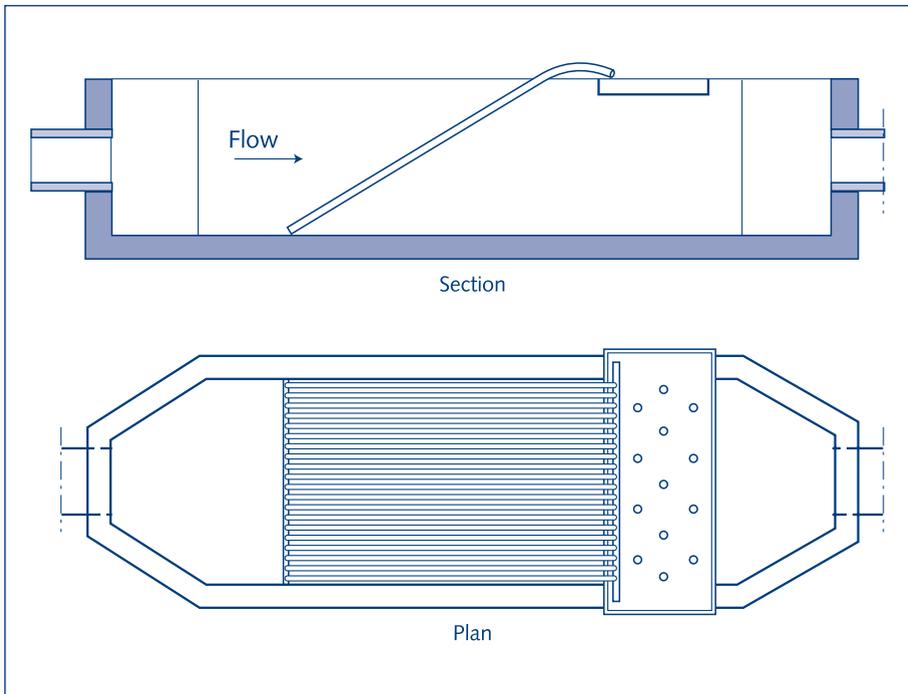


Fig. 11.11. Fixed bar screen

In the openings between the bars the velocity of flow should be limited to a maximum of 0.7 m/s; otherwise soft, deformable matter will be forced through the screen openings. A clean screen will allow the water to pass with a head loss of only a few centimetres. However, the head loss rises sharply when the clogging of the screen builds up. Regular cleaning should keep the head loss limited to 0.1-0.2 m head of water. Allowing for delayed cleaning and mechanical failures, it is good practice to design a bar screen for a head loss of 0.5-1.0 m.

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12 Water treatment

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12 Water treatment

12.1 Introduction

There are many situations where treatment of raw water is necessary to render it fit for drinking and domestic use. The problems of affordability and technical complexity associated with small to medium water treatment facilities are relevant for all high, medium and low income country economies. In more densely populated areas in the North, these problems have been reduced by encouraging large cities to extend water services to smaller communities, or, in areas with no large cities, by stimulating small communities to form regional water companies. In 1996, the 31 drinking water companies in the Netherlands supplied water to 15.5 million people, an average of 500,000 consumers per company. In contrast, the network for supplying drinking water in the United States of America (USA) is fragmented. In 1996 there were 54,728 community water systems (CWSs) in the USA serving 248 million people, i.e. an average of 4,500 consumers per company.

Box 12.1 Fragmentation of drinking water supply sector in the USA

They were distributed in 46,827 small systems (supplying water to a range of 25-3300 people) serving a total of 25 million people; 4332 medium (in the range of 3301-10,000 people) serving 25 million people in total; and 3569 large (range >10,000 people) serving 198 million people. While the small systems serve around 10% of the population covered by the USA CWSs, they account for an inordinate percentage of the violations under the USA Safe Drinking Water Act, SDWA.

Source: Stout and Bik, 1998

Small communities in the USA face great difficulty in continuously supplying water of adequate quality and quantity. This is because at the salary levels that the national economy requires they often lack the economies of scale needed to hire experienced operators and managers.

In Canada and the USA in the 1990s, special attention was given to the development and promotion of treatment technologies for small to medium-sized communities. The 1996 Safe Drinking Water Act (SDWA) Amendments in the USA contain provisions related to water supply systems serving less than 10,000 people, recognising their differences in costs, technology, management capacity, and risk characteristics. The Environmental Protection Agency (EPA) developed a list of water treatment technologies that smaller companies can apply to comply with the US regulations. In this list, the EPA suggests that rapid filtration should be used only in systems with full-time access to a skilled operator. It considers that

slow sand filtration may be the most suitable filtration technology for small systems, when used with source water of appropriate quality (EPA, 1998).

In spite of changes originating from decentralisation of the water supply and sanitation (WSS) sector, the water supply companies' field is also fragmented in most developing countries. In some countries solutions have been found in the formation of strong regional water companies. They make it feasible to introduce and sustain more complex technologies and systems. Smaller towns and rural communities are not able to run complicated water systems that surmount local capacity and feasible regional support structures. When planning and designing a small water treatment plant, the construction and running costs, and the operational and maintenance needs are key factors that must be considered carefully.

12.2 Multiple barriers strategy and basic water treatment concepts

Applying different barriers is important to reduce the sanitary risks from drinking water due to microbial, physical and chemical contaminants. Barriers include: watershed and land use management to protect surface and ground water; selection and protection of the best available water sources; on-site or off-site wastewater treatment and reuse; water treatment; adequate and well-maintained distribution systems; and safe water practices by consumers (Geldreich and Craun, 1996). Water source protection programmes are considered particularly relevant to small systems, where community participation is likely to be more effective.

Because water contamination is associated with so many variables, a good drinking water quality depends on more than water quality enhancement or stream self-purification capacity or the water treatment processes. Water treatment is viewed as just one of the barriers needed to ensure that water produced from a given source complies with the national drinking water quality standards or the World Health Organization (WHO) guidelines. This is particularly important in regions where the majority of the systems rely on surface water sources.

The level of water treatment technology should be in harmony with the types of risk existing in the supply source and the institutional and socio-economic conditions prevailing in the target community. This includes the availability of skilled O&M and management staff. Since infectious diseases caused by pathogens are the most common health risks associated with drinking water, priority in water treatment should be given to reducing this type of contaminants, but without ignoring the risks associated with chemical contaminants in the source water.

Some basic concepts can be identified to make water treatment more reliable and efficient:

- **Multi-stage water treatment.** This concept has a long history and has evolved gradually with the increased attention to water quality. Successive stages progressively remove contaminants from the raw water and consistently produce safe and wholesome final water. Ideally, the safe state should be achieved before the final treatment stage, so that failure of any one process does not result in significant risk of waterborne disease transmission. Consequently the system should be robust and close to fail-safe.
- **Integrated water treatment.** In applying the multistage water treatment concept it is important to understand that each unit process may not be equally effective in removing different types of pollutants. Integrated water treatment is therefore also an important concept. It requires that the strengths and weaknesses of each treatment stage be quantified and balanced, as well as the combination of the different treatment stages, so that all contaminants are effectively removed at a feasible cost. In general it is convenient to separate the heaviest and larger material first and gradually proceed by separating or inactivating the smaller material represented by particles that include colloidal solids and microbes.
- **Terminal water disinfection.** The last stage of treatment providing protection from waterborne pathogens is usually called terminal or safety disinfection. However, this stage will only be effective if the previous stages have efficiently removed most of the waterborne pathogens and reduced solids or other contaminants that may either interfere with the mechanisms involved in the disinfection process or contribute to unacceptable levels of disinfection by-products in the distribution system.

Application of a multiple barriers strategy should contribute significantly to reducing the cost and complexity of water treatment. The appropriate selection and combination of the different treatment stages should allow the use of only a small and fairly constant dose of disinfectant as final safety barrier. In this way, application of multistage and integrated water treatment concepts should contribute substantially to avoiding both customer complaints and excessive disinfection by-products. In turn, it will make the final treatment stage easier to operate and more reliable.

The benefit of applying multistage treatment is illustrated by the fact that the incidence of waterborne diseases in the USA is eight times higher in communities using surface water sources without filtration, than in those using filtration (Craun et al., 1994). Reinforcing the argument, the use of multistage treatment plants to produce safe and bio-stable effluents, together with good materials and engineering practices in the distribution systems, are allowing the Dutch water industry to maintain the quality of treated surface water without disinfectant residuals.

12.3 Health risks associated with drinking water

The main health risk related to water supply systems that use surface water from unprotected catchment areas stems from the discharge of untreated wastewater from human settlements and industries. Contamination of a water source with human and animal excreta introduces a great variety of bacteria, viruses and protozoa. The health risks associated with microbiological contamination are so important that their control is the highest priority. Poor water quality may be particularly harmful to children, old people or members of the community with compromised immune systems. For these groups, infectious doses are significantly lower than for the rest of the population.

There are a few chemical compounds that pose an acute health risk to the users such as high levels of nitrate leading to methaemoglobinaemia (*blue baby syndrome*) in infants. Other high-risk chemicals include heavy metals, fluoride, arsenic and organic compounds that enter the environment via industrial discharges, mining activities or spraying of pesticides and herbicides. But chemicals like fluoride and arsenic may also be present in geological formations and be dissolved in the groundwater as a result of chemical and physical processes. Chemical pollution may pose a chronic health risk associated with long periods of exposure. Its control is therefore important, but is a secondary concern in water supply systems that are subject to severe bacteriological contamination.

Understanding that chlorine reacting with organic matter can cause oxidation by-products (OBPs) that may represent a chronic health risk, raised concerns about its application in controlling the transmission of cholera in Latin America. However, it has been established that the health risk associated with these by-products is very small, compared with the risk related to inadequate disinfection. In reality, the chronic risks must not be ignored, but the acute risks from microbiological contamination are clearly much more important, especially in the case of systems drawing water from polluted sources. This is the situation in many countries with a poor sanitary situation and a low level of socio-economic development. In the search for possible alternatives to chlorine (such as ozone and ultraviolet radiation), it is necessary to know if they produce OBPs, as well as assessing whether they are equally effective, economically competitive, and easy to dose and supply. Specific techniques have been developed for home-based disinfection including solar disinfection.

Consumers accept or reject drinking water mainly using aesthetic considerations. Issues such as turbidity, colour, taste (for instance caused by high salt content) and odour can make them turn to other water sources that may be more contaminated and involve a higher health risk. So, aesthetic aspects need to be taken into account in the development of water supply systems. Aesthetic standards vary with local culture. National standards can therefore not be applied as a matter of routine. They need to be

cross-checked with local women in particular, to ensure acceptability of any quality norms that will be applied.

With increasing life expectancy, enhanced institutional capacities and improved economic conditions, water treatment has progressively combined technologies to reduce initially the acute health risks, often of microbiological nature, and later the chronic health risks, usually of physical, chemical origin. This is illustrated in figure 12.1, based on the work of Coffey and Reid (1982).

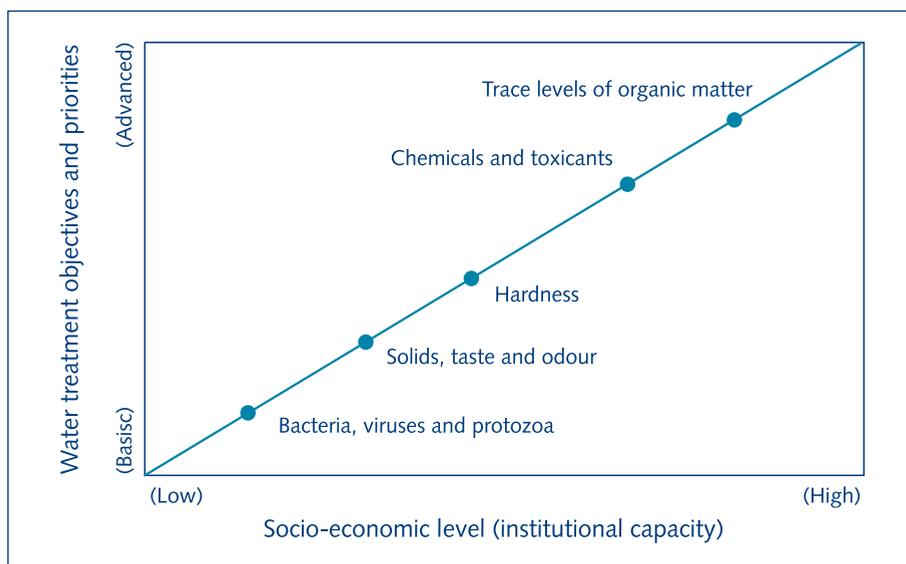


Fig. 12.1. Conceptual relation between socio-economic level and water treatment objectives

In summary, the most important criterion in the treatment of domestic water is the removal of all pathogenic organisms as well as high risk chemical substances such as heavy metals, fluoride, arsenic, nitrate and organic constituents. Other substances may also need to be removed or at least considerably reduced. These include suspended matter causing turbidity, iron and manganese compounds imparting a bitter taste or staining laundry, and excessive carbon dioxide corroding concrete and metal parts. For small community water supplies, other water quality characteristics such as hardness, total dissolved solids and organic content would generally be less important. The aim is a reduction to levels that are acceptable to the consumers so that use and sustainability are not affected negatively, although the extent to which the water is treated will be limited by economic and technical considerations. The quality guidelines for drinking water presented in chapter 4 should be a guide when the extent of the necessary treatment is determined.

12.4 Historical background of water treatment plants

The Romans made great strides in building and organising water supply systems. The water commissioner of Rome around 100 AD was the head of a well developed organisation run by the government. According to books describing the system, it included a settling reservoir at the head of one of the aqueducts supplying the city, and the piscanae, pebble catcher facilities built into most of the roman aqueducts (Baker, 1981; Coffey and Reid, 1982).

During the Middle Ages in Europe, after the decline of the Roman civilisation, WSS principles were widely ignored. Monasteries came to be leaders in providing water services. When the urban populations resumed their growth, municipalities took over leadership. By 1810 there were several private companies serving London. Due to the growth of towns and the increasing popularity of water closets in the early 1800s, sewage and other wastes were dumped into drains that discharged to the rivers being used as drinking water sources. Early water mains supplied water only intermittently and in some poor areas one well pump and a privy would serve many houses.

In 1831 the first recorded cholera epidemic reached Britain. At that time no one knew the cause or origin of diseases such as cholera, typhoid, and dysentery. Frequent complaints and difficulties arising from the poor quality of the water supplied in London led to successive political decisions and reforms. In 1842 Edwin Chadwick produced an important Report on the Sanitary Conditions of the Labouring Population of Great Britain. Although Chadwick did not make any specific connection between water and disease, clean water was considered part of the environmental improvements desirable for health. In 1848, an Act of Parliament charged government for the first time with the responsibility for safeguarding public health. According to this Act, water supplies had to be "pure, safe, and constant". It was another fifty years before this sanitary ambition was practically fulfilled. A metropolitan water act of 1852 required that domestic water from the Thames River had to be filtered. London's population was close to 2.5 million then. However, the water companies were often not fulfilling these requirements.

It was in 1849, during the second epidemic of cholera in Britain, that John Snow's first essay on the waterborne nature of cholera appeared. It was based on evidence collected and analysed on fatal cases around a well on Broad Street in London. Still, it was not until the third epidemic in 1854, that John Snow's views on the dissemination of cholera were vindicated. After Snow's work and the events of 1854 and 1856, the connection between water supply and waterborne disease was firmly established.

In the second half of the 19th century it was generally accepted that solutions to the prevailing public health problems depended on improvements in the sanitary

infrastructure, requiring large engineering projects. Towards the end of the century scientific developments in the medical world started to become influential. With the development of bacteriology, after the discoveries of Pasteur and Koch in the 1880s, the germ theory became important in the fight against contagious diseases. This stimulated interest in other issues such as water source protection, water supply, basic sanitation, hygiene education and water treatment.

Pioneering work in water treatment by filtration

Plain sedimentation improves the clarity of surface water, but filtration gives much better results. In Paris, sand filters in copper containers were used for two centuries. The first water filter patent was granted in France around 1750. These filters were to be constructed of lead, pewter or earthenware. The filtering material of sand (or sponge) was packed between two plates, the lower one to serve as a false bottom to the filter, the upper to prevent disturbance of the sand when the water was poured into the vessel.

Towards the end of the 18th century, the first British patent was granted on a process and apparatus for water filtration, with ascending flow to clarify the water and reverse flow to clean the filter medium. To accomplish this, the patent proposed either three tanks or one tank with three compartments. The first received the turbid water from a service pipe, the second contained a stratified medium for filtration, and the third received the clarified water. The coarse filter material was placed at the bottom of the filter with regularly decreasing sizes above it, so that interstitial spaces would increase in geometric ratio.

Crude versions of slow sand filters (SSF) were used for industrial water supplies in Britain before the end of the 18th century. A Scottish industrialist started to sell water from his industrial plant to city dwellers. The muddy and industrially polluted river water flowed to a well through a coarse filter 23 m long, 2.4 m wide and 1.2 m deep. A steam engine placed over the well lifted the water to an *air chest* about 5 m above the river, from which it flowed to the plant. The plant had sedimentation and double filtration stages with lateral flow as shown in figure 12.2. Due to its radial layout, maintenance problems are likely to have been a serious limitation for long-term operation.

The early filters were never completely successful because an adequate cleaning procedure was not available to the operators. Two filter cleaning methods were gradually developed by testing and learning. The first was the self-cleaning filter, washed by reverse flow. The other was achieved by scraping off the thin dirty top layer, removing, washing, and restoring it at intervals. Both were based on the fact that surface clogging caused the filtration process to fail. The common filter material arrangement was from coarse at the bottom to fine material at the top of the filter.

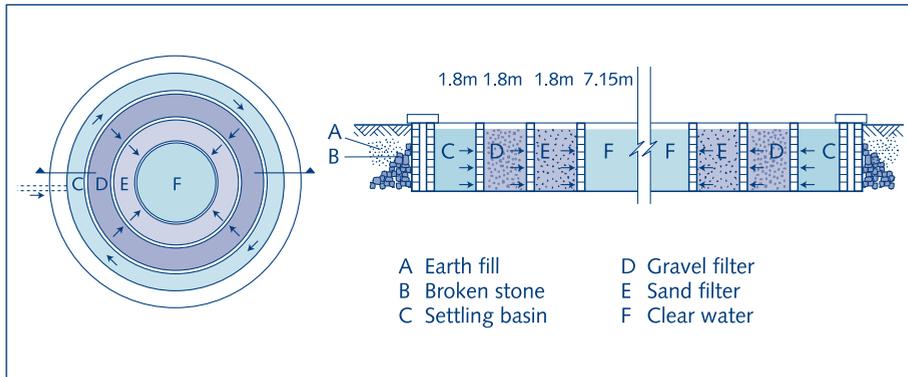


Fig. 12.2. Water treatment plant at Paisley, Scotland, 1804 (Adapted from Baker, 1981)

The experimental slow sand filter plant in London had two settling reservoirs working in parallel, followed by a filter (fig. 12.3). The filter was 13.4 m^2 at the top and 1.8 m deep. The plant filtered 3.9 l/s^{-1} , at a rate of 0.15 m/h^{-1} . During tests the filter was being scraped about once a fortnight. This scraping procedure was seen as the best way of overcoming cleaning limitations.

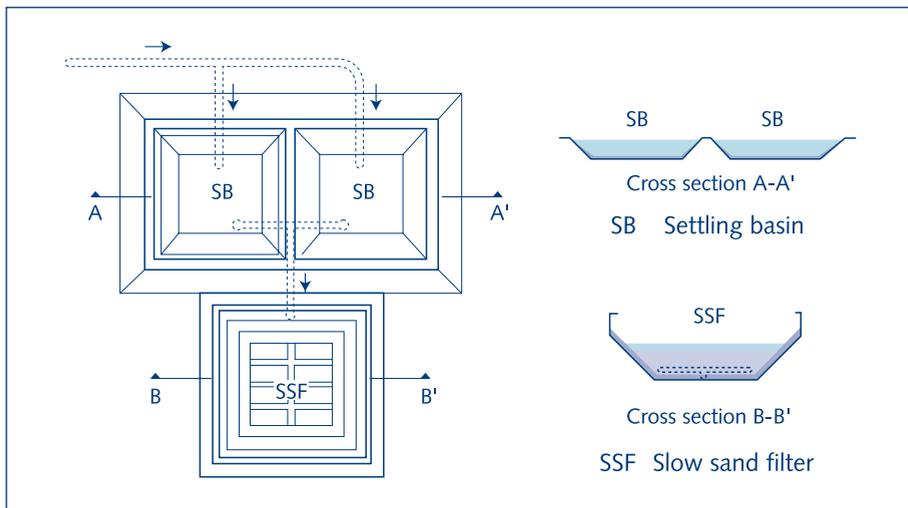


Fig. 12.3. Simpson's London experimental slow sand filter, 1827-1828 (Adapted from Baker, 1981).

Based upon this experimental SSF, a first English SSF with a size of one acre was put into operation at Chelsea in 1829. This became the classical model of SSF. The other filter design with the elements of reverse-flow wash and a false bottom were to become principal features of the rapid filter developed in the USA during the 1880s.

In France, Darcy patented a filter that included all of the elements of the American rapid filter except one – coagulation. Darcy made an innovative combination of previously developed filtration principles with sound hydraulic considerations about flow through

porous media, now known as Darcy's Law. This law states that the flow per unit of area of filtering bed is proportional to the hydraulic gradient in the porous media.

Over the years, disinfection processes associated with water treatment have included heat, copper, silver, chlorine, ozone, ultraviolet radiation, and membranes. The most popular has been chlorine. Chlorine, as hypochlorite solution, started to be used at treatment plant level in Belgium in 1902, the United States in 1904, Britain in 1911, and London in 1916. Chlorine as a liquid (compressed in metal cylinders) was applied first in the USA in 1913, and in Britain (London) in 1917. Recent regulation concerning the control of disinfection by-products has led to a better sequential application of the multiple stage and integrated water treatment concepts, in which chlorination is logically the final treatment stage. Additionally, there is a growing awareness that some pathogens (e.g. giardia cysts and cryptosporidia) are best removed by filtration at an earlier stage, as they are resistant to chemical disinfection.

Though SSF technology has been successfully applied in northwest Europe since the 19th century, in other regions the use and the impact of this technology has been rather limited. The growing application of rapid filtration (RF) and chemical disinfection in the 20th century contributed significantly to the reduction of waterborne diseases and to the improvement of productivity and quality of life in urban settlements that had the opportunity to install and sustain these technologies. Today, a large number of different types of treatment exist for the purification of water. However, selection of the most suitable option for a given community remains a major challenge, particularly if it is a low-income community with limited institutional, organisational and financial capacity.

There needs to be a systematic partnership approach involving representatives of the different user groups and the availability and use of all relevant information on robust, efficient treatment technologies. The goal is to arrive at a solution that includes selection and protection of the best available water resources combined with a treatment system that is economically sound and easy to operate, manage and maintain. For small communities, the partners need to compare the technical, managerial and institutional feasibility of potential central water treatment systems with other water treatment and water supply options. The other options could include home-based water treatment and family-based small systems such as private, protected wells.

12.5 General effectiveness of water treatment process for contaminant removal

Some water treatment processes serve a single purpose and others have multiple applicability. Often a treatment result can be obtained in different ways. The removal of some water contaminants by various well-designed and properly maintained and

operated treatment processes is illustrated in table 12.1. The comparison is necessarily general, due to the multiple factors affecting water treatment efficiency.

Storage of water can be regarded as treatment. For example, *Schistosoma* cercariae are normally unable to survive 48 hours of storage. The number of faecal coliforms and faecal streptococci will be considerably reduced when the raw water is subjected to storage. 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 is often another drawback.

By exposing water to air (aeration), volatile dissolved components that are in excess of their saturation concentration can be removed from the water while gases from the atmosphere can be transferred to the water. Carbon dioxide (CO₂) and some volatile toxic organic and taste- and odour- causing compounds may be removed to satisfactory levels. The addition of oxygen (O₂) will enhance the oxidation of metals (e.g. iron and manganese) to higher and more insoluble oxidation states.

In chemical coagulation, colloidal particles are destabilised, to be subsequently agglomerated or flocculated to form larger particles that are easier to filter (direct filtration) or settle. In sedimentation, water is exposed to relatively quiescent conditions and suspended solids (SS) may be removed by the action of gravity.

Filtration usually fulfils the role of “polishing” the treated water. SSF and RF are the main filtration alternatives in common use. Only high quality raw water can be directly filtered (without pre-treatment) to remove small quantities of suspended solids. Other raw waters must be conditioned before reaching the main filtration stage. Coarse media (e.g. sand or gravel) filters can be used to reduce concentration of contaminants such as algae, micro-organisms or suspended solids. The combination of coarse media and slow sand filtration units can be used to treat polluted surface water, without dosing it with chemical coagulants. Influent water to RF must have a coagulant agent added to it first.

There are some typical operational differences between SSF and RF units. Filtration rates are around 50-150 times lower for SSF. Flow retention periods are about 30-90 times longer for SSF. Filter run lengths are about 30-90 times longer for SSF, and the surfaces of SSF units are usually scraped at the end of the filter runs, whereas RF units are cleaned by backwashing. These differences originate from the most relevant and distinctive feature of SSF, the biological life at the sandbed surface. In fact, the main treatment mechanisms in RF are of a physical-chemical nature and the water treatment in SSF is the result of a combination of physical-chemical and biological mechanisms that interact in a complex way. For a more detailed discussion of multi-stage filtration including SSF see chapter 16. RF is described in detail in chapter 17.

Table 12.1 General effectiveness of water treatment processes for contaminant removal

Water quality parameters Treatment processes	Aeration and stripping					
	Sedimentation	Coagulation,	Flocculation, sedimentation, and filtration	Slow Sand filtration	Multistage filtration ¹	Chemical oxidation: disinfection
Bacteria	0	0	+++	+++	++++	++++
Viruses	0	0	+++	+++	++++	++++
Giardia cysts	0	0	+++	+++	++++	++
Cryptosporidium oocysts	0	0	+++	+++	+++	+
Turbidity	0	+	++++	+++	++++	0
Suspended solids	0	+++	++++	+++	++++	0
Taste and Odour	++	0	+++	++	+++	+
Iron and manganese	++ ²	+	+++	+++	+++	++
Fluoride	0	0	+	0	0	0
Arsenic	0	0	++	+	+	0
Heavy metals	++ ²	0	++	+	++	+
Dissolved Oxygen	+	0	0	-	-	0
Carbon Dioxide	-	0	0	+	+	0
Colour and organics	0	0	++	++	++	+

0, no effect; +, increasing positive effect; -, negative effect

1 Multistage filtration: Combinations of one or more stages of gravel filtration in series with slow sand filtration

2. Aeration in combination with sedimentation.

Disinfection removes pathogenic micro-organisms or renders them inactive. Water must be in contact with the dose of disinfectant agent for a time long enough to assure the required reductions of indicator micro-organisms, usually bacteria. Disinfection processes are described in chapter 19.

12.6 Groundwater quality and treatment

For the most part, groundwater originates from rainwater that infiltrates through the soil and is stored in aquifers. During infiltration the water can pick up impurities such as inorganic and organic soil particles, debris from plant and animal life, micro-organisms, natural or man-made fertilisers, pesticides, etc. During its flow underground, however, a great improvement in water quality will usually occur. Suspended particles are removed by filtration, organic substances are degraded by oxidation, and micro-organisms die away because of lack of nutrients. The dissolved mineral compounds, though, are not removed. In fact, the mineral content of the water can increase considerably through the leaching of salts from the underground layers.

Groundwater, if properly withdrawn, will be free from turbidity and pathogenic organisms. When it originates from a clean sand aquifer, other hazardous objectionable substances will also be absent. In these cases, disinfection as a safety barrier is desirable, but direct use of the water as drinking water may be permitted without any treatment. When the water comes from an aquifer containing organic matter, oxygen will have been consumed and the carbon dioxide content of the water is likely to be high. The water will then be corrosive unless calcium carbonate in one form or another is present. When the amount of organic matter in the aquifer is high, the oxygen content may be completely depleted. Water containing no oxygen (anaerobic water) will dissolve iron, manganese and heavy metals from the underground strata. These substances can be removed, i.e. by aeration and filtration. It depends on the type of aerator whether the carbon dioxide content of the water will be reduced or left unchanged. A reduction is desirable if the water is corrosive but in other cases it can result in troublesome deposits of calcium carbonate.

Sometimes groundwater contains excessive amounts of iron, manganese and ammonia, but also fluoride, arsenic and salts. Groundwater may also sometimes be polluted by industrial and other hazardous waste. These compounds may be dissolved in the groundwater and they can pose a very high health risk even in small concentrations. Where alternative water is scarce, even these polluted groundwater sources may have to be used and the source water treated with chemical coagulation and flocculation, ion exchange and different filtration technologies (including GAC¹) to render them fit for drinking and domestic purposes.

Many of these treatment processes involve expensive technologies. They are also technically very complicated to manage and the O&M is expensive. For small community water supplies in the South many of these processes are too complicated and they should be avoided whenever possible. New small-scale water treatment techniques and methods have been and still are being developed which are appropriate for the specific conditions in poor communities in the South. Table 12.2 summarises the treatment processes described above. If groundwater is under the direct influence of surface water, it should be protected or treated as surface water. For chemicals such as fluoride, arsenic and salts (dissolved solids) more complicated treatment processes are required. They are described in chapters 22 ("Fluoride"), 23 ("Arsenic and Iron") and 18 ("Desalination").

1 GAC = granular active carbon used as a filter medium

Table 12.2 Treatment of groundwater

Water quality parameters	Treatment processes				
	Aeration for		Plain sedimentation	(Rapid) filtration	Safety disinfection (chlorination)
	Increasing O ₂	Reducing CO ₂			
Aerobic, fairly hard and not corrosive					0
Aerobic, soft, and corrosive		X			0
Anaerobic, fairly hard, and not corrosive	X				0
Anaerobic, fairly hard, not corrosive with iron and manganese	X		0	X	0
Anaerobic, soft, corrosive no iron and manganese	X	X			0
Anaerobic, soft corrosive with iron and manganese	X	X	0	X	0

(X, necessary; 0, optional)

12.7 Surface water quality and treatment

Surface water can be taken from streams, rivers, lakes or irrigation canals (see chapter 11). Water in such surface sources originates partly from groundwater outflows and partly from rainwater that has flowed over the ground to the receiving bodies of surface water. The groundwater outflows will bring dissolved solids into the surface water; the surface run-off is the main contributor of turbidity and organic matter, as well as pathogenic organisms. In surface water bodies, the dissolved mineral particles will remain unchanged but the organic impurities are degraded through chemical and microbial processes. Sedimentation in impounded or slow-flowing surface water results

in the removal of suspended solids. Pathogenic organisms will die off due to lack of suitable food. However, new contamination of the surface water is likely to take place as a result of waste influents and algal growth.

The criteria for the degree of treatment recommended by WHO (1993) to produce drinking water from surface sources with a negligible risk of containing viruses are summarised in table 12.3. Although pre-disinfection is recommended in this table, other treatment stages such as storage or coarse filtration should be preferred to reduce the required doses of chemical disinfectants and risks associated with oxidation by-products. WHO (1993) considers that “the attainment of the bacteriological criteria [absence of *E. coli* or thermotolerant coliform bacteria] and the application of treatment for virological reduction [table 12.3] should, except in extraordinary cases of extreme contamination by parasites, ensure that the water has a negligible risk of transmitting parasitic diseases”. However, the guidelines published by WHO (1993) do not include information on performance of the treatment steps to facilitate process selection and combination to fulfil water treatment objectives with different levels of contamination in the water sources.

Table 12.3 Treatment steps recommended by WHO to produce water with negligible virus risk from surface water sources (WHO, 1993)

Type of surface water source	Recommended treatment
• Protected, impounded upland water; essentially free of faecal contamination	• Disinfection ¹
• Protected, impounded water; or upland river; faecal contamination	• Filtration ² and disinfection
• Unprotected lowland rivers; faecal contamination	• Pre-disinfection ³ or storage, filtration, disinfection
• Unprotected watershed; heavy faecal contamination	• Pre-disinfection ³ or storage, filtration, additional treatment ⁴ , and disinfection
• Unprotected watershed; gross faecal contamination	• Not recommended for drinking water supply

1. Before terminal disinfection median turbidity < 1 NTU and < 5 NTU in single samples. Residual of free chlorine > 0.5 mg/l after at least 30 minutes of contact time at pH < 8.0, or an equivalent disinfection process for > 99.99% of enterovirus inactivation.
2. Slow sand filtration (SSF), [Multi-stage filtration (a combination of gravel filtration and SSF)] or rapid filtration (RF) preceded by coagulation-flocculation, or an equivalent filtration process for > 90% enterovirus reduction.
3. Although pre-disinfection is recommended in this table, other treatment stages, such as storage or coarse filtration, should be preferred to reduce the required doses of chemical disinfectants and risks associated with oxidation by-products
4. Additional treatment may consist of SSF, granular activated carbon adsorption with ozonation, or any other process demonstrated to achieve > 99 % enterovirus reduction.

Unpolluted surface water of permanently low turbidity may be purified by slow sand filtration (SSF), or by direct rapid filtration followed by chlorination. SSF has the great advantage of relatively low operational requirements, and local communities can build the filters if required materials and appropriate supervision are available.

When the turbidity of the water to be treated is high, or when algae are present, SSF units would rapidly clog. Pre-treatment is needed, such as sedimentation, coarse (gravel) media filtration, rapid filtration or a combination of two or more of these processes. For colloidal suspended particles, the removal by settling or filtering can be greatly improved through chemical coagulation and flocculation. All these processes are required in most instances where the organic content of the raw water is high. Water from rivers and lakes is of a very wide variety in composition and it is impossible to describe in detail all the treatment systems required in every case. Leaving complicated processes out, table 12.4 shows the systems most applicable to small community water supplies.

Table 12.4 Treatment of surface water

Water quality	Treatment processes			
	Plain sedimentation	Rapid filtration	Multi-stage ¹ filtration	Disinfection (chlorination)
Clear and unpolluted			○	○
Slightly polluted and low turbidity		X	X	X
Slightly polluted and medium turbidity	○	X	X	X
Slightly polluted and high turbidity	X	X	X	X
Slightly polluted and many algae	○	X	X	X
Heavily polluted and little turbidity	○	X	X	X
Heavily polluted and much turbidity	X	X	X	X

(X, necessary, ○, optional)

1. Multi-stage filtration: Combinations of one or more stages of gravel filtration in series with slow sand filtration

Water treatment plant efficiencies are meaningless without reference to the quality of the raw and treated water. For this reason it is important not only to have an assessment of the raw water quality but also performance efficiencies and treatment objectives for the treatment plant. Table 12.5 illustrates this situation based on data from pilot and full-scale multi-stage filtration projects treating water from tropical Andean rivers.

Table 12.5 An example of performance objectives for removal of turbidity and faecal coliform bacteria in multistage filtration water treatment plants (adapted from Lloyd et al., 1991 and Galvis 1999)

Stage and process	Turbidity			Thermotolerant coliform bacteria		
	Removal ¹ (%)	Average loading (NTU)	Maximum ² loading (NTU)	Removal ¹ (%)	Average loading (Per 100 ml)	Maximum loading (Per 100 ml)
Screening	NA ³	NA	NA	NA	NA	NA
Plain Sedimentation	50	60	600	50	2000	10,000
Dynamic gravel filter	30	30	300	40	1000	5000
Coarse (Gravel) filters ⁴	80	21	210	90	600	3000
Slow Sand Filter	>90	<5	42	95	60	300
Disinfection	NA	<1	<5	>99.9	3	15
Distributed Water	NA	<1	<5	NA	<1	<1

- 1 Required performance
- 2 Maximum loading corresponds to turbidity peaks of short duration (< one day)
- 3 NA, not applicable. Process not designed to remove turbidity and/or bacteria
- 4 Three stages of coarse up-flow gravel filtration

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http://www.who.int/water_sanitation_health/GDWQ/GWDWQindex.html

Web sites

IRC Portal: <http://www.irc.nl/>

PAHO/WHO. Virtual Library in Health and Environment:
Water Treatment: <http://www.cepis.ops-oms.org/index.html>

Cost-Effective Technologies in Water Supply and Sanitation: <http://www.skat.ch/htn/>
and <http://waternet.com/>

Roughing Filtration Technology: <http://www.sandec.ch/water/roughfilter.html>

For arsenic and fluoride removal and disinfection, also see the specific chapters.

GARNET discussion groups

Water and Sanitation Applied Research:
<http://www.jiscmail.ac.uk/cgi-bin/wa.exe?SUBED1=water-and-san-applied-research&A=1>

Iron and manganese removal:
<http://info.lut.ac.uk/departments/cv/wedc/garnet/tncironm.html>

Water quality monitoring: <http://info.lut.ac.uk/departments/cv/wedc/garnet/tncwq.html>

13 Aeration

Update by Ken Ives

13 Aeration

13.1 Introduction

Aeration is the treatment process whereby water is brought into intimate contact with air for the purpose of (a) increasing the oxygen content, (b) reducing the carbon dioxide content, and (c) removing hydrogen sulphide, methane and various volatile organic compounds responsible for bad taste and odour. The treatment results mentioned under (a) and (c) are always useful in the production of good drinking water. Reducing the carbon dioxide content, however, may shift the carbonate-bicarbonate equilibrium in the water so that deposits of calcium carbonate are formed which may cause problems.

Aeration is widely used for the treatment of ground-water having too high an iron and manganese content. These substances impart a bitter taste to the water, discolour rice cooked in it and give brownish-black stains to clothes washed and white enamel buckets, bowls, sinks, baths and toilets. The atmospheric oxygen brought into the water through aeration will react with the dissolved ferrous and manganous compounds changing them into insoluble ferric and manganic oxide hydrates. Sedimentation or filtration can then remove these. It is important to note that the oxidation of the iron and manganese compounds in the water is not always readily achieved. Particularly when the water contains organic matter, the formation of iron and manganese precipitates through aeration is likely to be not very effective. Chemical oxidation, a change in alkalinity or special filters may then be required for iron and manganese removal. These treatment methods, however, are expensive and complex, and for rural areas in developing countries it would be better to search for another source of water. For the treatment of surface water, aeration would only be useful when the water has a high content of organic matter. The overall quality of this type of water will generally be poor and to search for another water source would probably be appropriate.

The intimate contact between water and air, as needed for aeration, can be obtained in a number of ways. For drinking water treatment it is mostly achieved by dispersing the water through the air in thin sheets or fine droplets (waterfall aerators), or by mixing the water with dispersed air (bubble aerators). In both ways the oxygen content of the water can be raised to 60-80% of the maximum oxygen content that the water could contain when fully saturated. In waterfall aerators there is an appreciable release of gasses from the water; in bubble aerators this effect is negligible. The reduction of carbon dioxide by waterfall aerators can be considerable, but is not always sufficient when treating very corrosive water. A chemical treatment such as lime dosing or filtration over marble or burned dolomite would be required for this type of water.

13.2 Waterfall aerators

The **multiple tray aerator** shown in figure 13.1 provides a very simple and inexpensive arrangement and it occupies little space.

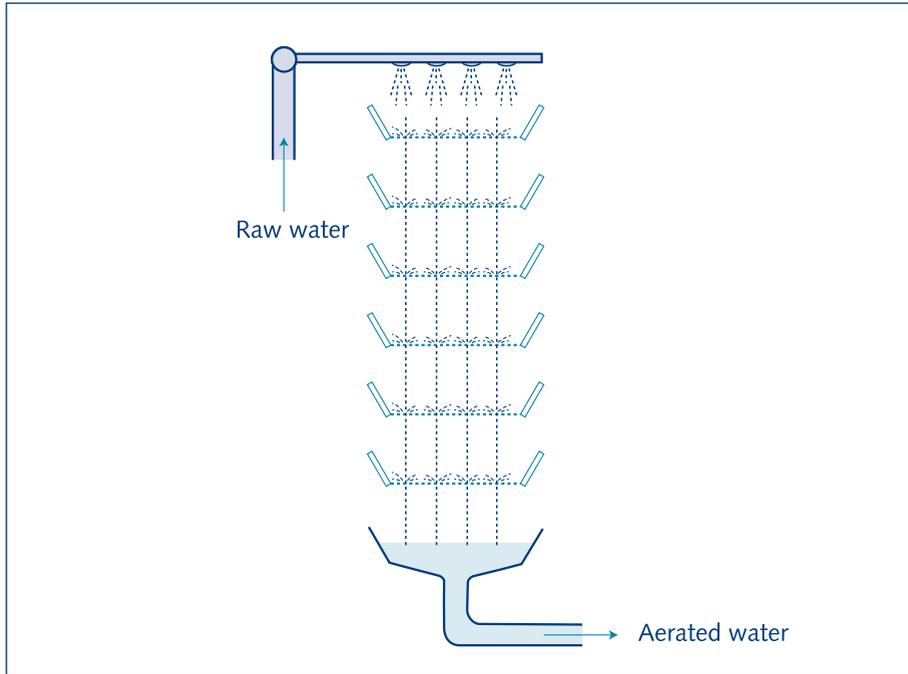


Fig. 13.1. Multiple-tray aerator

This type of aerator consists of 4-8 trays with per-forated bottoms at intervals of 30-50 cm. Through perforated pipes the water is divided evenly over the upper tray, from where it trickles down at a rate of about $0.02 \text{ m}^3/\text{s}$ per m^2 of tray surface. The drop-lets are dispersed and re-collected at each following tray. The trays can be made of any suitable material, such as ferro-cement or plastic plates with holes, small dia-meter plastic pipes or parallel wooden slats. For finer dispersion of the water, the aerator trays can be filled with coarse gravel about 10 cm deep. Sometimes a layer of coke is used that acts as catalyst and promotes the precipitation of iron from the water. A hand-operated aeration/filtration unit for treatment of water having high iron and manganese content is shown in figure 13.2.

A type of aerator with similar features is the cascade aerator (Fig. 13.3). Essentially this aerator consists of a flight of 4-6 steps, each about 30 cm high with a capacity of about $0.01 \text{ m}^3/\text{s}$ per metre of width. To produce turbulence and thus pro-mote the aeration efficiency, obstacles are often set at the edge of each step. Compared with tray aerators, the space requirements of cascade aerators are some-what larger but the overall head loss is lower. Another advantage is that no maintenance is needed.

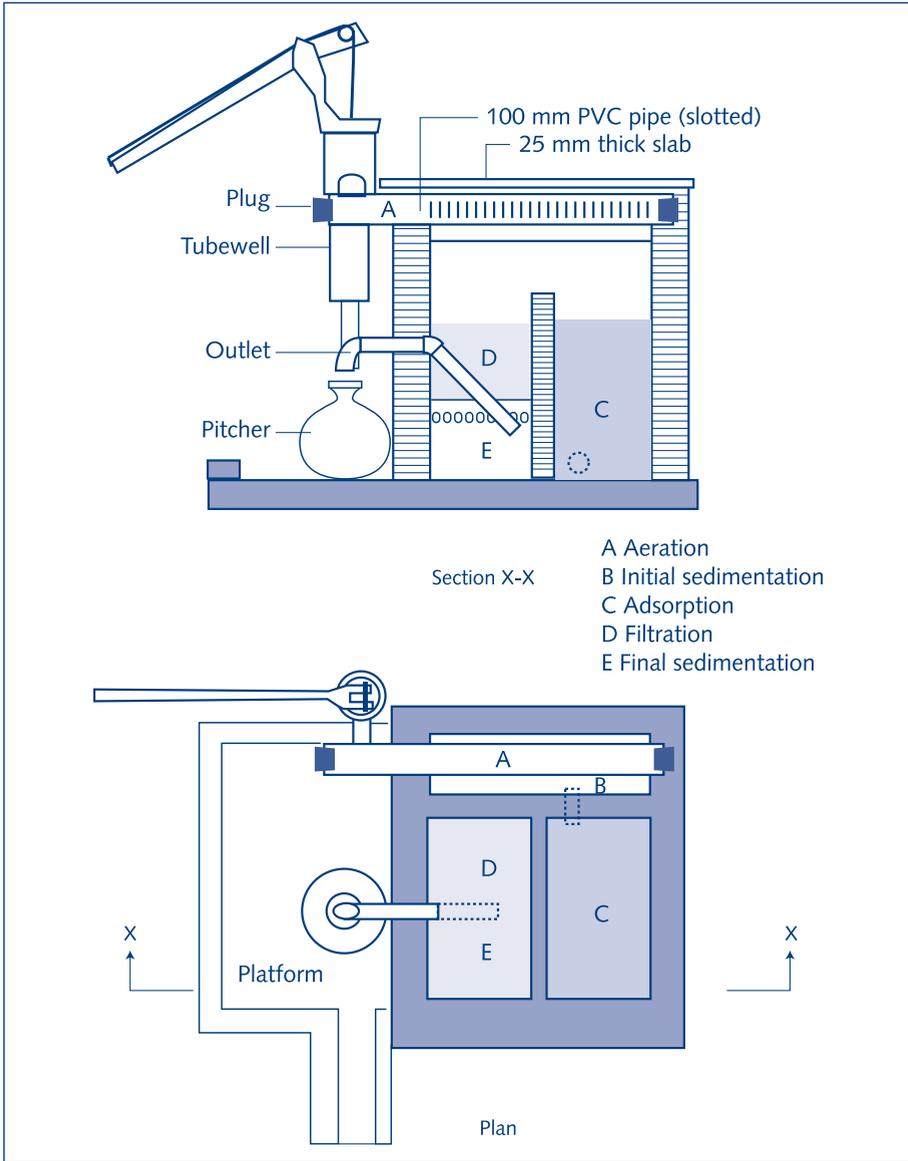


Fig. 13.2. Hand-operated aeration/filtration unit

A multiple platform aerator uses the same principles. Sheets of falling water are formed for full exposure of the water to the air (Fig. 13.4).

Spray aerators consist of stationary nozzles connected to a distribution grid through which the water is sprayed into the surrounding air at velocities of 5-7 m/s.

A very simple spray aerator discharges the water downwards through short pieces of pipe of some 25 cm length and with a dia-meter of 15-30 mm. A circular disk is placed

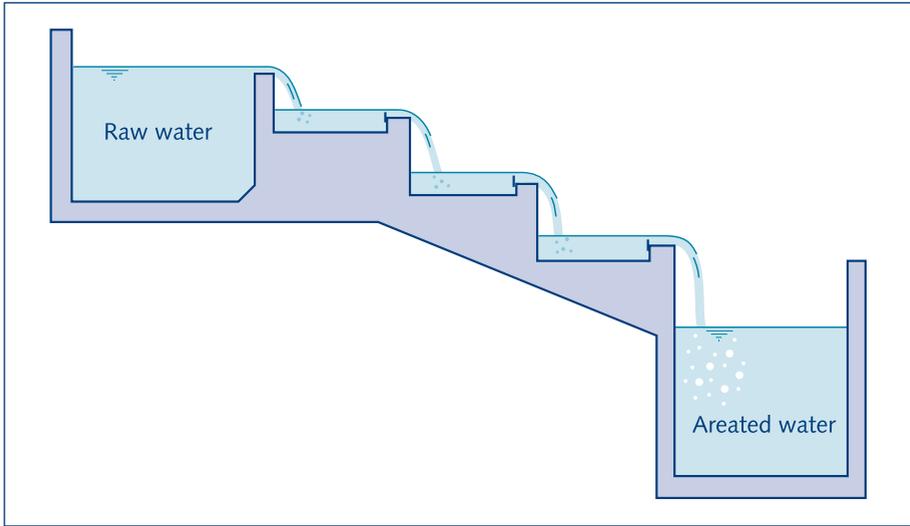


Fig. 13.3. Cascade aerator

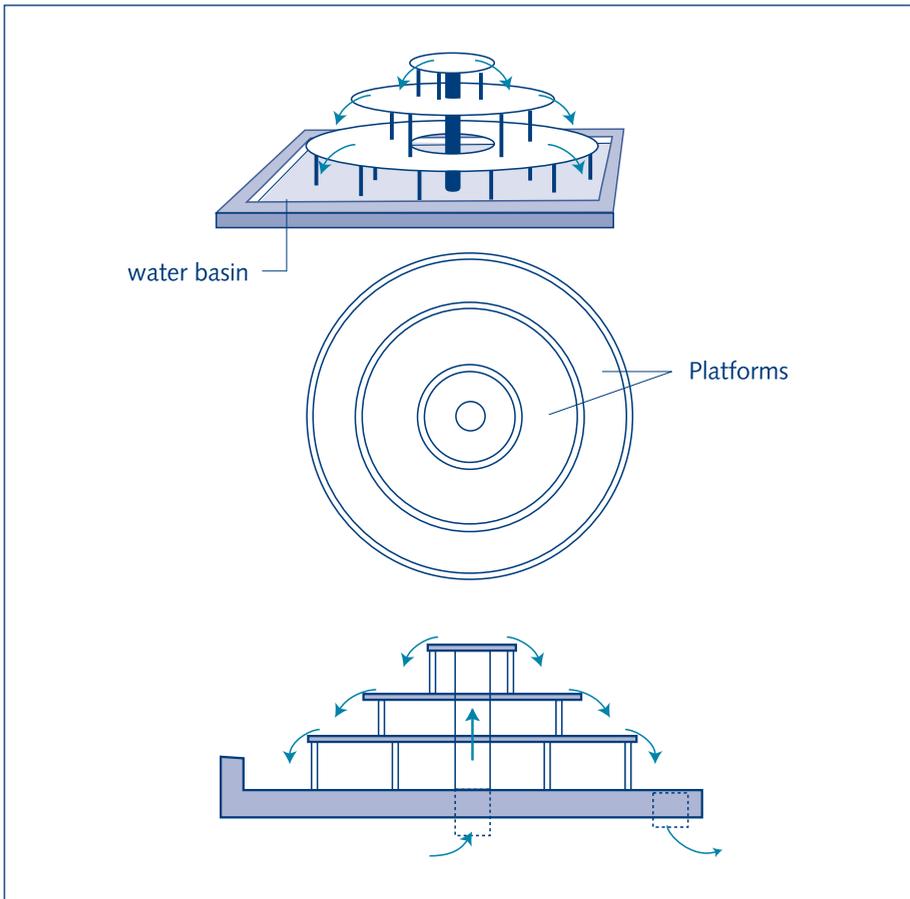


Fig. 13.4. Multiple platform aerator

a few centimetres below the end of each pipe, so that thin circular films of water are formed that further disperse into a fine spray of water droplets.

Another type of spray aerator uses nozzles fitted to feeding pipes, which spray the water upwards (Fig.13.5). Spray aerators are usually located above the settling tank or filter units, so as to save space, and to avoid the need for a separate collector basin for the aerated water.

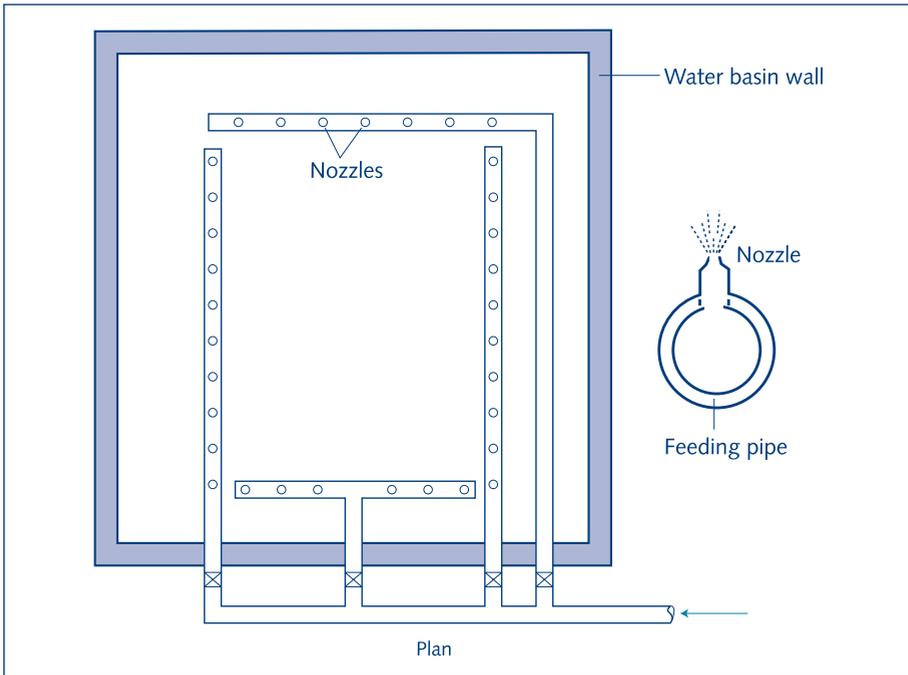


Fig. 13.5. Nozzled spray aerator

To avoid clogging, the nozzle openings should be fairly large, more than 5 mm, but at the same time the construction should be such that the water is dispersed into fine droplets. Many designs have been developed to meet these requirements. A simple spray aerator using a baffle plate is shown in figure 13.6.

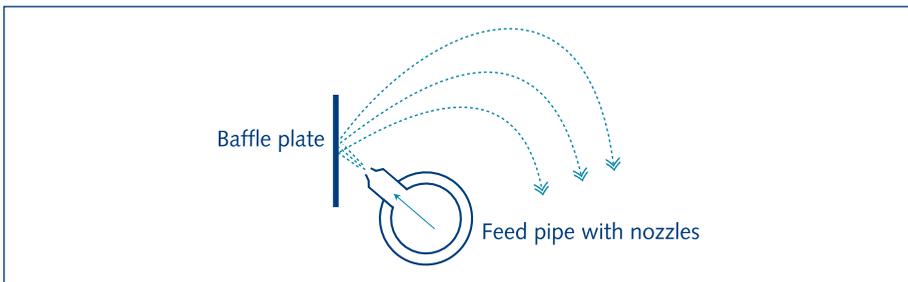


Fig. 13.6. Simple spray aerator using baffle plate

Figure 13.7 presents several examples of specially de-signed nozzles.

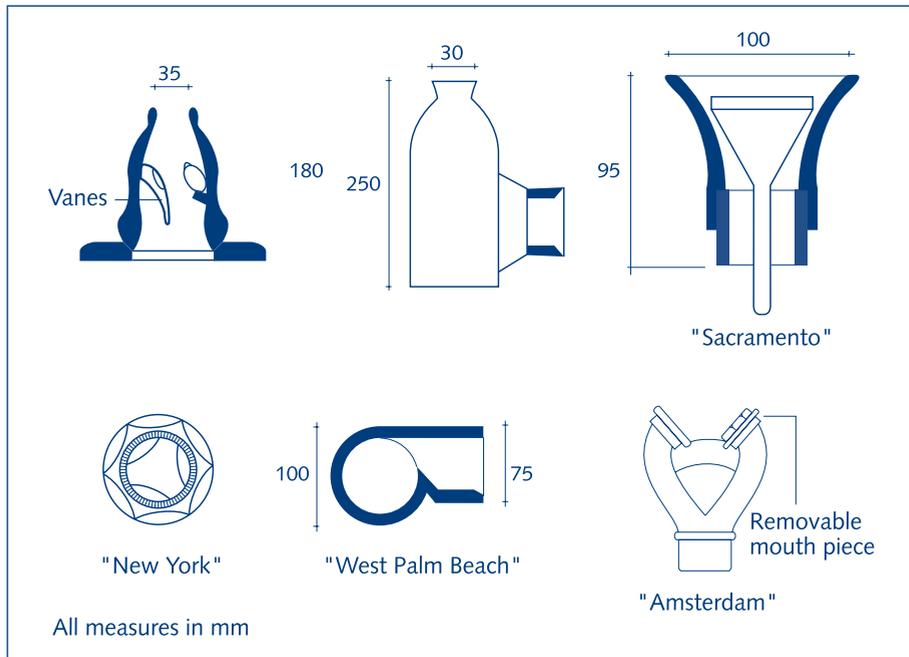


Fig. 13.7. Nozzles for spray aerators

13.3 Bubble aerators

The amount of air required for bubble aeration of water is small, no more than 0.3-0.5 m³ of air per m³ of water, and these volumes can easily be obtained by a sucking in of air. This is best demonstrated with the venturi aerator shown in figure 13.10. The aerator is set higher than the pipe carrying the raw water. In the venturi throat the velocity of flow is so high that the corresponding water pressure falls below the atmospheric pressure. Hence, air is sucked into the water. After passing the venturi throat, the water flows through a widening pipe section and the velocity of flow decreases with a corresponding rise of the water pressure. The fine air bubbles are mixed intimately with the water. From the air bubbles, oxygen is absorbed into the water. The release of carbon dioxide in this type of aerator is negligible, because the air volume of the bubbles is quite small.

Compared with spray aerators, the space requirements of venturi aerators are low; the overall head loss is about the same.

A submerged cascade aerator can operate by entrapping air in the falling sheets of water that carry it deep into the water collected in the troughs. Oxygen is then transferred from the air bubbles into the water. The total fall is about 1.5 m subdivided in 3-5 steps. The capacity varies between 0.005 and 0.05 m³/s. per metre of width.

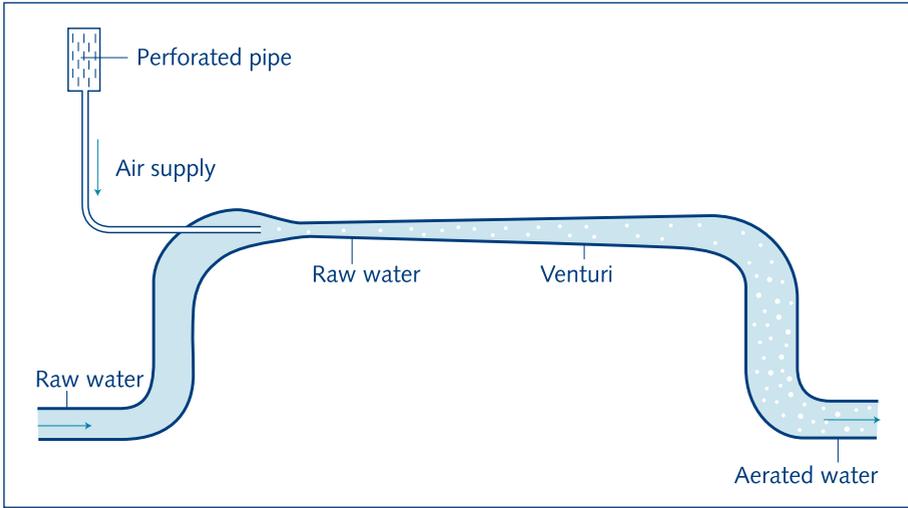


Fig. 13.8. Venturi aerator

Table 13.1 Aeration equipment characteristics (drinking water treatment)

Type	Hydraulic head required (m)	Air contact time (s)	Application	Notes
Spray*	1.5-7.6	1-2	CO ₂ removal taste and odour	Not so effective for iron and manganese
Cascade	0.9-3	0.5-1.5		
Multiple tray	1.5-3	0.5-1.5		

Source: Montgomery, James M, 1985

* typical area needed 10-30m² per 100 l/s flow rate.

Some additional information is available in the Degremont Water Treatment Handbook (1991), chapter 17 with some useful illustrations.

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14 Coagulation and flocculation

*Update by Ken Ives
Special contribution from
Samia Al Azharia Jahn*

14 Coagulation and flocculation

14.1 Introduction

Coagulation and flocculation provide the water treatment process by which finely divided suspended and colloidal matter in the water is made to agglomerate and form flocs. This enables their removal by sedimentation, dissolved air flotation or filtration. Colloidal particles (colloids) are midway in size¹ between dissolved solids and suspended matter. Colloids are kept in suspension (stabilised) by electrostatic repulsion and hydration. Electrostatic repulsion occurs because colloids usually have a surface charge due to the presence of a double layer of ions around each particle. Thus, the colloid has an electric charge, mostly a negative one. Hydration is the reaction of particles at their surface with the surrounding water. The resulting particle-water agglomerates have a specific gravity, which differs little from that of water itself.

The substances that frequently are to be removed by coagulation and flocculation are those that cause turbidity and colour. Surface waters in tropical countries often are turbid and contain colouring material. Turbidity may result from soil erosion, algal growth or animal/vegetable debris carried by surface run-off. Substances leached from decomposed organic matter, leaves, or soil such as peat may impart colour. Both turbidity and colour are mostly present as colloidal particles.

The electrostatic repulsion between colloidal particles effectively cancels out the electronic attraction forces (Van der Waals' forces) that would attach the particles together. Certain chemicals (called coagulating agents, coagulants) have the capacity to compress the double layer of ions around the colloidal particles. They reduce the range of the electrostatic repulsion, and thus enable the particles to flocculate, i.e. to form flocs. These flocs can grow to a sufficient size and specific weight to allow their removal by settling, flotation or filtration.

Generally water treatment processes involving the use of chemicals are not so suitable for small community water supplies. They should be avoided whenever possible. Chemical coagulation and flocculation should only be used when the needed treatment result cannot be achieved with another treatment process using no chemicals. If the turbidity and colour of the raw water are not much higher than is permissible for drinking water, it should be possible to avoid chemical coagulation in the treatment of the water. A process such as slow sand filtration or multi-stage filtration would serve both to reduce the turbidity and colour to acceptable levels, and to improve the other water quality characteristics, in a single unit. A roughing filter can serve to reduce the turbidity load on the slow sand filter, if necessary.

1 Size range: 5×10^{-3} – $2 \cdot 10^{-1}$ micrometer (0.2 microns)

14.2 Coagulants

Alum ($\text{Al}_2(\text{SO}_4)_3 \cdot n\text{H}_2\text{O}$) where $n=14, 16, \text{ or } 18$, depends on the form of alum supplied. This may be in liquid solution, broken crystalline granules 2-5 cm size (kibbled) or crystalline blocks. It is by far the most widely used coagulant. Iron salts (e.g. ferric chloride (FeCl_3), or ferric sulphate ($\text{Fe}_2(\text{SO}_4)_3 \cdot 9\text{H}_2\text{O}$) can be used as well and in some instances have advantages over alum. A significant advantage of iron salts over aluminium is the broader pH² range for good coagulation. Thus, in the treatment of soft coloured waters where colour removal is best obtained at low pHs, iron salts may be preferred as coagulants. Iron salts should also be considered for coagulation at high pHs, since ferric hydroxide is highly insoluble in contrast to aluminium salts, which form soluble aluminate ions at high pHs. Sodium aluminate is mostly used for coagulation at medium pHs. Synthetic organic polyelectrolytes have become available as coagulants but are generally not economical for small water supply systems, nor are they readily available.

The use of extracts from the crushed seeds of certain tropical trees is described in detail in the next section, 14.3.

Coagulants such as soluble aluminium and iron salts react with the alkalinity of the water, and hydrolyse in it. For example, alum reacts to form aluminium-hydroxide floc, $\text{Al}(\text{OH})_3$, a gelatinous precipitate. The required alkalinity may be naturally present in the water or it has to be added through dosage of lime, $\text{Ca}(\text{OH})_2$ or sodium carbonate, Na_2CO_3 (also called soda ash).

For good coagulation the optimal dose of coagulant should be fed into the water and quickly 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 experiment called the *jar test* is generally used for the periodic determination of the optimal dose.

The jar test may be briefly described as follows:

A series of samples of water are placed on a special multiple stirrer and the samples, typically 800 ml in 1-litre beakers (or jars), are dosed with a range of coagulant, e.g. 10, 20, 30, 40 and 50 mg/l, and are stirred vigorously for about one minute. Then follows a gentle stirring (10 minutes), after which the samples are allowed to stand and settle for 30-60 minutes. The samples are then examined for colour and turbidity and the lowest dose of coagulant that gives satisfactory clarification of the water is noted.

2 Measure of the acidity/alkalinity of water. Acid water has a pH below 7, the pH of alkaline water is higher than 7.

A second test involves the preparation of samples with the pH adjusted so that the samples cover a range (e.g. pH = 5, 6, 7 and 8). The coagulant dose determined previously is added to each beaker. Then follows stirring, flocculation and settlement as before. After this, the samples are examined and the optimum pH is determined. If necessary, a re-check of the minimum coagulant dose can be done.

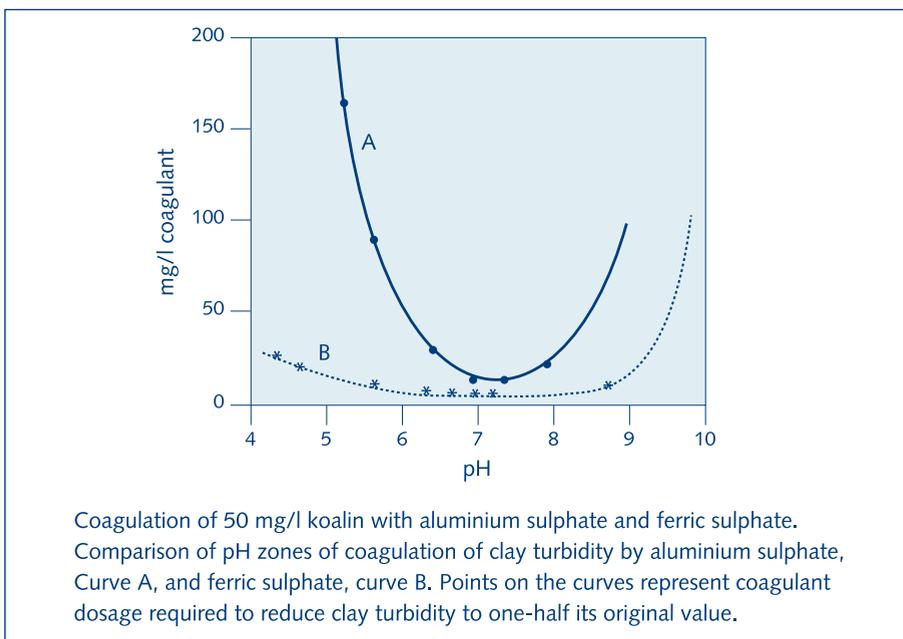


Fig. 14.1. pH zone-coagulation relationship
(Adapted from R.F. Packham)

As mentioned earlier, aluminium and iron salts have considerable differences in their pH zones of good coagulation. For alum the pH zone for optimum coagulation is quite narrow, ranging from about 6.5-7.5. The comparable range for ferric sulphate is considerably broader, a pH range of about 5.5-9.0 (Fig. 14.1). When the results of a jar test are plotted, this type of curve is typical.

The most common method of dosing the alum or ferric sulphate is in the form of a solution. Such a solution (usually of 3-7% strength, that is, 30-70 kg alum $\text{Al}_2(\text{SO}_4)_3 \cdot 14\text{H}_2\text{O}$ per m^3 water) is prepared in special tanks with a holding capacity of ten or more hours coagulant feeding requirements. Two tanks are required, one in operation, while the solution is being prepared in the other.

When using alum, one should keep in mind that in solutions of less than 1% strength, the chemical is hydrolysed (i.e. forms agglomerates with the chemical feed water) before it is dosed into the raw water. To prevent this, the solution should always have a strength of more than 1.5%.

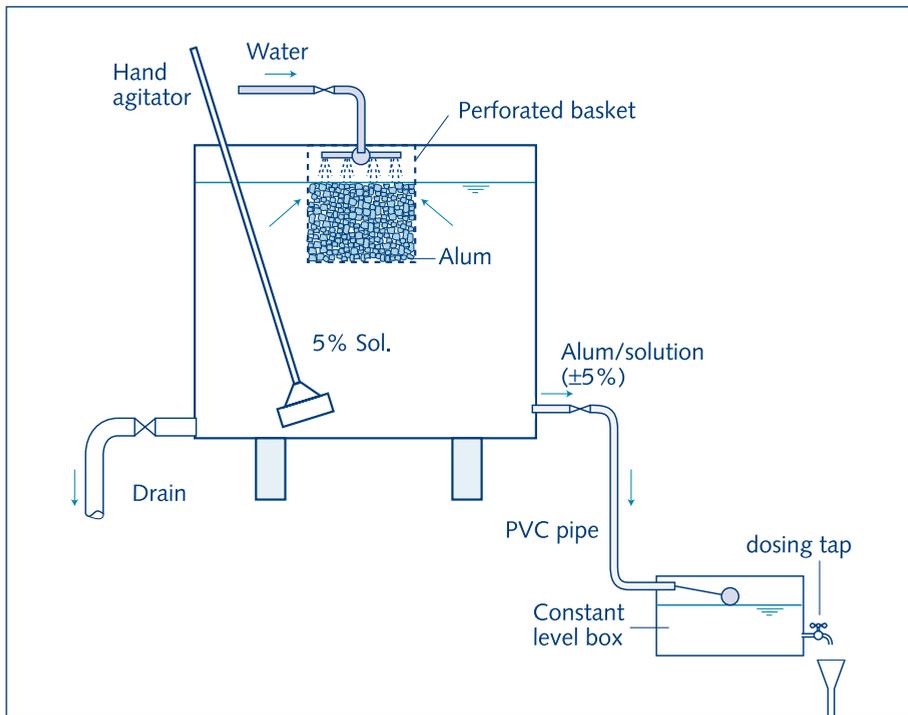


Fig. 14.2. Chemical feed arrangement for alum

Various chemical feed arrangements can be used. Figure 14.2 shows an example.

The simplest method of using lime is in the form of a suspension led into a special tank (called a *lime saturator*), to produce a saturated solution of calcium hydroxide. The size of the tank depends on the required lime dosage.

14.3 Coagulation and flocculation with extracts of particular seeds

Special contribution from Dr. Samia Al Azharia Jahn, Water Treatment Adviser

Botanical identification

Early attempts at water coagulation with plant materials arose through “self-help” by poor people who had no access to groundwater. In India the seeds of *Strychnos potatorum* (table 14.1) have been used to clarify turbid water since the first centuries AD. Where this wild tree was not available, kernels of *Prunus* species and seeds of legumes (*Papilionoidea*) became substitutes. In the Northern Sudan, seeds of *Moringa oleifera* were tried out instead of beans and groundnuts. A relationship between the botanical type and a content of coagulants (chemotaxonomy) could be detected for several plant genera and families.

Table 14.1 Seed extracts that contain outstanding natural coagulants¹

Plant	Chemical nature of coagulant	Raw water, turbidity range, application	Reference
<i>Strychnos potatorum</i> (clearing nut)	mannogalactan, probably anionic polyelectrolyte	Jamuna River (India): ca. 200-2200 NTU, recommended as coagulant aid	Bulusu and Sharma 1965
<i>Moringa oleifera</i> (horseradish tree)	basic polypeptides cationic polyelectrolyte	White Nile (Sudan): 50-250 NTU; Blue Nile: > 30 NTU-several thousand NTU, fast acting primary coagulant	Jahn 1986, 1988
<i>Moringa stenopetala</i>	similar basic polypeptides	performance like <i>Moringa oleifera</i>	Jahn 1986, 1988
five other species of <i>Moringa</i>	not yet known	performance like <i>Moringa oleifera</i>	Jahn 1986, 1988
<i>Vicia faba</i> (broad bean)	not known	Blue Nile: > 800 NTU slow acting primary coagulant	Jahn 1986, 1988
<i>Prunus armeniaca</i> (apricot) ² , and <i>P.amygdalis</i> (almond)	not known	River Nile, Egypt Changjiang, China	Alpino 1591, d'Arcet 1838, Lu Yu 1170
<i>Prunus persica</i> (peach)	not known	Oranje River, Lesotho rivers of Potosí Bolivia	Offringa 1993, Soliz Rodriguez 1984

1 For details and bibliography cf. Jahn 1986, 1988, 1994

2 Apricot kernels were in Egypt, North Sudan and Sichuan (China) still in use in the 20th century

Chemical composition and toxicology

Certain seeds contain substances for the growth of the seedling, which also have flocculating properties. The polyelectrolytes of *Moringa oleifera*, *M. stenopetala* and *Strychnos potatorum* are examples of such flocculating substances (table 14.1). The destabilisation of water colloids is due to positive or negative charges. The cationic electrolytes of *Moringa* are presumed to act like cations of metal coagulants. Alum and a seed extract different impacts on several chemical and physical characteristics of coagulation (table 14.2). *Moringa* seeds contain seed oil and a germicide. The smell and taste effect on the water is very small, even for the maximum *Moringa* seed dose (200-250 mg/l). In the low range of the optimal doses there is no germicide effect and there are no risks from toxic substances. The possible health effects from toxic substances in *Strychnos potatorum* seeds and *Prunus* kernels have not been investigated yet (Jahn, 1988, 1994).

Table 14.2 Differences between water coagulation with alum and *Moringa oleifera* seed extract

Parameters	Aluminium sulphate	Moringa seed extract
Destabilisation	Al ³⁺	cationic polyelectrolytes
Support of aggregation	adsorption at Al(OH) ₃ flocs	none
Efficiency at low turbidity	still satisfactory at 3 NTU	failing below 30 NTU
Mineralogical nature of suspended matter	minor impact on optimal doses	doses about 50% lower if montmorillonite clays are present (Blue Nile)
Pollution by organic matter	minor impact on optimal doses	great impact on doses and additional organic load
Floc	coarse due to aluminium hydroxides	very fine; result is slower settling; further decrease of settling at lower temperature
Sludge volume	great	small
pH of treated water	increase of acidity (effect of SO ₄ ²⁻)	no change
Hardness	increase of Ca ²⁺ due to neutralisation	no change

Preparation and application of Moringa seed coagulants

Seed quality

Only ripe *Moringa oleifera* seeds (with brown seed coat) give a high coagulant efficiency (Jahn, 1986). Improper treatment of the seeds leads to loss of coagulation capacity. It can be prevented if collection and treatment is done by agro-foresters or trained women and by using correct equipment.

Turbidity range

Experiences with seed coagulation relate to highly turbid waters only. If **optimal doses** and **proper stirring** are applied, seed extracts of *Moringa oleifera* can also reduce low turbidity levels to 5-10 NTU after one hour of settling. For raw water with turbidity below 30 NTU, alum is superior in performance (table 14.2). Thus, surface waters with varying turbidity (below 30 NTU for certain periods of the year) cannot always be treated adequately with seed extracts alone but need coagulant aids during such low turbidity periods. Use of coagulant aids is not feasible at household level.

Chemical water quality

Seed coagulants are more sensitive than alum to the mineralogical composition of suspended matter. Waters polluted by organic matter are only suitable for long-term treatment as long as the required seed coagulant dose is less than 250 mg/l. This is for both economic and microbiological considerations.

Climatic factors

Seed coagulants are mainly applicable as primary coagulants for tropical and subtropical countries. At low temperatures the settling of the fine flocs (table 14.2) is too slow and incomplete, unless seed treatment is combined with other coagulants.

Removal of pathogens

Like alum, Moringa seed extracts achieve a removal of faecal coliforms of more than 98% under optimal conditions. However, if the residual turbidity is more than 30-40 NTU, the treated water still has the same contamination as raw water. Therefore, optimal dosing is required for small plants and for household application.

Determination of optimal doses

Simplified ("village") jar tests are a convenient alternative where the usual measurements cannot be afforded or electricity is not available. Special devices have been designed for stirring the seed coagulant at plant level. A seed coagulant suspension has different reaction properties than a solution of a purified chemical such as alum. The coagulant efficiency is strongly reduced by shortening the time needed for thorough mixing of the seed extract with turbid water.

Involvement of women, teachers and commerce

Asian and African women from the lower classes are mainly responsible for the discovery of seed extracts as coagulants for turbid water. Lack of knowledge on application has limited the possible improvement of water quality. Water coagulation at household level cannot be learned from a demonstration alone. Therefore, the involvement of teachers or commercial outlets is required to help the women in determining the optimal seed coagulant dose, and to distribute or sell standard solutions of the seed coagulant.

Socio-economic challenges

Since Moringa oleifera is a multipurpose tree, certain economic interests may endanger long-term uses of seed extracts for water coagulation. Sometimes people prefer to have the fruit trees in their own gardens rather than on community lands (women then have no right to harvest the seeds). There may also be a commercial preference to sell green Moringa pods as a delicacy to urban customers or factories. On the other hand, removal of the seed oil for other uses and application of press cake extracts instead of seed extracts has been tried out in small plants of Malawi and Tanzania (Folkard et al., 1993). There are commercial developments too in the production of a completely biodegradable Moringa coagulant, known under the commercial name Phytofloc™.

14.4 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 mixing has to be rapid, because the hydrolysis of the coagulant is almost instantaneous (within a few seconds). The destabilisation of colloids also takes a similar short time.

The location of the rapid mixer should be near to the “chemical house” where solutions of chemicals are prepared. The feeding pipes then will be of short length. It is also desirable to place the rapid mixing device close to the flocculators. To combine both these requirements in the layout of a treatment plant is often quite difficult.

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

- Hydraulic rapid mixing
- Mechanical rapid mixing

Hydraulic rapid mixing

For hydraulic rapid mixing, arrangements are used such as channels or chambers with baffles producing turbulent flow conditions, overflow weirs, and hydraulic jumps (Figs. 14.3; 14.4; 14.5). 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.

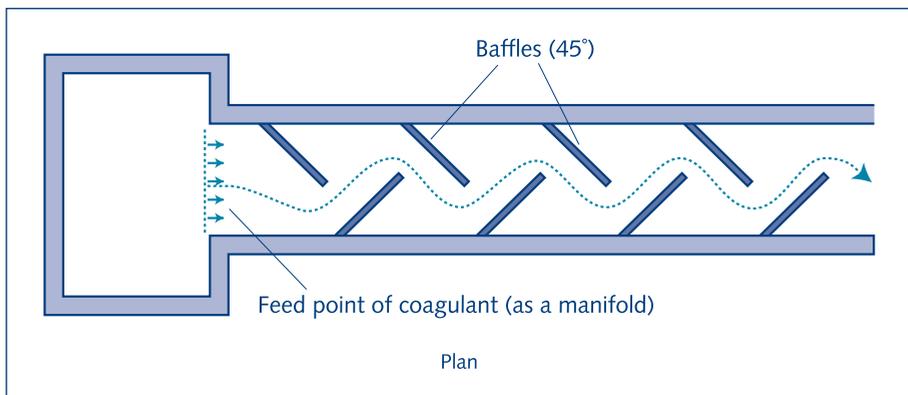


Fig. 14.3. Baffled channel for rapid mixing (not very effective)

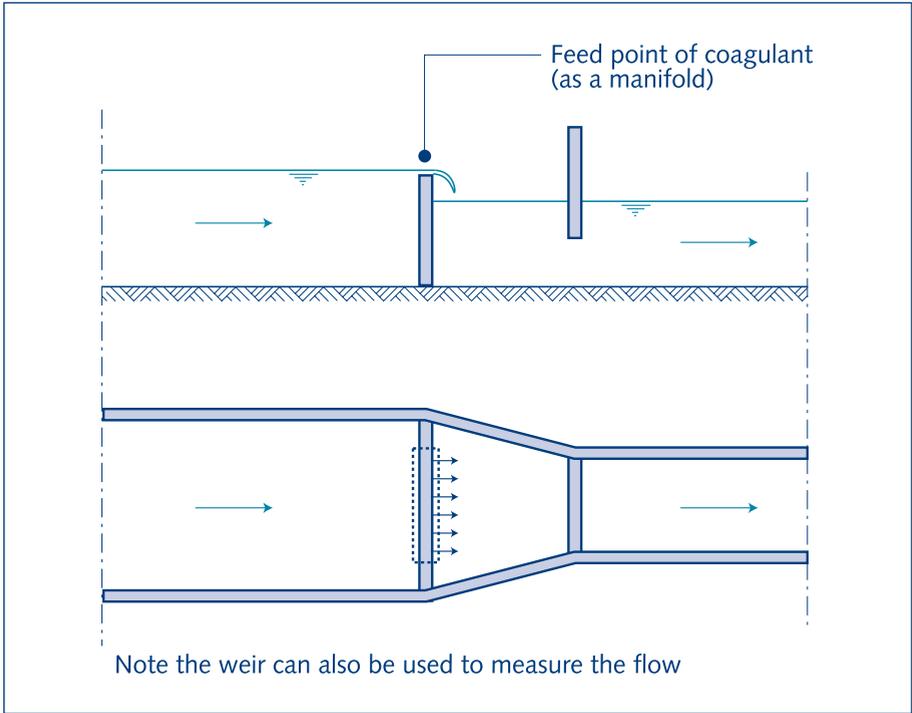


Fig. 14.4. Overflow weir

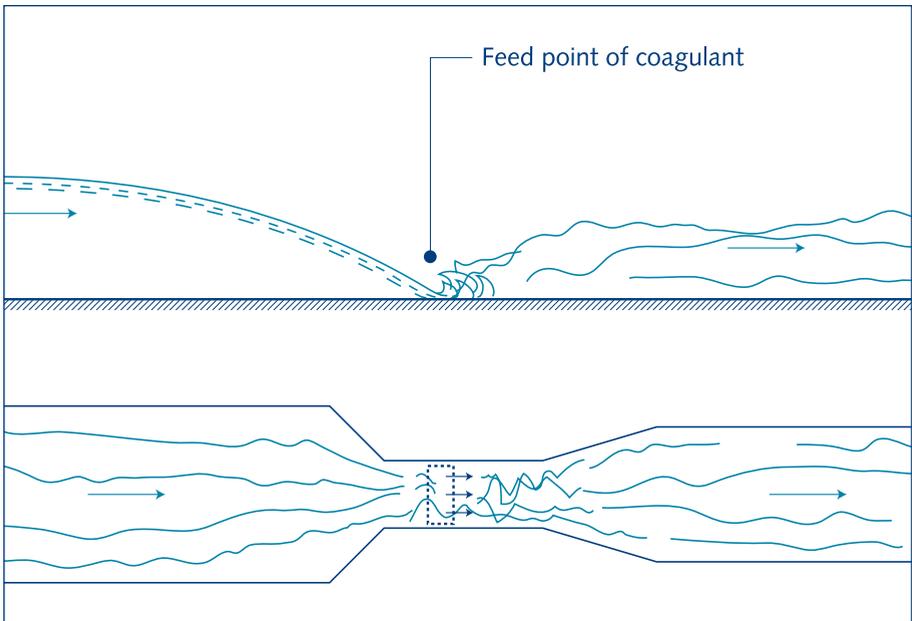


Fig. 14.5. Hydraulic jump

Mechanical rapid mixing

With mechanical mixing the power required for agitation of water is imparted by impellers, propellers or turbines (*rapid mixers, flash mixers, and turbo mixers*). See figure 14.6.

Generally mechanical rapid mixers are less suitable for small treatment plants than hydraulic ones since they require a reliable and continuous supply of power and maintenance.

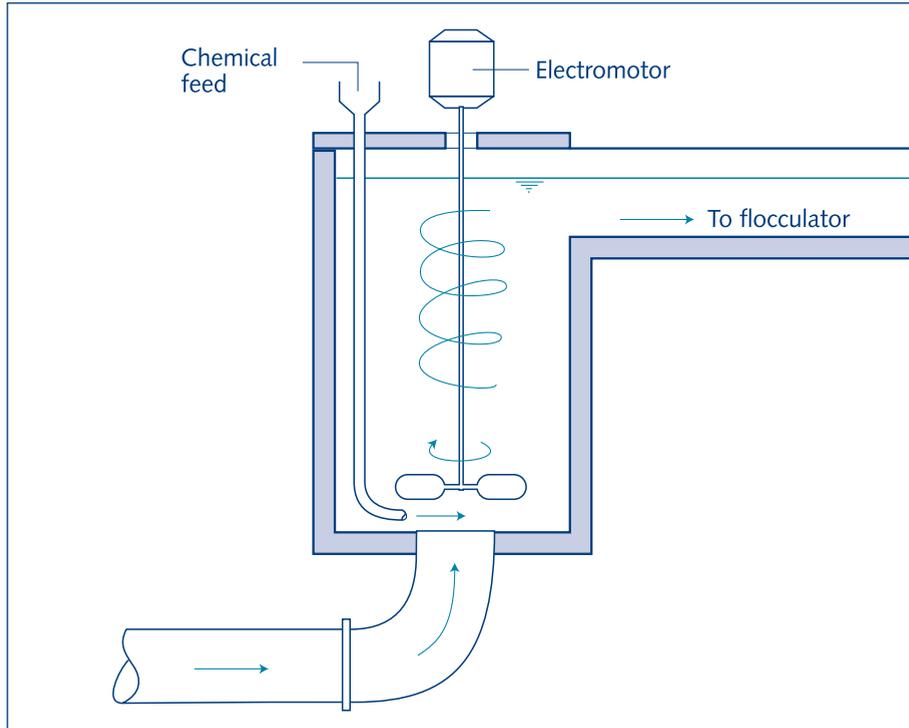


Fig. 14.6. Mechanical mixer

14.5 Flocculation

Flocculation is the process of gentle and continuous stirring of coagulated water for the purpose of forming 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, dissolved air flotation or filtration. The efficiency of the flocculation process is largely determined by the number of collisions between the minute coagulated particles per unit of time. There are mechanical and hydraulic flocculators.

In mechanical flocculators the stirring of the water is achieved with devices such as paddles, paddle reels or rakes.

These devices can be fitted to a vertical or horizontal shaft. Vertical shaft flocculators are usually placed in a square tank with several chambers (four or more). They have the advantage of having only bearing in the water, and no gland is necessary as the motor and gearing are above the water. With horizontal shaft flocculators having a traverse flow, one should provide at least four rows of shafts, with partitions of baffles (stop logs), so as to avoid short-circuiting.

In hydraulic flocculators the flow of the water is so influenced by small hydraulic structures that a stirring action results. Typical examples are channels with baffles, flocculator chambers placed in series (e.g. *Alabama*-type flocculator) and gravel bed flocculators.

The main shortcomings of hydraulic flocculators are:

- No adjustment is possible to changes of raw water composition.
- No adjustment is possible to the water production rate of the treatment plant.
- The head loss is often appreciable.
- They may be difficult to clean.

Their advantages are that they have no motor power, electric cables switchgear, etc. to maintain and general maintenance is easier.

Design of flocculators

In the design of a flocculator installation not only the velocity gradient (G) should be taken into account, but also the detention time (t). The product $G.t$ gives a measure for the number of particle collisions, and thus for the floc formation action⁵.

The equation for computing the velocity gradient is: $G = (P/(V.m))^{1/2}$ in which

G = velocity gradient (s^{-1})

P = power transmitted to the water (watt = $rg hQ$)

r = density of water = 1000 kg/m^3

g = gravitational acceleration = 9.81 m/s^2

h = head loss of the water passing through the flocculator in m

Q = volumetric flow rate in m^3/s

V = volume of water to which the power is applied; where applicable, the volume of the mixing tank or basin (m^3)

m = dynamic viscosity of water (kg/m.s), see table 14.3

5 Scientifically the complete dimensionless group is $G.t.C$. where C = concentration volume/volume of floc particles in the water. But in the types of mechanical and hydraulic flocculators described here, which are flow-through (that is, no flocs are retained) the value of C is very small and usually ignored. C becomes more important in flocculators where the floc is retained by recirculation or in suspension as a floc blanket. Operationally these are not suitable for small community water supplies.

Table 14.3 Dynamic viscosity of water μ (kg/m.s)

Temperature $t(^{\circ}\text{C})$	15	20	25
Dynamic viscosity μ (kg/m.s)	1.14×10^{-3}	1.01×10^{-3}	0.96×10^{-3}

Table 14.4 Flocculator design criteria

Design factor	G (s^{-1})	t (s)	$G.t$
Range	10-100	1 200-1 800	30 000-150 000
Typical value	45-90	1 800	50 000-100 000

For each individual flocculator the optimal $G.t$ value should be carefully selected, and taken as high as is consistent with the optimal formation of flocs without causing disruption or disintegration of the flocs after they have formed. Chemicals such as activated silica or polyelectrolytes (coagulant aids) can improve the internal cohesion of the flocs.

14.6 Hydraulic flocculators

Baffled channel flocculators

For horizontal-flow baffled flocculation channels (Fig. 14.7), the design water velocity usually is in the 0.10-0.30 m/s range. Detention time normally is 15-20 minutes.

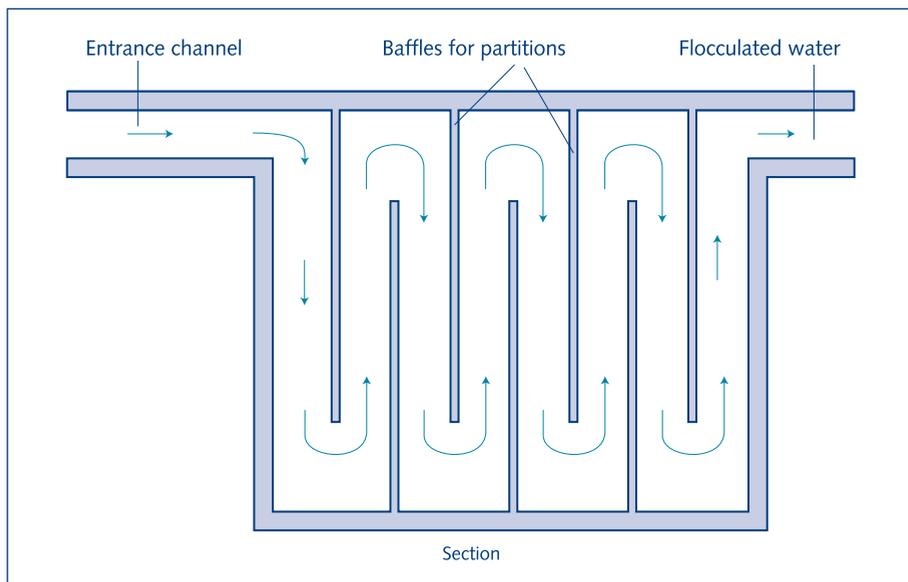


Fig. 14.7. Horizontal-flow baffled channel flocculator (plan)

This type of flocculator is well suited for very small treatment plants. The efficiency, however, is highly dependent on the depth of water in the baffled channel.

Flocculators with vertical flow through baffled chambers (Fig. 14.8) are mostly used for medium and larger-size water treatment plants. The water flow velocity range is 0.1-0.2 m/s. Detention time is 10-20 minutes. Cleaning arrangements are called for because of deposits in the flocculator.

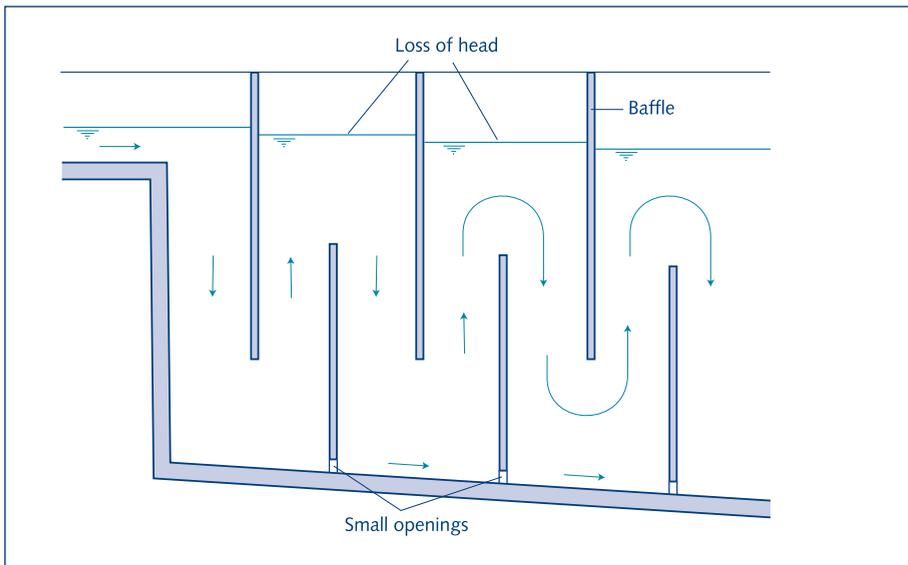


Fig. 14.8. Vertical-flow baffled chamber flocculator (cross-section)

Alabama-type flocculators

Alabama-type flocculators are hydraulic flocculators having separate chambers placed in series through which the water flows in two directions (Fig. 14.9). The water flows from one chamber to the next, entering each adjacent partition at the bottom end through outlets turned upwards. This type of flocculator was initially developed and used in the state of Alabama (U.S.A.) and later introduced in Latin America.

For effective flocculation in each chamber, the outlets should be placed at a depth of about 2.50 m below the water level.

Common design criteria are:

Rated capacity per unit chamber:	25-50 l/s per square metre
Velocity at turns	0.40-0.60 m/s
Length of unit chamber (L)	0.75-1.50 m
Width (B)	0.50-1.25 m
Depth (h)	2.50-3.50 m
Detention time (t)	15-25 minutes

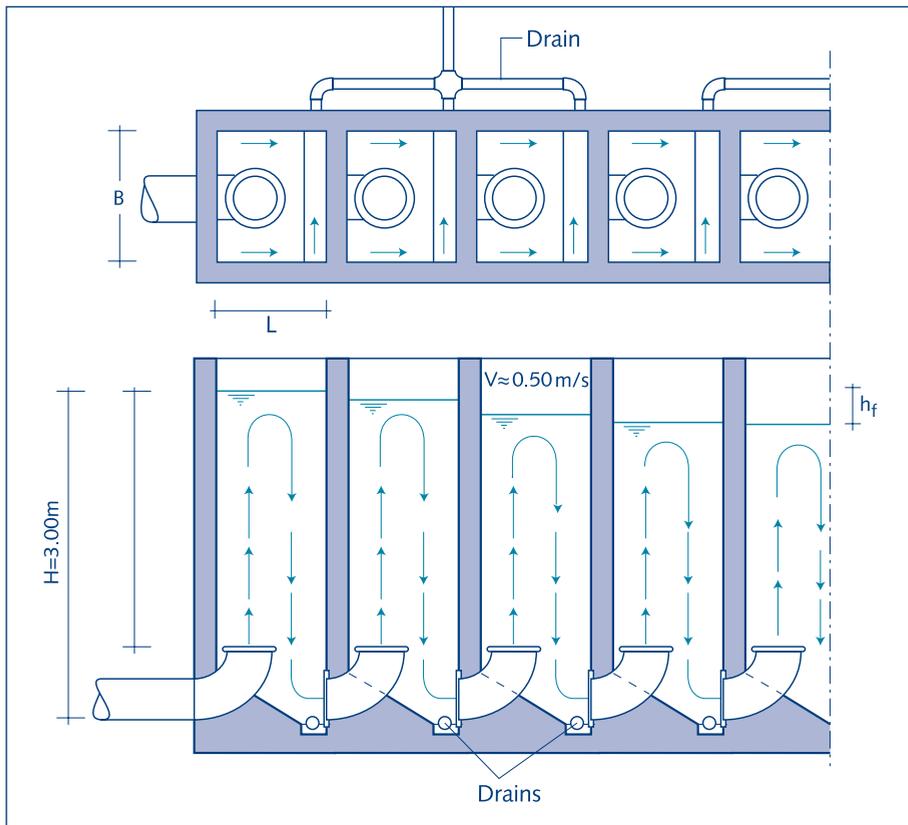


Fig. 14.9. Alabama-type flocculator

The loss of head in this type of flocculator normally is about 0.35-0.50 m for the entire unit. The velocity gradient is usually in the 40-50 s^{-1} range.

Table 14.5 provides practical guidance for the design of an *Alabama*-type flocculator.

Example:

Flow $Q = 1.2 \text{ m}^3/\text{minute}$. Detention time = 15 mm. Size of curved pipe: 250 mm (10").

Unit chamber measures $0.60 \times 0.75 \text{ m}^2$. Volume of unit chamber: 1.3 m^3 .

Total volume required. $15 \times 1.2 = 18 \text{ m}^3$ Number of chambers $18/1.3 = 14$.

Hydraulic jet mixer and flocculator

In a jet flocculator the coagulant (alum) is injected in the raw water using a special orifice device. The water is then jetted into a tapered cylinder placed above the nozzle. The resulting jet pump action gives a gentle stirring of the water for floc formation, and part of the formed flocs are recycled (Fig. 14.10). Through the combination of these two actions excellent flocculation results can be achieved.

Table 14.5 Guidance for *Alabama-type* flocculator design

Flow Rate Q (l/s)	Width B (m)	Length L (m)	Diameter D (mm)	Unit chamber area (m ²)	Unit chamber volume (m ³)
10	0.60	0.60	150	0.35	1.1
20	0.60	0.75	250	0.45	1.3
30	0.70	0.85	300	0.6	1.8
40	0.80	1.00	350	0.8	2.4
50	0.90	1.10	350	1.0	3.0
60	1.00	1.20	400	1.2	3.6
70	1.05	1.35	450	1.4	4.2
80	1.15	1.40	450	1.6	4.8
90	1.20	1.50	500	1.8	5.4
100	1.25	1.60	500	2.0	6.0

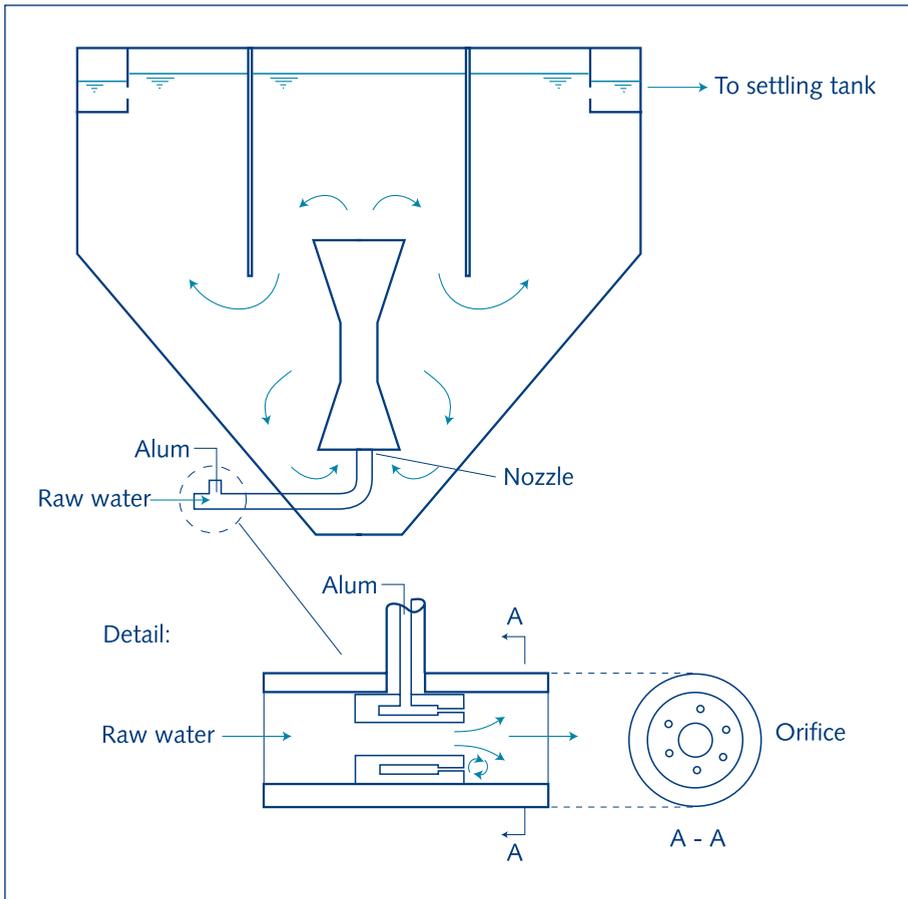


Fig. 14.10. Hydraulic jet flocculator

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Web sites

Water Treatment Plant Operation (training programme):
<http://www.owp.csus.edu/WTPO1.html>

15 Sedimentation

Update by Ken Ives

15 Sedimentation

15.1 Introduction

Sedimentation is the settling and removal of sus-pended particles that takes place when water stands still in, or flows slowly through a basin. Due to the low velocity of flow, turbulence will generally be absent or negligible, and particles having a mass density (specific weight) higher than that of the water will be allowed to settle. These particles will ultimately be deposited on the bottom of the tank forming a sludge layer. The water reaching the tank outlet will be in a clarified condition.

Sedimentation takes place in any basin. Storage basins, through which the water flows very slowly, are particularly effective but not always available. In water treatment plants, settling tanks specially designed for sedimentation are widely used. The most common design provides for the water flowing horizontally through the tank but there are also designs for vertical¹ or radial flow. For small water treatment plants, horizontal-flow, rectangular tanks generally are both simple to construct and adequate.

The efficiency of the settling process will be much reduced if there is turbulence or cross-circulation in the tank. To avoid this, the raw water should enter the settling tank through a separate inlet structure. Here the water must be divided evenly over the full width and depth of the tank. Similarly, at the end of the tank an outlet structure is required to collect the clarified water evenly. The settled-out material will form a sludge layer on the bottom of the tank. Settling tanks need to be cleaned out regularly. The sludge can be drained off or removed in another way. For manual cleaning (e.g. scraping), the tank must first be drained.

15.2 Settling tank design

The efficiency of a settling tank in the removal of suspended particles can be determined using as a basis the settling velocity (s_0) of a particle that in the detention time (T) will just traverse the full depth (H) of the tank. Using these notations (see fig. 15.1), the following equations are applicable:

$$s_0 = \frac{H}{T}, \quad T = \frac{BLH}{Q}, \quad \text{so that } s_0 = \frac{Q}{BL} \text{ (m}^3\text{/m}^2\cdot\text{h} = \text{m/h)}$$

s_0 = settling velocity (m/h)

T = detention time (h)

Q = flow rate (m³/h)

H = depth of tank (m)

B = width of tank (m)

L = length (m)

1 The operational requirements of vertical-flow floc blanket-type settling (formerly known as sludge blanket) tanks are so strict that they are generally not suitable for small water treatment plants.

Assuming an even distribution of all suspended particles in the water over the full depth of the tank (by way of an ideal inlet structure), particles having a settling velocity (s) higher than s_0 will be completely removed. And particles that settle slower than s_0 will be removed for a proportional part, $s:s_0$.

This analysis shows that the settling efficiency basically only depends on the ratio between the influent flow rate and the surface area of the tank. This is called the *surface loading*. It is independent of the depth of the tank. In principle, there is no difference in settling efficiency between a shallow and a deep tank.

The settling efficiency of a tank may therefore be greatly improved by the installation of an extra bottom as indicated in figure 15.1. The effective surface area would be greatly increased and the surface loading would be much lower.

The design of a settling tank should properly be based on an analysis of the settling velocities of the settleable particles in the raw water.

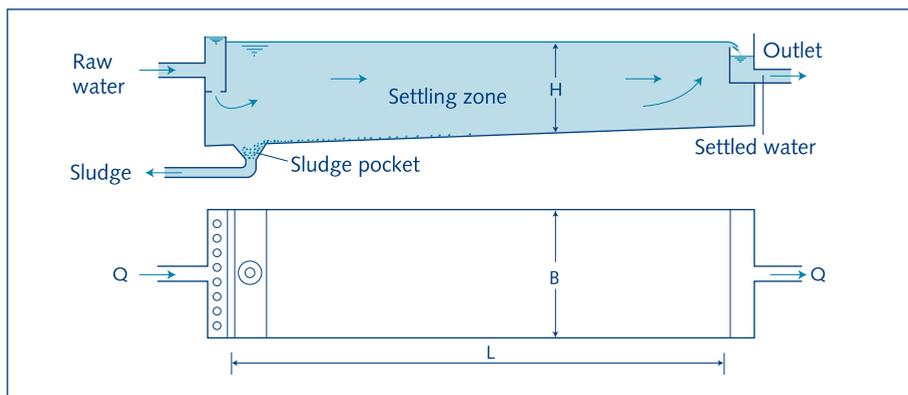


Fig. 15.1. Settling tank with extra bottom

Where sedimentation is used without pre-treatment, (this is called *plain sedimentation*) for the clarification of river water, the surface loading generally should be in the range from 0.1-1 m³/hour. For settling tanks receiving water that has been treated by chemical coagulation and flocculation, a higher loading is possible, somewhere between 1 and 3 m³/hour. In both cases, the lower the surface loading the better the clarification of the water: the settled water will have less turbidity.

The above considerations ignore the effects of turbulence, short-circuiting and bottom scour caused by inlet and outlet design and wind. Weirs and orifices should be arranged to reduce jetting at the inlet, and upward flow at the outlet. Windbreaks of about 2 m high can reduce wind disturbance. To keep these effects to a minimum, the tank should not be too shallow, at least 2 m deep or more, and the ratio between length and width should be between 3 and 8.

The horizontal velocity of flow, computed as $v = Q/BH$, will then be in the 4-36 m/hour range. A tank 2 m deep or more could accommodate mechanical sludge re-moval equipment but small installations are better cleaned manually. This is done at intervals varying from one to several weeks. The depth of the tank should be adequate to accommodate the sludge accumulating at the bottom between the cleanings.

For a further elaboration of settling tank design, take the example of a town with a future population of 10,000 inhabitants, requiring an average of 40 litres/day per person. Assuming a maximum daily de-mand of 1.2 times the average demand, the design capacity should be:

$$Q = 10,000 \times \frac{4}{1000} \times 1.2 = 480 \text{ m}^3/\text{day} = 20 \text{ m}^3/\text{hour}$$

If the raw water source is turbid river water, it may be subjected to plain sedimentation as a first treat-ment. Two settling tanks should be built. If the second tank serves only as a reserve for when the first tank is out of operation, then each of the two settling tanks has to be designed to take the full design flow ($Q = 20 \text{ m}^3/\text{hour}$). An alternative is to provide two tanks of $10 \text{ m}^3/\text{hour}$ capacity each. This would give a saving in construction costs. With one of these tanks out of operation for cleaning, the other tank has to be overloaded for the duration of the cleaning operation. In many situations this can be quite acceptable. Whether this approach may be followed must be determined in each individual case.

If experience with other installations using the same water source indicates that a surface loading of 0.5 m/hour gives satisfactory results, the sizing of the tank for a design capacity of $20 \text{ m}^3/\text{hour}$ would be as follows:

$$\frac{Q}{BL} = \frac{20}{BL} = 0.5, \text{ so that } BL = 40 \text{ m}^2$$

The tank dimensions could be, for instance: $B = 3 \text{ m}$, $L = 14 \text{ m}$.

With a depth of 2 m the tank would have about 0.5 m available for filling up with sludge deposits before cleaning is needed. The horizontal flow velocity would be:

$$v = \frac{Q}{BH} = \frac{20}{(3).(1.5)} = 4.44 \text{ m/hour}$$

This is well within the design limits quoted above. Assuming that during periods of high turbidity the river water contains a suspended load of 120 mg/l which is to be reduced to 10 mg/l by sedimentation, then 110 grams of silt will be retained from every cubic metre of water clarified. With a surface loading of 0.5 m/hour this means an average sludge accumulation of 55 gram/m^2 per hour; that is for sludge having a dry matter content of 3%, an amount of $55:0.03 = 1830 \text{ cm}^3/\text{m}^2$ per hour = 1.83 mm/hour. At the inlet end of the tank the deposits will accumulate faster, probably about 4 mm/hour, so that for an allowable accumulation of 0.5 m an interval of 125 hours or 5 days between

cleanings is to be expected. When the periods of high turbidity are infrequent and of short duration, this is certainly acceptable.

15.3 Construction

Settling tanks with vertical walls are normally built of masonry or concrete; dug settling basins mostly have sloping banks of compacted ground with a protective lining, if necessary.

Medium- and large-sized settling tanks generally have a rectangular plan and cross-section. To facilitate sludge removal it is convenient to have the tank bottom slope lightly towards the inlet end of the tank where the sludge pocket is situated.

As described in section 15.1, a settling tank should have a separate inlet arrangement ensuring an even distribution of the water over the full width and depth of the tank. Many designs can be used; figure 15.2 shows a few examples. The arrangement shown to the left consists of a channel over the full width of the tank with a large number of small openings in the bottom through which the water enters the settling zone. For a uniform distribution of the influent these openings should be spaced close to each other, less than 0.5 m apart, and their diameter should not be too small (e.g. 3-5 cm) otherwise they may clog up. The channel should be generously sized, with a cross-sectional area of at least twice the combined area of the openings. A settling tank as elaborated in the example given earlier, with a capacity of 20 m³/h and a width of 3 m, would have in the inlet channel about 6 holes of 4 cm diameter. The inlet channel itself would be about 0.4 m deep and 0.3 m wide.

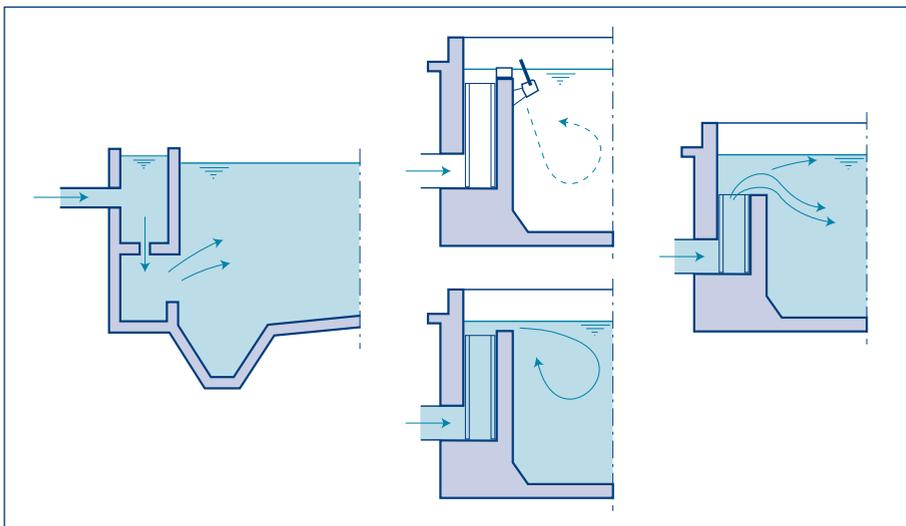


Fig. 15.2. Inlet arrangements

Frequently, the effluent water leaves the tank over weirs. Sometimes one weir is adequate but to prevent the settled material from being picked up again, the draw-off of the water should always be gentle and more weirs may have to be provided (combined length nB). The following formula can be used for computing the total weir length required:

$$nB = \frac{Q}{5 H_s^0}$$

In the example of the preceding section:

$$(n)(3) = \frac{20}{(5)(1.5)(0.5)} \text{ or } n = 2$$

Outlet arrangements using one and more overflow weirs are shown in figure 15.3.

When low weir overflow rates are used, the precise horizontal positioning of the weir crest is of importance. In the above example the $5 \text{ m}^3/\text{hour}$ overflow rate would give an overflow height Δ of only 8 mm. A slight deviation of the weir crest from the horizontal would then already cause a very uneven withdrawal of the settled water. To avoid this as much as possible, the weir crest may be made of a special metal strip fastened with bolts to the con-crete weir wall. The top of such a strip is not straight; it has triangular notches at intervals (Fig. 15.4). Another solution is shown in figure 15.3 to the left. Openings in the settling tank wall are used which should be of a smaller diameter than for the similar inlet construction. For $Q = 20 \text{ m}^3/\text{hour}$, 6 openings of 2.5 cm diameter should be adequate. The suspended matter content of the effluent water normally being low, the danger of clogging the holes is small and cleaning should not be needed frequently.

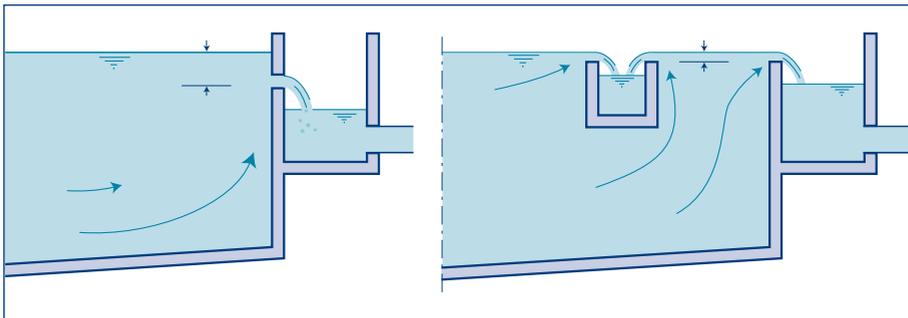


Fig. 15.3. Outlet arrangements

Earlier it was mentioned that small settling tanks may also be constructed simply as a basin with vertical walls of wooden sheet piling or similar material (Fig. 15.5), or with sloping walls (Fig.15.6). In the latter case only half the wetted slope length should be taken into account when computing the effective surface area and, thus, the surface loading of the settling basin. In both cases the basin should be constructed on raised ground to prevent flooding during wet periods.

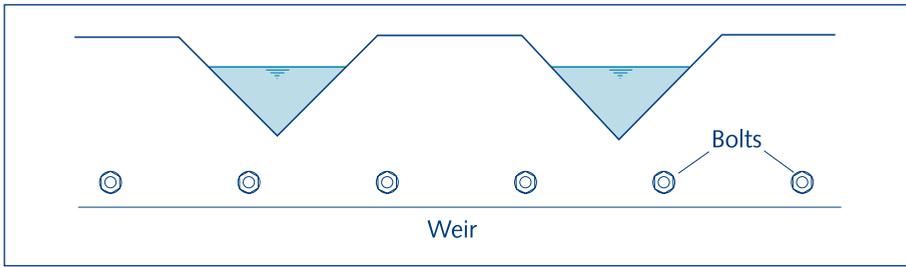


Fig. 15.4. Notched overflow weir

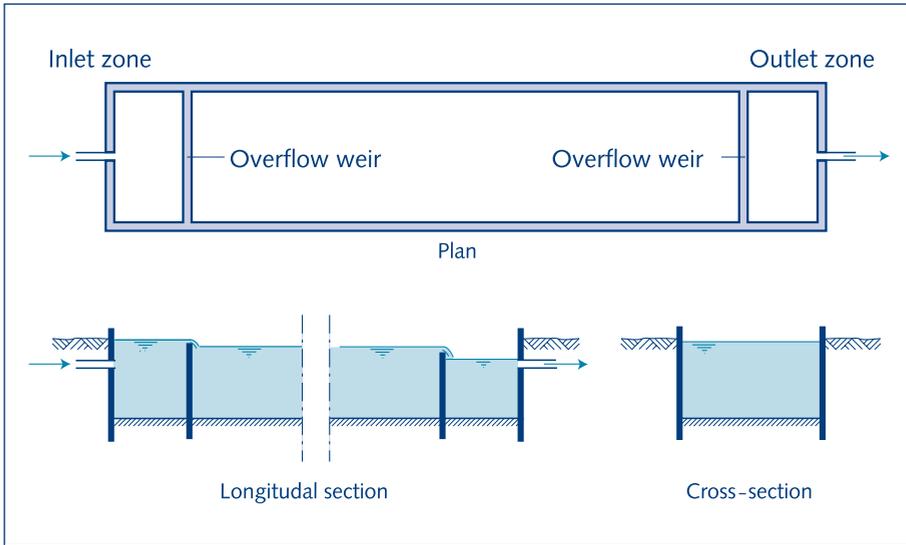


Fig. 15.5. Settling basin constructed with wooden sheet piles

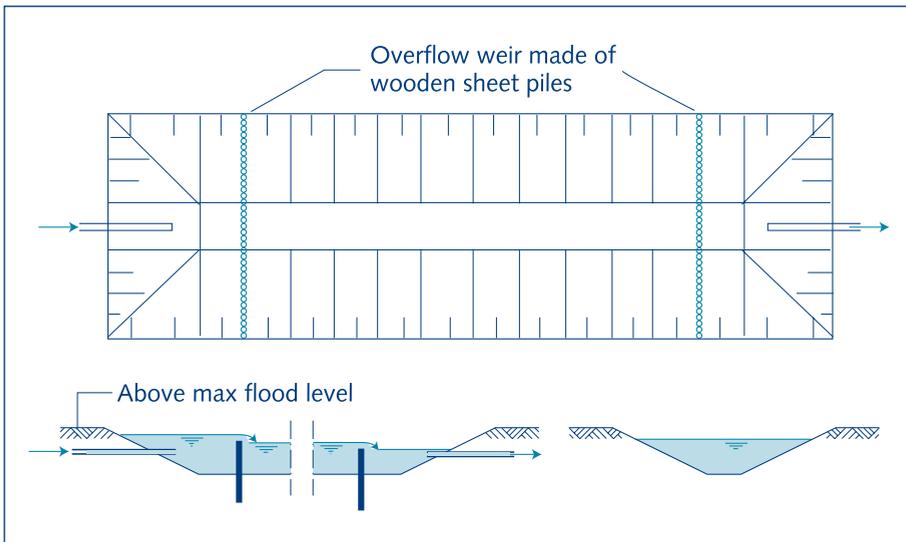


Fig. 15.6. Dug basin as settling tank

15.4. Tilted plate and tube settlers

The improvement in settling efficiency that can be obtained by the installation of one extra bottom (tray) in a settling tank can be greatly increased by using more trays as shown in figure 15.7. 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 steeply at an angle of 40-60° to the horizontal. The most suitable angle depends on the characteristics of the sludge, which will vary for different types of raw water. Such installations are called tilted plate settling tanks. This type of tank is shown schematically in figure 15.8. Figure 15.9 shows a cross-section.

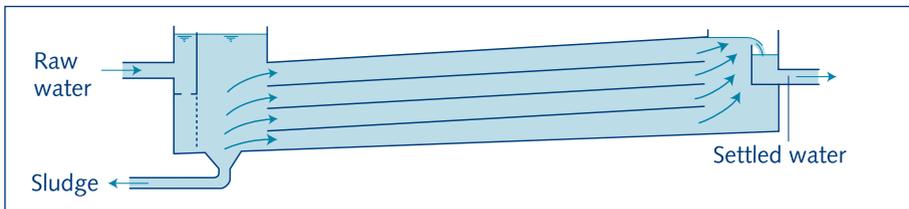


Fig. 15.7. Multiple-tray settling tank

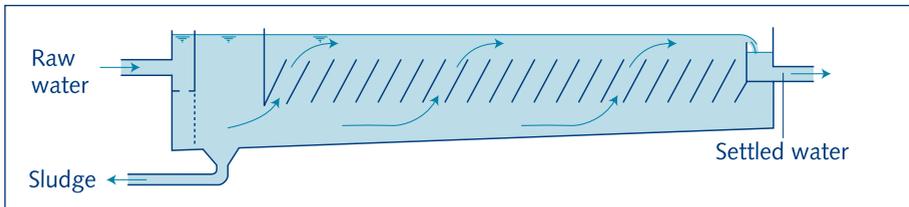


Fig. 15.8. Tilted plate settling tank

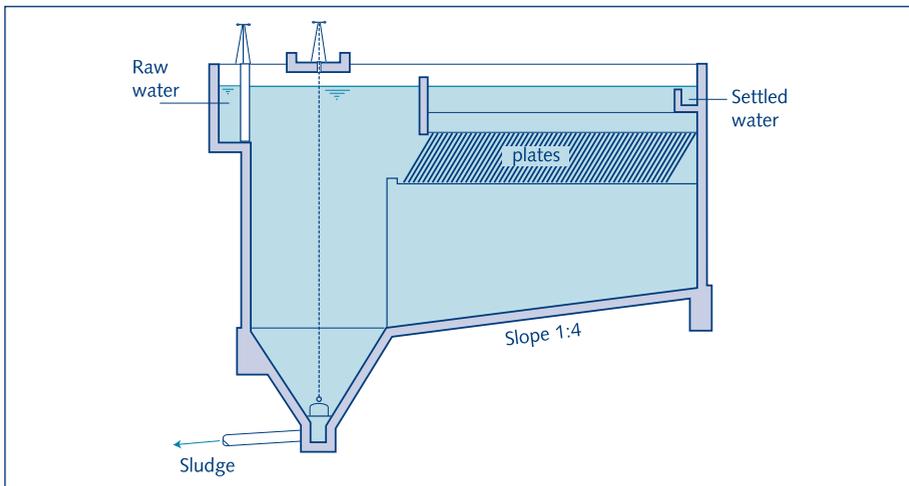


Fig. 15.9. Settling tank with tilted plates

For large tanks, quite sophisticated systems of trays or plates, have been devised but in small installations flat or corrugated plates and upward flow of the water are frequently the most suitable. For any clarification duty, tilted plate settling tanks have the advantage of packing a large capacity in a small volume. The effective surface being large, the surface loading will be low, and the settling efficiency, therefore, high. The surface loading may be computed as:

$$s = \frac{Q}{nA}$$

s = surface loading ($\text{m}^3/\text{m}^2\cdot\text{h}$)

Q = rate of flow (m^3/h)

A = bottom area of the tank (m^2)

n = multiplication factor depending on the type and position of the tilted plates.

Water enters at the bottom of the settling tank, flows upwards, passes the tilted plates, and is collected in troughs (Fig. 15.10). As the water flows upwards past the plates the settleable particles fall to the plates. When they strike it they slide downwards, eventually falling to the area beneath the plates. An individual particle might enter the plate channels several times before it agglomerates and gathers sufficient weight to eventually settle to the tank floor.

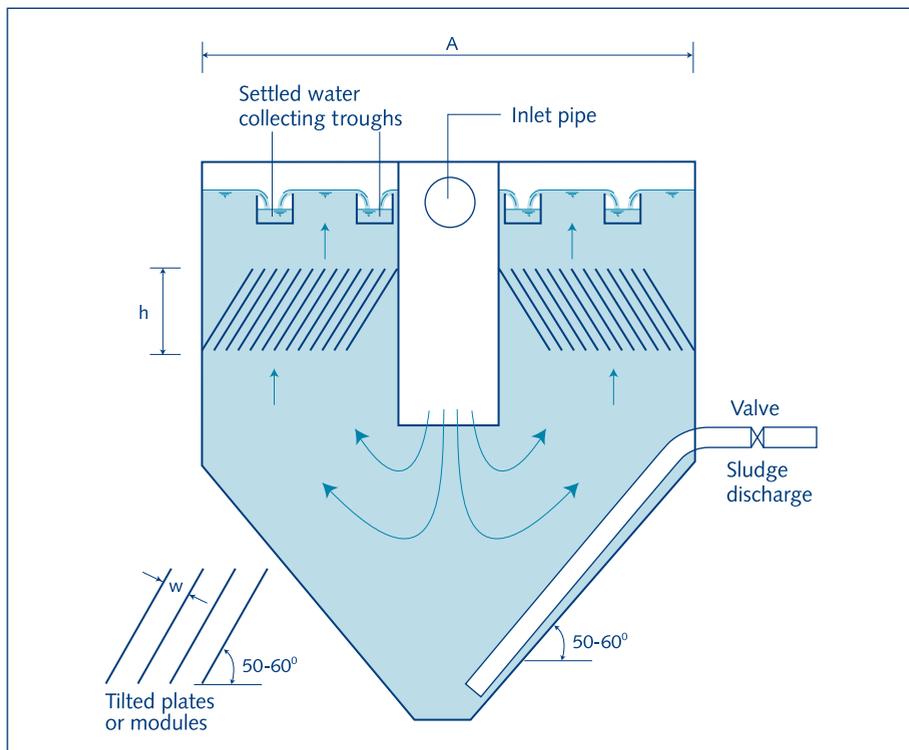


Fig. 15.10. Tilted plate settling tank design parameters

Assuming $h = 1.5 \text{ m}$, $w = 0.05 \text{ m}$, $\alpha = 55^\circ$ and plates with a thickness of 6 mm, we find $n = 16!$ One should keep in mind that the sludge deposits per unit bottom area will also be 16 times greater, for the same influent flow rate. Manual sludge removal will probably be impractical. In a tank having a square plan, rotating sludge scrapers might be used. Another possibility is the use of hopper-bottomed tanks with the walls sloping at about 50° to the horizontal. The depth of such a tank will be considerable and the costs of construction are likely to be much greater than for flat-bottomed tanks. Sludge discharge is carried out through the draining of water from the hopper-bottom section of the tank (this is called *bleeding*).

Instead of tilted plates, closely packed tubes may be used. These can easily be made of PVC pipes, usually of 3-5 cm internal diameter and sloping about 60° to the horizontal. For large installations commercially available tube models can have merit. An example is shown in figure 15.11. There are many other designs that may give an equally good settling efficiency.

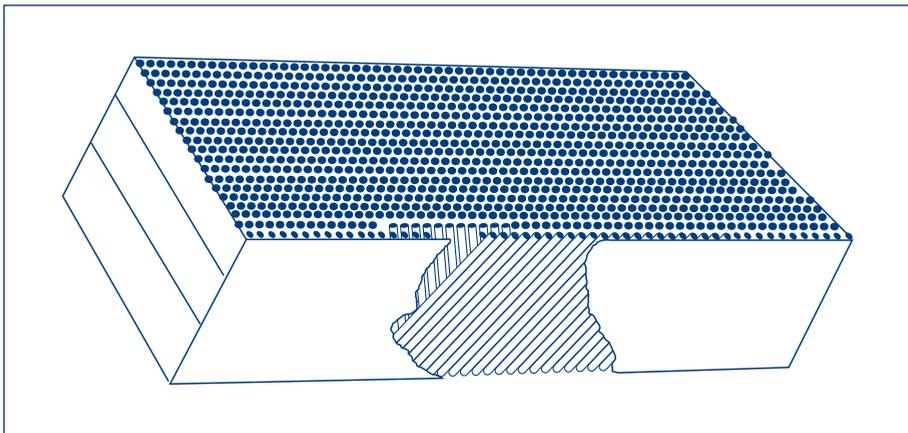


Fig. 15.11. Module for tube settler

In a 5 cm diameter tube, the farthest distance any particle must settle is from the top of the tube to the bottom. If the particle's settling rate is 2.5 cm/minute it will take only two minutes for the particle to reach the bottom. In contrast, if the same particle were to settle in a 3 m deep tank, it would take 120 minutes (2 hours) for it to fall to the tank bottom. Tube modules commonly are approximately 76 cm wide, 3 m long and 54 cm deep. Because the tubes are at an angle of 60° the effective tube length is 61 cm.

Tube modules can be constructed from flat sheets of ABS plastic with the passageways formed by slabs of PVC. The passageways are slanted in a criss-cross pattern for structural strength so that the module needs to be supported only at its ends. Being of plastic these modules can be easily trimmed to fit the available space in a settling tank.

The effective settling surface is very great and, thus, the “surface loading” (overflow rate) very low. To illustrate this: a flow rate of $2 \text{ m}^3/\text{hour}$ through a settling basin of 0.1 m^2 surface represents a surface loading of $20 \text{ m}^3/\text{m}^2.\text{hour}$. If twenty rows of tubes are used, the surface loading will be reduced to $1 \text{ m}^3/\text{m}^2.\text{hour}$. The detention time of the water in each tube will be just a few minutes.

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 could carry the increased loading.

15.4 Dissolved air flotation (DAF)

Introduction

Since the 1970's there has been a growing interest in flotation of particles from water as opposed to traditional forms of settlement. For more than a century the separation and collection of useful minerals by flotation with coarse air bubbles, called *dispersed air flotation*, had been usual in the mineral processing and metallurgical industries, but it could not be usefully applied to the fine and light particles found in water purification. Development of very fine bubbles from air in solution, called *dissolved air flotation* (DAF) made it possible to collect and remove fine light particles, such as flocs containing colour, or algae, from water intended for drinking water supply.

In most cases the fine particles and colour need a coagulation and flocculation step, with the addition of appropriate chemicals such as aluminium or ferric salts (see chapter 14). This immediately raises a problem for small communities, but if chemical dosing is already established or envisaged for processes requiring flocculation (floc blanket, sedimentation, direct filtration), then it may be useful to consider dissolved air flotation as an alternative process. In cases where significantly high algal concentrations have occurred, DAF has been applied to remove the algae without the need for prior chemical flocculation.

Basic technology

The basic technology of DAF is illustrated in figure 15.12. The main flow of water to be treated is from left to right and comprises chemical addition and mixing, flocculation, injection of water saturated with air under pressure, nozzles for the release of pressure,

a flotation tank with froth removal, and clarified water to rapid filtration. A recycle flow of clarified water has to be pumped to a saturator vessel, which contains an inert packing supplied with compressed air. This allows air to dissolve in the recycle water, which is then fed to the air release nozzles at the entrance to the flotation tank in the main flow.

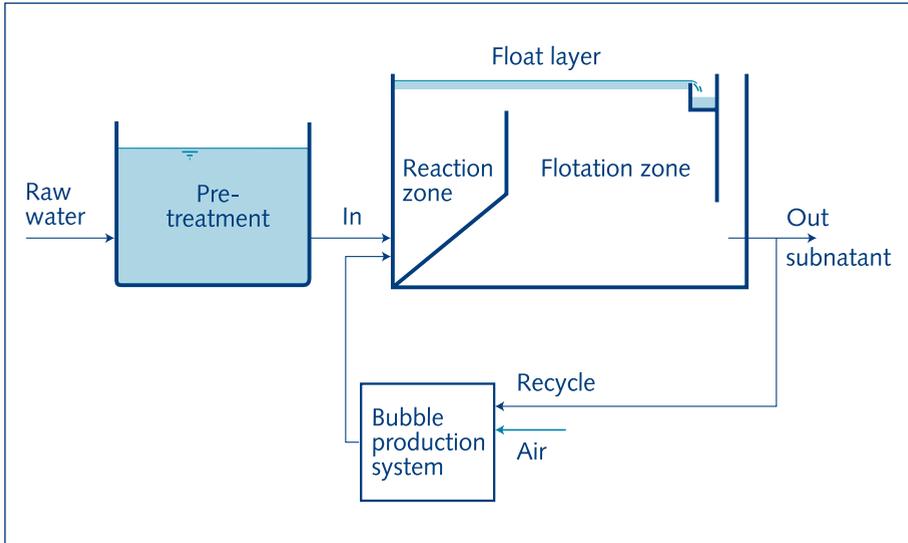


Fig. 15.12. Dissolved Air Flotation schematic diagram
Source: Van Duuren, FA., 1997

Characteristic values for the process (after coagulation-flocculation) are:

Air injection	708 air per m ³ of raw water
Operating pressure	350-420 kPa (3.5-4.2 bar)
Water recycle	7-8% of flow (limits 6-10% to avoid inadequate mixing, or excessive floc break-up)
Flotation tank loading	12m ³ /m ² .h (surface loading rate, as in settling tank design).

Application

Because there are several technical steps necessary in DAF, it would not normally be advisable to use the process in small community water supplies. The technical steps are: coagulation and flocculation, dissolved air pressure release nozzles, removal of froth sludge by mechanical surface scraping devices, recycle pumping, saturator vessel operation and air compressor. The air compressor can be simplified as a venturi tube eductor on the recycle pressure line, which will suck in air due to the pressure drop at the venturi throat, open to the atmosphere. However, if responsible maintenance (a qualified technician) is available, it is possible to install package plants, delivered as a whole unit, which will sit on a concrete pad, with appropriate inflow and outflow pipe, and sludge drainage connections. A reliable electricity supply is essential.

The principal application for DAF in small community water supplies is in the flotation of algae, particularly blue-green algae, which have a natural tendency to float anyway, but which can give rise to filtration problems if not reduced beforehand. DAF has the advantage that it responds quickly to switch-on, and does not suffer when not operating in idle periods (although it must be kept clean when not working). Consequently, if algal blooms start to occur in the source (raw) water, a DAF unit can be switched on to avoid trouble with the filtration stage during the period of algal infestation, and it can be switched off (and bypassed) when it is over.

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16 Multi-stage filtration technology

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16 Multi-stage filtration technology

16.1 Introduction

The technology of multi-stage filtration (MSF) presented in this chapter is a combination of coarse gravel filtration (CGF) and slow sand filtration (SSF). This combination allows the treatment of water with considerable levels of contamination, well above the levels that can be treated by SSF alone. MSF retains the advantages of SSF in that it is a robust and reliable treatment method that can be maintained by operators with low levels of formal education. It is much better suited than chemical water treatment to the conditions in rural communities and small and medium-sized municipalities in the South and in more remote areas in the North. Other treatment processes such as simple sedimentation, sand traps and screens can precede MSF technology. Wherever possible, terminal disinfection needs to be included as a safety barrier after the MSF. This chapter provides a summary description of the components of MSF systems. It gives an overview of indicative cost implications and ends with a selection guide.

16.2 Slow sand filtration technology

There are some typical operational differences between SSF and rapid filtration (RF) units. Filtration rates are around 50-150 times lower for SSF. Flow retention periods are about 30-90 times longer for SSF. Filter run lengths are about 30-90 times longer for SSF, and the surfaces of the SSF units are usually scraped at the end of the filter runs, whereas RF units are cleaned by backwashing. These differences originate from the most distinctive feature of SSF, its biological life. The water treatment in SSF is the result of a combination of physio-chemical and biological mechanisms that interact in a complex way.

Inorganic and organic matter enter the SSF units in the raw water. Photosynthesis gives rise to another fraction of organic matter. Soluble matter in the sand bed is utilised by bacteria and other micro-organisms. Zooplankton grazing occurs and respiration of the entire biomass is continuous.

The principal physical mechanisms contributing to particle removal are surface straining, interception, transport, and attachment and detachment mechanisms. Physical particle removal in SSF is not exactly the same as in RF since in RF the particles have previously been destabilised by chemical coagulants and the biological activity is not so relevant.

Design characteristics of slow sand filtration units

In an SSF treatment plant at least two units should operate in parallel for continuous supply. A unit basically consists of a structure that contains flow control and drainage systems, a supernatant water layer and a filter bed (Fig. 16.1).

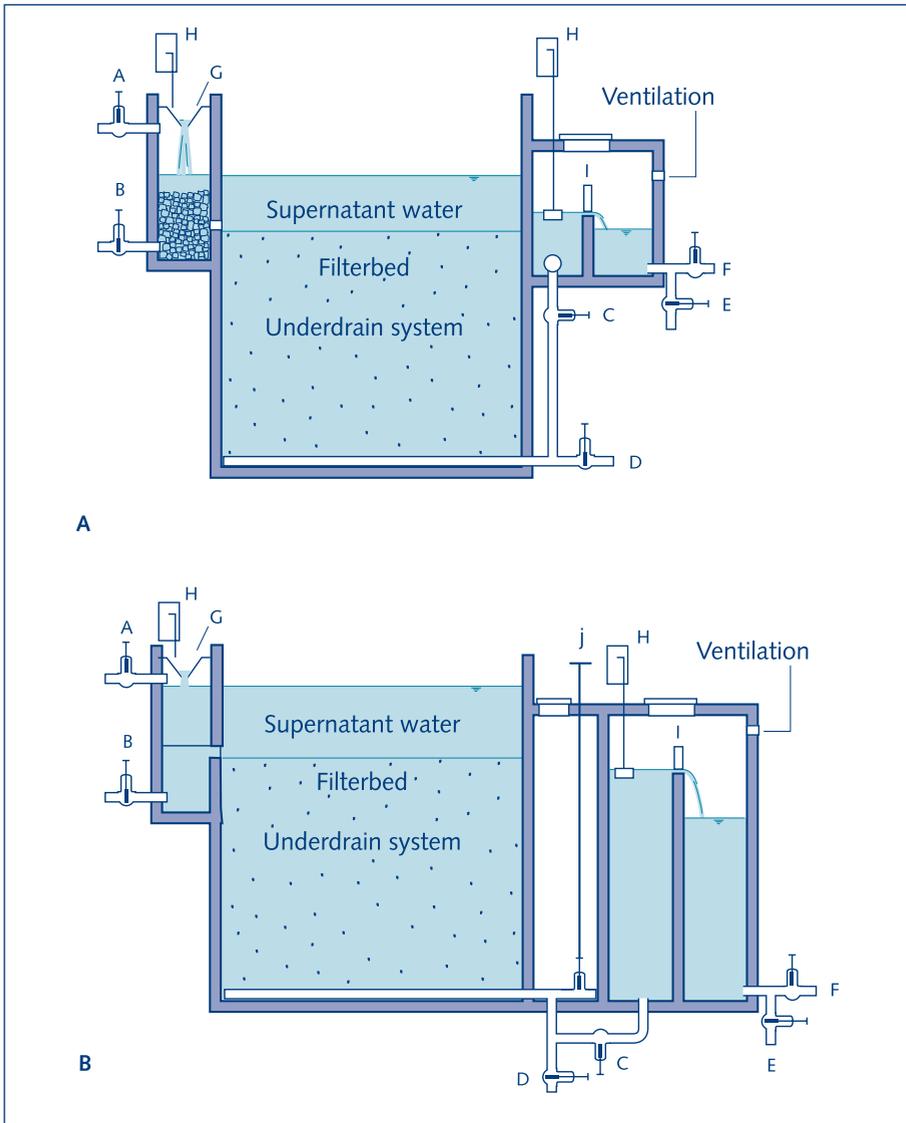


Fig. 16.1. Basic components of SSF units with inlet (A) and outlet (B) flow control

- | | |
|---|---|
| A: Inlet valve to regulate filtration rate | F: Valve to contact tank or water storage |
| B: Valve to drain the supernatant layer of water | G: Inlet weir |
| C: Valve for backfilling unit with filtered water | H: Calibrated flow indicator |
| D: Valve to drain the filter bed | I: Outlet weir |
| E: Valve to waste filtered water | J: Outlet control valve |

Flow control systems

Controlling the flow in SSF units is necessary to maintain the proper filtration rate through the filter bed and the submergence of the media under all conditions of operation. Abrupt filtration rate increases should be avoided. Two types of flow rate control are used, outlet- and inlet-controlled flow.

In an outlet-controlled filter the supernatant water level is kept close to the maximum desired level above the filter bed. To control the flow rate, the outlet valve is gradually opened to compensate for the increase in the head loss over the filter media. This is the usual control method in Europe and has been adopted in some of the units built in the Americas. The storage capacity above the sand bed provides for some equalisation of the influent water quality, sedimentation of heavier particles, and time for some biological activity, as well as some buffer capacity.

In inlet-controlled filters any increase in head loss is compensated by an increase in the height of the supernatant water. Some researchers have found similar performance in terms of effluent water quality, head loss in the filtering bed, and filter run times for inlet- and outlet-controlled SSF units run in parallel, with filtration velocities in the range of 0.13-0.5 m h^{-1} . In the inlet flow control option, the inlet box has several purposes. It provides flow control, reduces excess energy to protect the filtering bed from scouring, facilitates flow distribution to the SSF units filtering in parallel, and permits possible overflow.

The drainage system consists of a principal drain with lateral branches, usually constructed in perforated pipes, brickwork or tiles and covered with a layer of graded gravel and a layer of coarse sand. The drainage system of SSF has to achieve the following functions:

- Support the filter material and prevent it from being drained from the filter
- Ensure uniform abstraction of the water over the filter unit
- Allow for the backfilling of the filter and drive out possible air pockets

The main drain should discharge the filtered water freely at atmospheric pressure into the outlet box. A flow indicator is required at both inlet and outlet side of the units to facilitate operational procedures and to verify water balance, as an indication of possible water losses in the main filtering boxes. The outlet weir is also necessary to maintain the supernatant water layer above the maximum level of sand, protecting biological activity, preventing pressure drops in the filter bed, and ensuring the functioning of the units independently of the level fluctuations in the contact or storage tanks.

Supernatant water layer

The layer of supernatant water provides the static head necessary for the passage of water through the sand bed. In a clean bed the initial head loss is usually below 0.1 m and it gradually increases until the maximum level is reached. In units with outlet control, variations of the supernatant depth for small systems have been reported in the range of 0.6-1.2 m. At the Weesperkaspel plant in Amsterdam, where the SSF units deal with highly pre-treated water, the average supernatant water height is 2 m. Filter shading may contribute to improve filter runs if significant production of filter-blocking algae is occurring on the filter skin or in the supernatant water layer, but few definitive advantages in terms of filtrate quality have been reported.

Filter bed

The adequate selection of sand includes size grading, characterised by the effective size diameter d_{10} , and the uniformity coefficient, $uc = d_{60}/d_{10}$. Huisman and Wood (1974) advise that d_{10} should be small enough to produce safe water and to prevent penetration of clogging matter to such depth that it cannot be removed by surface scraping. Experiences in the USA report a total coliform removal reduction from 99.4% at d_{10} of 0.1 mm to 96% at d_{10} of 0.6 mm.

Deeper sand beds should result in improved removal of particles. However, due to the development of the filter skin and the biological activity concentrated mainly in the upper sand layers, particle removal is more effectively accomplished in this part of the SSF units. Experimental evidence supports the practice of having a minimum sand depth of 0.3-0.5 m in the SSF units to achieve more than one log reduction of indicator bacteria. This is relevant for small systems working with low flow rates ($0.1-0.2 \text{ m h}^{-1}$), but having to filter at higher rates during short periods due to their lower buffering capacity when one of the units is out of operation.

The sand to be put into the SSF units should be clean and free of clay, earth and organic material. The presence of dust or fine material produces high initial head losses and seems to limit the essential development of an active and effective microbial population in the filter bed. Placing dirty sand in the filter may interfere with the treatment process and makes it necessary to remove the sand earlier for correct washing.

Operation and maintenance procedures

SSF units must operate continuously, since this contributes to better quality effluents and a smaller filtration area is required for a given daily water production. Declining-rate filtration can be applied, but intermittent operation should be avoided, since oxygen depletion in the bed compromises biological activity. Research carried out in India reports deterioration of effluent bacteriological quality when filters recommence operation after 5 hours. In the USA, initial ripening periods in the range of 35-100 days were identified before the effluents of the SSF units became stabilised for parameters such as viruses, indicator bacteria and turbidity.

After several weeks or months of running, the SSF unit will gradually become clogged as a result of the accumulation of inorganic and organic material, including the biomass that is formed on top of the filter bed. The major increase in head loss occurs in this top layer. By scraping off this layer, the hydraulic conductivity is restored to the level at the beginning of the filter run. Classically, this is achieved by scraping the top 1-3 cm of the filtering bed. After several scrapings, when the filter bed reaches its minimum depth (0.3-0.5 m), resanding is required.

Manual cleaning has been the option for most small SSF units. In general, a high frequency of scraping is associated with one or more of the following factors: high solids concentration in the raw water; growth of algae in the supernatant water; small media grains; low available head; and high water temperature. The filter runs (periods between scrapings) of small SSF units in the USA range from one week to one year, with the average about 1.5 months. There, manual scraping labour requirements are in the range of 1.3-8 (average 4.2) person-hours per 100 m² of area scraped. The labour requirement increases significantly when the depth scraped is greater than about 2.5 cm. Filter runs for small systems could vary in the range of 20-60 days and, for cycles shorter than 1.5 months, labour costs will escalate and operator satisfaction with the plant will diminish.

After scraping the sand surface, a secondary ripening period may be necessary for the SSF units to recover their previous treatment capacity. Values in the range of 0-10 days have been recorded for this secondary period. The most important factor affecting the duration of a secondary ripening period appears not to be the removal of the filter skin, but the dewatering of the sand bed. The cleaning is best done in warm periods and by keeping the water table within 10 cm of the sand surface. This procedure ensures that spirotrichs and peritrichs protozoa are retained in the sand (inoculation); they are susceptible to desiccation and are unable to re-establish themselves at less than 3°C.

Scraped sand should be washed and stored. After several filter runs this activity leads to a gradual reduction of the sand bed depth until a minimum value, usually in the range of 0.3-0.5 m, is reached. Then re-sanding becomes necessary. For resanding, the remaining sand in the filtering bed should be lifted to become the top portion, with the stored and washed sand becoming the bottom. In this way the sand on top of the filtering bed should provide seed organisms to shorten the ripening period. Resanding in the USA requires around 50 person-hours per m².

The wet-harrow cleaning technique uses a horizontal and sometimes vertical pressurised water flow below the sand surface for washing across the filter skin being harrowed, without dewatering the sand beds. The wash water is passed out via a surface overflow weir. Shorter cleaning and ripening periods have been recorded with this technique in the USA, where it is applied in SSF units treating clear raw waters with low turbidity.

Design guidelines

Great differences exist in the application of SSF technology around the world, as it depends on drinking water quality standards, raw water quality, the type and level of pre-treatment specified; and the local conditions. These conditions include institutional development and support capacity to community-based organisations, availability of materials and financial resources, user income, and willingness to contribute to capital investment and running costs of the water supply infrastructure.

Design criteria presented by various authors and based on different experiences and conditions are summarised in table 16.1. Those recommended by Visscher et al. (1987), although oriented worldwide, were considered adequate for small systems in the USA, where the experience with SSF was being re-established. The last column in table 16.1 corresponds to the design criteria proposed by Cinara - IRC based on planning, design, monitoring and evaluation of over 100 SSF systems built in Colombia and other Latin American countries.

Table 16.1 Comparison of design criteria for slow sand filtration from various authors

Design criteria	Recommendation			
	Ten states standards USA (1987)	Huisman and Wood (1974)	Visscher, et al. (1987)	Cinara – IRC (1997)
Design period (years)	Not stated	Not stated	10 -15	8 - 12
Period of operation (hd ⁻¹)	24	24	24	24
Filtration rate (mh ⁻¹)	0.08 - 0.24	0.1 - 0.4	0.1 - 0.2	0.1 - 0.3
Sand bed: Initial height (m)	0.8	1.2	0.9	0.8
Minimum height (m)	Not stated	0.7	0.5	0.5
Effective size (mm)	0.30 - 0.45	0.15 - 0.35	0.15 - 0.30	0.15 - 0.3
Uniformity coefficient:	Not stated	< 3	< 5	< 4
Acceptable	≤ 2.5	< 2	< 3	< 2
Support bed. Height including drainage (m)	0.4 - 0.6	Not stated	0.3 - 0.5	0.25
Supernatant water. Maximum height (m)	0.9	1 - 1.5	1	0.75
Freeboard (m)	Not stated	0.2 - 0.3	0.1	0.1
Maximum surface area (m ²)	Not stated	Not stated	< 200	< 100

Water quality limitations of slow sand filtration

Slow sand filtration has been recognised as a simple, reliable and efficient treatment technology and a most effective unit treatment process in improving water quality. However, SSF does not necessarily remove all harmful substances to the extent required by relevant drinking water quality standards. Table 16.2 presents typical treatment efficiencies that SSF can achieve. The reported efficiencies have normally been achieved in filter units operated at filtration rates in the range of 0.04 and 0.20 mh⁻¹, temperature above 5°C, and sandbed depths greater than 0.5 m.

The efficiencies in table 16.2 cannot always be achieved though, because much depends on the nature, composition, and concentration of the components in the influent waters; and the effect of design parameters, and ambient and operating conditions. Even if high removal efficiencies can be obtained, SSF alone cannot always produce water of a high standard. Raw water sources in many countries are already so deteriorated that a combination of treatment processes is required to meet water treatment objectives or national drinking water standards

Clearly, SSF, like all other treatment processes, is not a panacea for every water quality problem. In general, two situations can be identified under which SSF presents limitations:

- Levels of contamination in the raw water may exceed the treatment capacity, or may result in short filter runs to comply with existing standards;
- Conditions that inhibit or reduce the efficiency of the treatment process.

Levels of contamination that exceed the treatment capacity

Suspended solids or turbidity. The most frequently mentioned limitation of SSF when it is used as a single treatment step is its inability to treat water with a high level of suspended solids or turbidity. These solids can create major increases in head loss and adverse conditions for the biomass active in the filtering bed. Even short peaks of solids may bury the large number of bacterial predators present in the sand bed and thus reduce their capacity to remove harmful micro-organisms. This important potential reduction in biological performance is, however, rarely cited in the technical literature, despite the fact that it may have a very negative effect on the quality of the treated water. The literature seems to focus instead on the difficulties of treating water sources with small particles of a colloidal nature or the impact of high concentrations of particulate matter on the duration of filter runs.

To prevent high effluent turbidity, frequent blockage of the filter bed (filter runs shorter than one month) or an environment that is unfavourable for microbiological activity, upper limits are usually specified for the influent turbidity. The limits vary, however, between < 5 NTU and < 50 NTU. Furthermore, the majority of the references accept higher values in the range of 50-120 NTU, provided these are of short duration, i.e. less than few hours to 1-2 days, though they recognise these high limits as undesirable. Nevertheless, turbidity alone is not sufficient to identify the limitations associated with the duration of filter runs.

Table 16.2 Treatment efficiencies of slow sand filters (Galvis et al., 1992a; Fox et al., 1994; Lambert and Graham, 1995)

Water quality parameter	Performance or removal capacity	Comments
Enteric bacteria	90-99.9%	Reduced by low temperatures; increased hydraulic rates; coarse and shallow sand beds; and decreased contaminant level
Enteric viruses	99-99.99%	At 20°C: 5 logs at 0.2 mh ⁻¹ and 3 logs at 0.4 mh ⁻¹ At 6°C: 3 logs at 0.2 mh ⁻¹ and 1 log at 0.4 mh ⁻¹
Giardia cysts	99-99.99%	High removal efficiencies, even directly after cleaning (removal of the filter skin)
Cryptosporidium	> 99.9%	Cryptosporidium oocytes. Pilot scale studies
Cercaria	100%	Virtually complete removal
Turbidity	< 1 NTU	The level of turbidity and the nature and distribution of particles affect treatment capacity
Pesticides	0-100%	Affected by the rate of biodegradation
DOC ¹	5-40%	Mean around 16%. Removal appears to be site specific and varies with raw water and O&M
UV-absorbance (254 nm)	5-35 %	A slight, but not significant difference in treating upland and lowland water sources. Mean 16-18% Colour associated with organic material and humic acids.
True colour	25-40%	Colour associated with organic material and humic acids. 30% being the average
UV-absorbance (400 nm)	15-80%	Colour (°Hazen). Mean 34%, but upland water sources 42% and lowland water sources 26%
TOC ² ; COD ³	< 15-25%	Total organic carbon; chemical oxygen demand
AOC	14-40%	Assimilable organic carbon. Mean about 26%.
BDOC	46-75%	Biodegradable dissolved organic carbon. Mean 60%
Iron, manganese	30-90%	Fe levels > 1 mg l ⁻¹ reduce the filter runs

1. DOC = dissolved organic carbon
2. TOC = total organic carbon
3. COD = chemical oxygen demand

Turbidity is accepted as an indirect indicator of the presence of particulate matter, because of its ease of application. This parameter does not always properly reflect the load of solids that the filter receives though, particularly if the particles are of an organic nature such as algae. In addition, very few recommendations exist about the maximum load of suspended solids (SS) an SSF can accept. Technical literature suggests a SS load below 5 mg/l but without evidence related to the impact of this level of SS on SSF units.

Iron and manganese. Bacteria that contribute to the oxidation of iron and manganese are present in the filter bed. Small quantities of iron deposits improve the removal capacity for organic components. On the other hand, high concentrations of iron (above 1 mg/l) may contribute significantly to the clogging of the SSF unit.

Algae. Algae may grow in rivers, lakes, storage reservoirs, or even in the supernatant of the SSF. The presence of algae in moderate quantities is usually beneficial for functioning of the SSF units. Most algae are retained by the SSF, but under certain conditions occasional and significant algal growth or algal blooms may develop. This massive growth can cause a quick reduction of the permeability of the filtering bed, greatly reducing the filter run. Algae may also play an important role in the production of high concentrations of soluble and biodegradable organic material in the water, which in turn create smell and taste problems, and contribute to microbial growth in the distribution system. Furthermore, as a result of photosynthesis, algae may affect the buffer capacity of the water and increase the pH to levels of 10 or 11. This can result in the precipitation of magnesium and calcium hydroxides in the sand bed (calcification) and contribute to the obstruction of the filter bed, increase the effective diameter of the sand, and reduce the efficiency of the process.

Controlling algae is difficult, but possible methods are based on reducing the nutrient content of the raw water, or creating a storage system or a supernatant environment in which algae can be controlled by the exclusion of light. This is done by covering the filters. Before investing in covers for the SSF, it is prudent to check if standard operation and maintenance procedures are not enough to manage moderate quantities of algae by occasional harvesting. Different levels have been established for the concentration of algae and other parameters (table 16.3).

Organic colour and organic carbon. A limitation of SSF is its low efficiency in the removal of organic colour and organic carbon. In fact, some studies report no removal at all and others indicate TOC and COD removal in the range of 15-19%. However, there are also studies reporting COD removals in the range of 50-68%. The discrepancy lies in the diverse composition of organic compounds, which are grouped together under surrogate parameters such as COD or TOC. SSF units generally remove between 5 and 40% of DOC, although the mean value is only 16%, and the difference between upland and lowland water sources is not significant (data from wide literature review).

True colour. True colour removal, as colour units of Pt-Co in filtered or centrifuged samples, includes only colloidal and soluble substances, especially natural organic matter. The removal of true colour is normally reported to be in the range of 25-30%. Because of the potential formation of disinfection by-products in the presence of organic material, low colour levels are desirable. The colour level, however, should not determine the application of final disinfection, as the risk of acute microbiological contamination is far more significant.

Heavy microbiological contamination. In some communities the only source available for water supply may be so heavily contaminated with harmful micro-organisms that SSF alone will not be able to produce a good quality effluent. Whilst long-term efforts are directed at protecting catchments, pre-treatment of the raw water may be necessary before SSF can be properly applied.

Conditions that inhibit or reduce the efficiency of the treatment process

Various circumstances can interfere with the treatment process in the SSF units and prevent the expected efficiencies from being obtained. Some of these are related to the short filter runs considered in the previous item. Other important inhibiting conditions are low temperatures, low nutrient content and low dissolved oxygen content.

Low temperature. Low temperature increases the viscosity of water and reduces the biochemical activity in the sand bed, affecting the treatment efficiency. E. coli removal may be reduced from 99 to 50% when the temperature falls from 20°C to 2°C. The strategy in countries that face cold periods during the year has been to cover the filters or to build them underground to prevent the freezing of the units and reduce the impact of low temperatures. This, of course, has considerable economic implications. Reducing the filtration rate is another way to reduce the impact of low temperature on the treatment process.

Nutrients. The micro-organisms active in the sand bed require nutrients such as carbon, nitrogen, phosphorus and sulphur for their metabolism and growth. Humic and fulvic acids are rich in carbon but low in the other elements. This may be part of the explanation for the low removal of natural colour in SSF treating water sources that are well protected. In experimental SSF units, adding nutrients has been shown to increase the biological activity and improve removal efficiency for turbidity and microbiological contamination.

Dissolved oxygen. When the flow velocities and the dissolved oxygen level in the water source are low, particularly if this is combined with a high amount of biodegradable material, the oxygen in the water can be depleted, resulting in anaerobic conditions in the filter skin. This anaerobic condition in the filter must be avoided because it may create serious water quality problems such as bad smell and taste, as well as

re-suspension of heavy metals, with aesthetic implications and interference with the final disinfection stage.

In summary, in spite of the potential of the SSF process illustrated in table 16.2, surface waters presenting relatively moderate to high levels of contamination could not be treated directly by conventional SSF units. Far too great a strain would be placed on the terminal disinfection, limiting its role as a final safety barrier. This is critical in most developing countries, where the reliability of disinfection is low.

Table 16.3 Some water quality guidelines that permit direct slow sand filtration treatment

Water quality parameters	Quality limitations based on references of 1991		
	Spencer, et al.	Cleasby	Di Bernardo
Turbidity (NTU) ⁽¹⁾	5 - 10	5	10
Algae (units/ml)	200 ⁽²⁾	5 µg l ⁻¹ ⁽³⁾	250
True colour (PCU)	15 – 25		5
Dissolved oxygen (mg l ⁻¹)	> 6		
Phosphate (PO ₄) (mg l ⁻¹)	30		
Ammonia (mg l ⁻¹)	3		
Total iron (mg l ⁻¹)	1	0.3	2.0
Manganese (mg l ⁻¹)		0.05	0.2
Faecal coliforms (CFU/100ml)			200

- (1) The type of turbidity and the particle distribution may produce changes in the water quality of the effluent of the SSF.
- (2) Both the number and the type of species present in the water source are important. This reference suggests covered filters.
- (3) This limit corresponds with chlorophyll-a in the supernatant water as an indirect measure for the algae content.

16.3 Overcoming the water quality limitations of slow sand filtration

Multi-stage and integrated water treatment concepts take advantage of the great potential of SSF technology. They have made it possible to overcome many of the water quality limitations previously identified and to meet drinking water quality requirements. In practice they are not new concepts as can be seen from the gradual evolution of water treatment in two important European cities.

London

By the beginning of the twentieth century, SSF was already accepted as a vital barrier in the provision of safe drinking water in London. A few years later, long-term storage reservoirs and terminal disinfection with chlorine were incorporated as additional

treatment steps. Each of these treatment stages was fundamental in contributing to improve drinking water quality. Nevertheless algal growth in the reservoirs and the increased load of suspended solids gradually created premature clogging problems in the SSF units. This problem was overcome in 1923 when the Metropolitan Water Board introduced its first "rapid" sand filter (without coagulants). This double filtration was used without major modifications until the 1980s. The gradual microbial improvement of each step in this four-stage treatment is illustrated in figure 16.2. In the 1990s, to comply with the requirements of the European Community, the treatment plants were improved by including ozone treatment and a layer of activated carbon in the filter bed to increase the biodegradability and the removal of organic compounds and improve the reliability of disinfection.

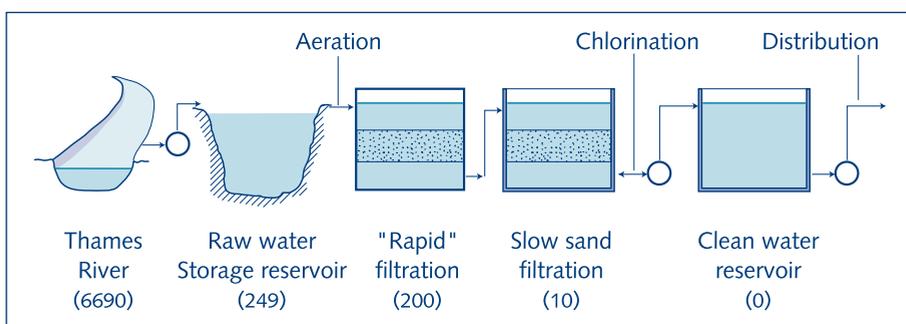


Fig. 16.2. Gradual removal of microbes indicating pollution (*Escherichia coli*) from a half pint glass (284 ml) of water at each stage of a typical London water treatment plant, based on a 10-year (1961- 1970) average. (Adapted from Windle-Taylor, 1974)

Zurich

The city of Zurich draws its water from three sources: Lake Zurich, groundwater and springs. The first treatment plant with SSF began operation in 1871. Gradually other processes were added due to water quality deterioration and higher water quality standards setting lower acceptable levels of organic contamination. Today lakewater provides 70% of the water supply and is treated in two water treatment plants. In 1975, SSF became the seventh of an eight-stage treatment system comprising: pre-oxidation in the lake water collectors, coagulation/flocculation, pH adjustment, rapid sand filtration, ozone treatment, activated carbon filtration, SSF, and disinfection (Fig. 16.3). Velocities up to 0.7 mh^{-1} are now applied in the SSF. One of the benefits of SSF in this treatment plant is to contribute to removal of the organic compounds that support biofilm growth in the distribution system, reducing the requirements for high levels of disinfectant residuals.

So, SSF continues to be used as a treatment process in large European cities, but today it is one of the final treatment stages, after quite complex pre-treatment stages. As a result the SSF units receive water of very good quality. Hence, these systems, with reliable operation, maintenance, and management conditions, can operate at high filtration rates of around 0.3-0.7 mh^{-1} . In these European cities the multiple barriers

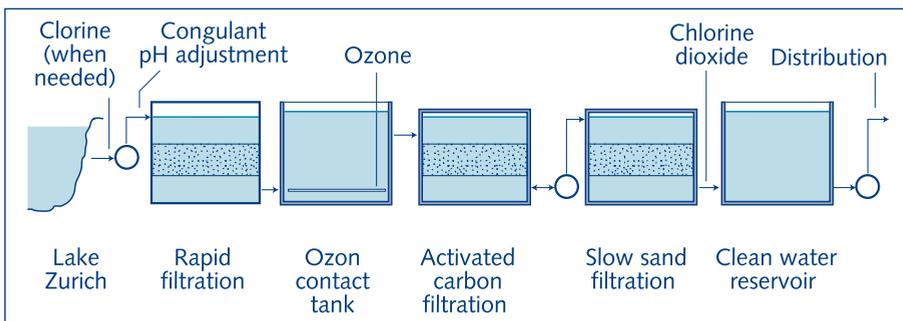


Fig. 16.3. Flow diagram of the water treatment system in Leng, Zurich (Adapted from Huck, 1988)

strategy and basic water treatment concepts gradually developed from field experience and under the pressure of tighter regulations. To extend the possibility of surface water treatment to rural areas and small towns, these same concepts can be used in identifying, developing and promoting pre-treatment alternatives in harmony with the simplicity of operation and maintenance of SSF.

The search for pre-treatment alternatives for small water supply systems

The adequate use of SSF technology in small systems has often been determined by the availability of good quality water resources, as is apparent from the application of SSF in the USA. Pre-treatment appears to be the *technical* link missing from the SSF technology for small communities with lower raw water qualities. During the last few decades pre-treatment alternatives have been developed to extend the application of SSF to poorer water sources without requiring skilled staff, complex mechanical equipment, or chemical supplies. Some of these methods, such as riverbank filtration (infiltration wells) and riverbed filtration (infiltration galleries), are oriented towards improving surface water quality at the abstraction point. Other methods, using plain sedimentation, are long- and short-term storage, and tilted plate settling. Others are based on coarse filtration, such as dynamic filtration, and horizontal flow, downflow, and upflow gravel filtration.

Infiltration wells

One of the oldest pre-treatment techniques is filtration in infiltration wells or riverbanks along a river or stream (Fig. 16.4). Depending on the surface water quality and the abstraction soil strata, the abstracted water may be acceptable for direct human consumption or to be feed water for SSF units. Experiences with river Rhine water showed that riverbank filtration reduced turbidity from a range of 1-6 NTU to a range of 0.2-0.8 NTU. Trace metals, DOC and COD were also significantly reduced. However some problems were reported with the re-suspension of iron and manganese oxides when the oxygen level in the river fell below 1 mg^l. Changes in sediment transport in the river may also affect the capacity of the wells. One possible disadvantage of this system is that changes may occur underground, and can be difficult to remedy by maintenance activities.

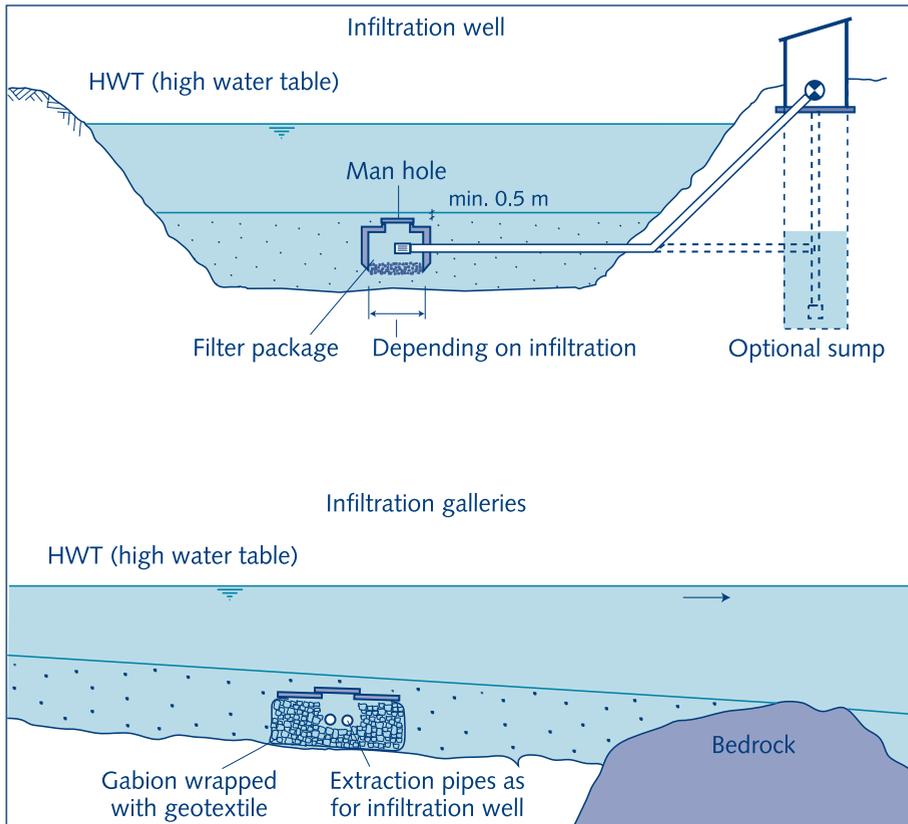


Fig. 16.4. Infiltration wells and infiltration galleries
 Source: Wehrle, personal communication; and Galvis and Visscher, 1987

Infiltration galleries

In infiltration galleries or riverbed filtration, water is abstracted using perforated pipes through the natural riverbed material, or if the permeability is too low, through an artificial bed of coarse sand and gravel. Riverbed filtration systems include longitudinal and lateral drain systems, modular sub-sand abstraction, and river dam filtration systems. Flow velocity through the filtering bed ranges from $0.25\text{--}1.5\text{ mh}^{-1}$, depending on turbidity levels and effluent requirements. Removal efficiencies up to 98% have been reported for riverbed dam filtration from rivers with turbidity levels in the range of 48–200 NTU. However, in a field evaluation the efficiencies were found to be around 20%. This may be due to difficulties in implementing periodic cleaning or repositioning of the clogged filtering material, particularly during the rainy season, when the rivers have high flows and high solids transport capacity. Because clogging of the infiltration area can mean reconstruction of the riverbed filter or the infiltration area, pre-treatment filtration alternatives completely separated from the surface water source are receiving more attention.

Plain sedimentation

Exposing the water to very slow or non-moving conditions allows suspended matter to be removed by the action of gravity and natural particle aggregation without the use of coagulants. This process is called plain sedimentation. Ideally, the clarification efficiency of a settling basin, for a particular suspension of discrete particles, depends only on S_0 , the surface charge (relation between the flow and the settling surface area). In practice, however, disturbing factors such as turbulence and short-circuiting reduce the effective settling velocity. Plain sedimentation is described in detail in chapter 15.

Tilted plate settlers

Improved flow conditions in the settling zone (laminar and stable flow) and lower values of S_0 (greater surface area for a given flow) can be obtained in a given conventional sedimentation tank by introducing parallel plates set a short distance apart (5-10 cm). To achieve self-cleaning, these plates are tilted or inclined at an angle of 50-60° to the horizontal. Tilted plate settlers may reduce the required area of a conventional settler (without plates) by some 65%. They are widely used in chemical water treatment, but their application with non-coagulated water is very limited. Besides, in small systems, if area is not a critical issue, this option may have comparable capital costs to conventional settling, but higher running costs, since more frequent attention and cleaning is needed because of its lower sludge storage capacity. Tilted plate settlers are also described in chapter 15.

Prolonged storage basins

Plain sedimentation may have long retention times, measured in days or weeks. In this case other factors are important, including wind, thermal, and photosynthetic effects. This usually makes it an expensive solution to be adopted exclusively for water supply purposes in small systems. A classical goal of storage basins is to provide supplies during periods of low rainfall in multipurpose projects, and off-channel storage can provide a source during short-term pollution events. Storage basins can be used as preliminary treatment. Indeed, for extremely turbid waters, above 1000 NTU, storage provides the best pre-treatment. In England the water depth in pumped storage reservoirs is typically about 10-20 m and the theoretical retention time ranges from about 10-50 days. In London, in long-term storage prior to SSF, turbidity reductions from around 30 NTU to below 4 NTU have been reported. As shown in figure 16.2, the average *E. coli* faecal coliform counts were reduced by 96%. However, the periodic blooms of algae made it necessary to introduce microstrainers or rapid filters before the SSF units. Management techniques have been developed to minimise algal blooms and other detrimental water quality effects in the reservoirs. These techniques include pumping devices to control the thermal stratification. The potential of long-term storage to protect SSF in small systems directly or in combination with other treatment steps needs to be evaluated under local conditions, introducing the possibility of a multipurpose reservoir.

Coarse media filtration (CMF)

Porous media such as gravel and sand are old water clarification processes with documented applications in several European countries since the 1800s. Development and promotion of this technology was interrupted with the arrival of chemical and mechanised water treatment technologies. Since the 1970s the use of SSF technology in small WS systems has gained increasing attention because of the potential of CMF to improve the quality of deteriorating surface waters. During the 1980s it became clear that CMF was a good option to condition the water before it reached the SSF units, based on studies conducted in Africa, Asia, Europe, and Latin America. These technologies and new ones are still being developed.

Coarse media filtration as a pre-treatment step for slow sand filtration

Short-term plain sedimentation may be the first conditioning stage of surface waters that transport relatively large and heavy particles, such as grit or sand. However, rivers usually transport a wide range of particles, including those with sizes of less than 10-20 μm . Most streams or small rivers in the tropics have peaks in suspended solids for short durations, giving a high load on the water treatment system; these can happen in the absence of the water treatment plant caretaker.

CMF is considered to be a promising pre-treatment technique for small water supply systems since it is more effective in removing suspended particles than short-term plain sedimentation and because of its ability to maintain treatment simplicity comparable to that of SSF. CMF units are easier to operate and maintain than long-term storage reservoirs and are not dependent on the hydraulic behaviour of streams or rivers, as are riverbank and riverbed filtration, particularly during the rainy seasons in tropical countries.

Classification of coarse gravel filters

Different CMF alternatives using gravel as the filter media are described in the following sections and schematically illustrated in figure 16.5. CMF alternatives have been classified according to the main application purpose and the flow direction as shown in figure 16.5.

Dynamic gravel filters (DyGF)

Dynamic gravel filters include a shallow layer of fine gravel in their upper part and coarse gravel that covers the underdrains. The water enters the unit and passes through the fine gravel to the drainage system. With moderate levels of suspended solids in the source water, the DyGF gradually clogs. If quick changes in water quality occur, the clogging may be much faster. Eventually the gravel bed will be blocked and the total water volume will just flow over the clogged surface area to waste, protecting the subsequent treatment steps that are more difficult to maintain.

Depending on the flow direction in the layer of gravel, the second treatment step – the gravel filters – are called upflow (UGF), downflow (DGF) or horizontal flow (HGF) systems. A comparative study of these alternatives showed that the option of UGF was technically and economically preferable over the DGF and HGF, although these also achieve good removal efficiencies.

Upflow gravel filters (UGF)

Upflow gravel filters consist in principle of a compartment in which the gravel layer reduces in size in the direction of flow. A drainage system placed on the bottom of the structure serves to distribute the flow during the filtration period or to drain the gravel layers during periods of cleaning, discharging the water through the drainage system. There are two alternatives: upflow gravel filters in layers (UGFL) when the gravel layers of different size are installed in the same unit and upflow gravel filters in series (UGFS) when the gravel layers are installed in two or three different units, each having a main gravel size that decreases in the direction of flow.

Downflow gravel filters in series (DGFS)

Downflow gravel filters (as used Colombia) consist usually of three compartments with the coarsest gravel in the first unit and less coarse in subsequent units. The functioning or performance of the DGFS is similar to the UGFS in terms of removal efficiency, but maintenance is more difficult because the sludge tends to accumulate on the surface of the first unit. Cleaning is more difficult than for the UGFS units, where the sludge is accumulated basically in the bottom part close to the drains.

Horizontal-flow gravel filters (HGF)

Horizontal-flow gravel filters consist of at least two parallel modules constructed basically in three compartments separated by perforated walls. In the beginning this option was very voluminous, because it did not include a drainage system for hydraulic cleaning. Nowadays a drainage system is included. Although it is possible to reduce the size of the units, the activities of operation and maintenance in an HGF are more demanding in terms of manpower and water consumption. Research on HGF in series gives promising results in terms of hydraulic performance and the gain is a substantial reduction in the length of the gravel bed, while maintaining similar efficiency levels as the conventional HGF.

General considerations

Effluent water quality

Coarse gravel filters (CGF) have normally been specified to produce an effluent with turbidity $< 10\text{-}20$ NTU, or suspended solids < 5 mg/l, although the impact of these or other values on the SSF performance or maintenance is not clearly established. Besides, other parameters such as high levels of faecal contamination or natural organic matter

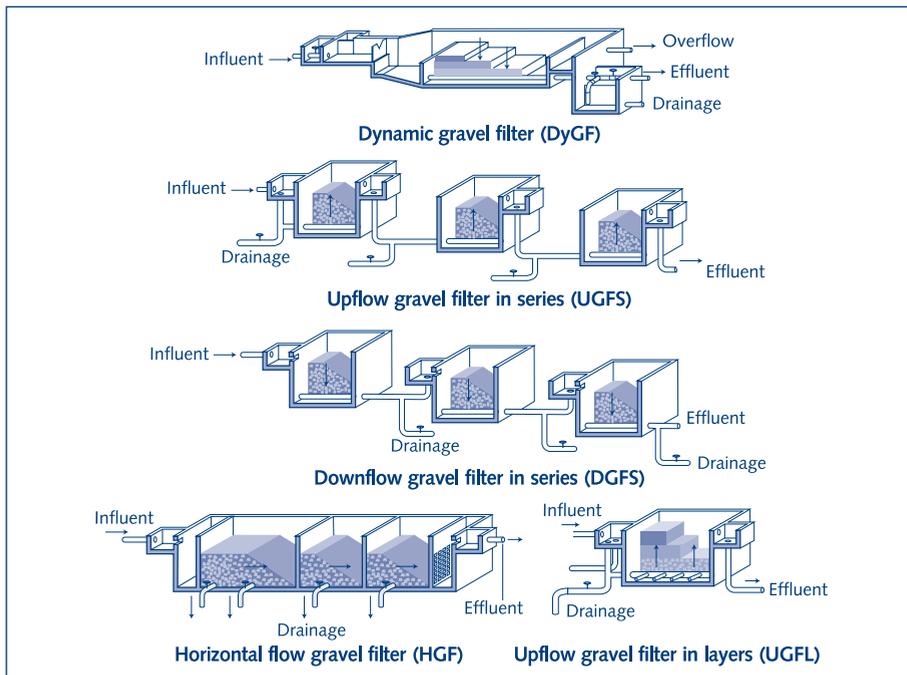


Fig. 16.5. Schematic view of coarse gravel filtration alternatives (Based on Galvis and Visscher, 1987)

that could limit the application of SSF, are not normally considered as critical factors in the specification of this technology.

Head loss and flow control

Final head loss in CMF units is small, usually a few centimetres, with a maximum value around 0.30 m. Because of these low values, CMF units usually have inlet flow control. The inlet structure should include facilities for energy dissipation, flow control, flow measurement, and overflow. A well-designed inlet box facilitates the operation and control of the system. A weir or a raised effluent pipe maintains the water above the filter bed level. Flow measurement devices are recommended at the inlet and outlet sides to control the operation and to verify that the filter boxes are watertight. Since the CMF units in small water supply systems deal with low flow and low pressure values, some simplified valves, gates, and weirs can be used together with more commercial hydraulic devices.

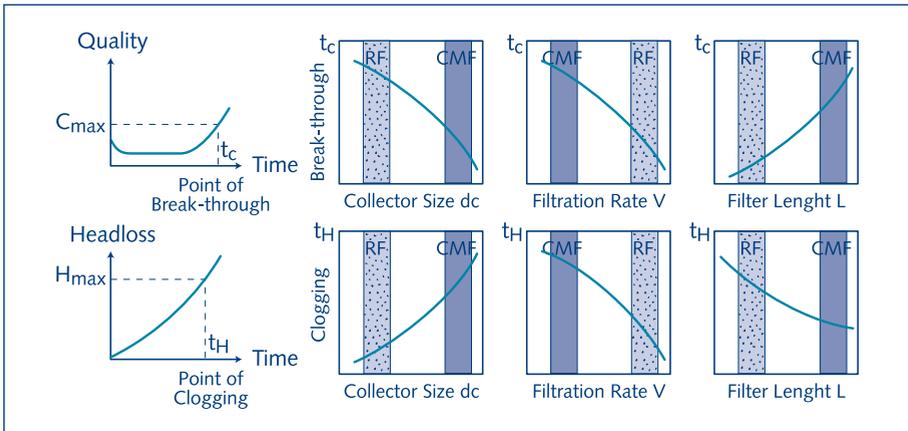
Design criteria and filter run time

The main criteria for CMF design have been removal efficiency and head loss related to particle retention in the filtering bed. Process variables such as particle nature and size distribution, collector size (d_c), filtration rate (v), and filter length (L) determine the filter run time up to the breakthrough, related to a maximum concentration value (C_m) in the effluent, or up to a clogging point, related to a maximum head loss (H_m).

A qualitative illustration of the impact of some process variables on breakthrough and clogging in RF and CMF is illustrated in figure 16.6. As predicted by the trajectory approach in filtration theory, removal efficiencies in coarse filtration will be smaller due to its greater collector size. This limitation is partly overcome by lower filtration rates and longer filtering beds in CMF.

Biological activity

Biological activity takes place in the coarse filtration units when they are processing natural waters and synthetic waters with organic matter or nutrients. Most probably, with mechanisms similar to those present for SSF, bacteria and other micro-organisms may form sticky layers in some areas of the filter media or produce exocellular polymers that contribute to particle destabilisation and attachment. Macro-biological creatures inhabiting the coarse filters are thought to contribute to the sloughing off of stored material or biofilm observed. There is evidence of organic matter decomposition during cleaning procedures of full scale HGF, calling for frequent maintenance of units susceptible to high biological activity.



Note: Although effluent water quality is expected to be lower in the CMF, its t_c (breakthrough) value should be higher than for the RF

Fig. 16.6. Effects of some process variables on the breakthrough and clogging points in rapid (RF) and coarse media filters (CMF). (Adapted from Boller, 1993)

Flow conditions and coarse media filtration efficiency

Research was carried out with vertical flow filter columns of 1 m depth filled with gravel varying from 1-64 mm in size and filtration rates from 0.5-8 mh^{-1} . The turbidity of the raw water mixture was maintained at around 60 NTU. Good turbidity reductions were obtained at filtration rates $< 2 \text{mh}^{-1}$. This experience shows that significant solids removal efficiency is only achieved under laminar flow conditions (see fig. 16.7).

Further laboratory and field tests with UGF and HGF confirmed that effluents with a turbidity below 10 NTU were achieved only at filtration rates of 0.5-1.0 mh^{-1} .

The filtering media

The filtering media should have a large surface area to enhance particle removal and a high porosity to allow the accumulation of the separated solids. Filtration tests with kaoline clay suspensions revealed that neither the roughness nor the shape of the filter material had a great influence on filter efficiency. Any inert, clean and insoluble material meeting the previous criteria could be used as filtering media. Gravel is the commonly used material, but broken bricks, palm fibre, and plastic material have also been reported in different experiences. In a review of CMF performance with different filter media, a filter filled with palm fibre achieved better turbidity removal than a gravel filter. This is the result of the greater porosity (92% versus 37%), resulting in a lower effective velocity. However, since the use of palm fibre causes a considerable drop of dissolved oxygen along with odour and taste problems, this filtering medium has serious limitations. The use of plastic material may be an alternative, but the uplift forces of the water have to be overcome.

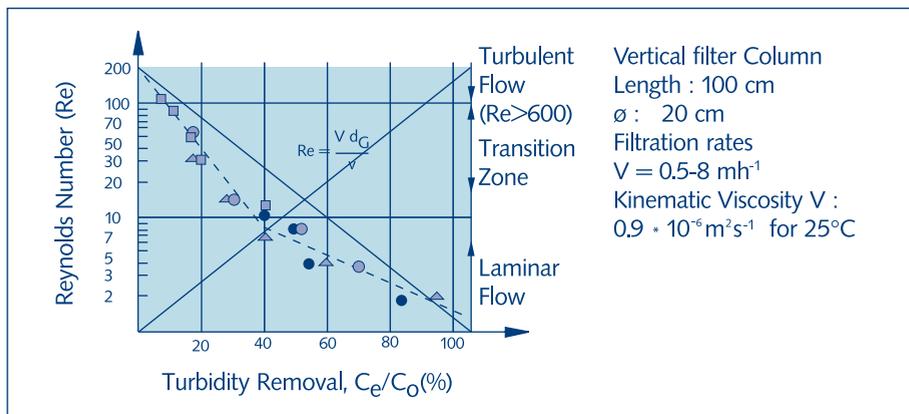


Fig. 16.7. Influence of flow conditions on coarse filtration efficiency
 Source: Wegelin and Mbwette (1989); Wegelin et al., (1991)

Operation and maintenance (O & M).

Operation of CMF units requires a frequent (at least daily) control of the influent and effluent flow and the quality of the raw and filtered water. Maintenance is associated mainly with the cleaning process, which tries to restore the initial head loss. To facilitate maintenance, a minimum of two units should be constructed in parallel. Frequent cleaning of the CMF units is recommended to limit head loss development and to avoid operational or maintenance difficulties due to solids consolidation or organic decomposition inside the filter media. CMF units are cleaned both manually and hydraulically. Manual cleaning involves media removal, washing and replacement, which is time consuming and labour intensive. So, hydraulic cleaning facilities for in-place

media flushing become a key component of the units to ensure a long-term sustainability of this treatment technology.

Initially only surface raking was used to clean dynamic gravel filters (DyGF). Later it was combined with filter bed drainage. Only manual cleaning was initially used to clean HGF and gradually fast drainage of the filter bed compartments has been incorporated in its application. Fast or moderate drainage velocities, combined in some cases with some surface raking, are being applied to maintain DGF and UGF. The area and the height of the filter boxes should be limited to facilitate both frequent hydraulic cleaning and eventual manual cleaning.

The drainage system

In the case of DyGF, HGF and DGF, the drainage system collects and provides an outlet for filtered water during normal operation, as well as for washing water during hydraulic cleaning by fast drainage. In the case of UGF, the drainage system distributes the water to be filtered, and collects and provides an outlet for washing water during hydraulic cleaning. The system may consist of a small trough, a false filter bottom, or perforated pipes or manifolds. One small trough would have limitations to produce an even flow distribution across the entire filter bed compartment. A good false bottom would ensure an even water collection or distribution but imply additional hydraulic structures. A properly designed manifold should have a good hydraulic efficiency with lower construction costs, although it requires an additional gravel layer to embed the pipes. The decision between false bottom and manifolds should be taken after analysing local conditions.

16.4 Considerations about multi-stage filtration

The combination of coarse gravel filtration and SSF is what in this publication is called multi-stage filtration (MSF).

The MSF technology has received a lot of positive response in Colombia and other Latin American countries, where over 100 systems are already in operation today. Ten of these built in Colombia date from the middle 1980s, each producing effluents with low sanitary risk before terminal disinfection and with low operation and maintenance costs that are to a large extent covered by the users. They pay a tariff of some 2 USD/month, in a country with a minimum official salary of some 140 USD. All systems are administered by community-based organisations with some technical support from sector institutions.

MSF does not compromise the advantages of an SSF system in terms of ease of operation and maintenance and the production of good water quality. It is an option that is applicable to many rural communities and small- and medium-sized municipalities, where treatment with chemical products has very little potential.

Table 16.4 presents a summary of the considerations concerning MSF treatment and figure 16.8 shows a layout of MSF with three components, DyGF, UGF and SSF.

The following combinations of CGF and SSF can be made:

DyGF + SSF

DyGF + UGFL + SSF

DyGF + UGFS2 + SSF

DyGF + UGFS3 + SSF

The criteria for selection of each combination will be discussed in chapter 16.6.

Table 16.4 Summary of considerations concerning MSF treatment

Issue	Comment concerning MSF treatment
Quality of treated water	It is a good alternative to improve the physical, chemical and bacteriological quality of the water. In many areas and particularly those with a less developed infrastructure, MSF is the only feasible treatment option.
Ease of construction	The relatively simple design facilitates the use of local materials and local manpower. There is no need for special equipment.
Construction cost	The construction in local materials and with local labour reduces the cost. Usually there is no need for imported materials.
Ease of operation and maintenance	After a short period of training, local operators with a minimum of formal education can operate and maintain the system.
Cost of operation and maintenance	The cost of operation and maintenance and the requirements in electrical energy are minimum and less than required for other systems. There is no need for chemical products for coagulation.
Reliability	A low risk of mechanical problems or problems related to the changes in the raw water quality, as these can be absorbed without interrupting the service in the majority of cases.
Cleaning	The cleaning process is simple although laborious, but almost always involving low cost, as in many countries labour is relatively cheap.
Requirements of surface area	A conventional RSF plant in respect to storage zones, management of chemicals etc., may require comparable areas to an MSF system.
It is not a panacea	There are levels of contamination that limit the efficiency or interfere with the treatment.

Performance of multi-stage filtration systems

The number of full-scale MSF plants in the world is limited. Most comprehensive research on functioning and performance has been carried out in Latin America. The following observations on performance are therefore mainly from that region. In general, performance findings are very satisfactory. Nevertheless, the performance may be different, that is higher or lower, in other regions of the world. Much depends on the characteristics of the raw water in terms of turbidity, suspended solids, particle size distribution, true colour and temperature. Climatic seasonal fluctuations also influence the performance of MSF.

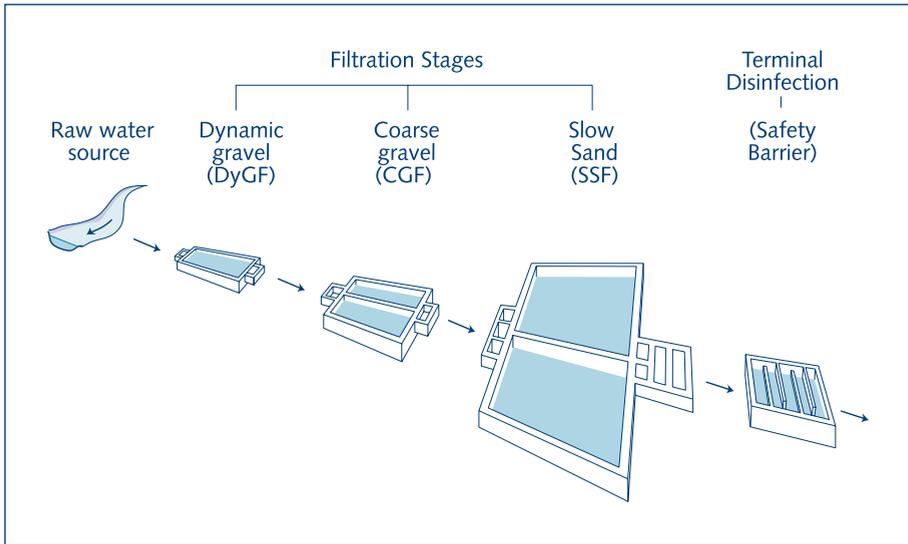


Fig. 16.8. Components of multi-stage filtration systems

The characteristics of the different coarse gravel media filter units are presented in table 16.5. The gravel filters were evaluated for filtration rates of 0.30, 0.45, 0.6, 0.7 and 1.0 m/h. The research included physical, chemical and bacteriological parameters and established the limits under which each unit could still operate. The last step in each treatment line was an SSF operated at 0.15 m/h.

The removal efficiencies of basic water quality parameters in MSF pilot system at Puerto Mallarino are shown in tables 16.6 and 16.7. Four different periods were evaluated, each with a different filtration rate.

The HGF, having a larger sludge storage capacity and similar removal efficiencies, may be an alternative for surface water high in suspended solids, even though it is more expensive.

Table 16.5 Characteristics of the treatment units in Puerto Mallarino Research Station, Cali, Colombia

Treatment Unit	Number of units in series	Filtration area (m ²)	Filter medium	
			Size (mm)	Length (m)
DyGF	1	0.75	6-25	0.6
URGS3	3	3.14	25-1.6	4.55
HGF	1	1.54	25-1.6	7.2
HGFS	3	1.54	25-1.6	4.55
UGFS2	2	3.14	25-1.6	3.1
UGFL	1	3.14	25-1.6	1.55
SSF	1	3.14	Uc = 1.57 D ₁₀ = 0.23 mm	max.: 1.0 min.: 0.6

Uc: Uniformity coefficient.
D10: Effective diameter

The results show that the combination of two stage gravel filters (DyGF + CGF) very much improves the performance of the SSF. Nevertheless, in cases of highly contaminated surface water sources, particularly if the levels of suspended solids are high (above 100 mg/l), a very critical selection of treatment barriers is required and has to be in coherence with the risk level in the water source and its variation over time. The preferred option would be to select an alternative water source. If this is not possible detailed pilot studies are needed to ensure the viability of the solution.

Performance of full-scale MSF systems in Colombia

Seven community managed MSF systems in the Cauca Valley have been monitored for a period of seven years. The systems receive water from catchment areas with low or moderate levels of human intervention. Water quality of the different sources indicates mean turbidity levels between 0.9 and 15 NTU, faecal coliform counts between 52 and 51,916 FCU/100 ml, and true colour levels between 3 and 30 PCU. The mean removal efficiencies of basic water quality parameters in full-scale MSF plants are shown in table 16.6 and 16.7. The wide ranges are due to different CMFs applied and the different local operations.

The composition of the systems matches the multi-barrier concept that implies that more than one stage of treatment is needed, combined in such a way that together the barriers have a removal efficiency that is sufficient to ensure low dose disinfection as the final and efficient safety barrier.

All systems produce water with turbidity below 1 NTU, with a frequency between 65 and 98%, and below 5 NTU in more than 98% of the samples. Faecal coliforms were below 25 FCU/100 ml, with a frequency above 97%, and true colour below 15 TCU in more than 98% of the samples. With these water qualities, constant dose disinfection with chlorine as suggested by WHO (1996) becomes an effective safety barrier.

Table 16.6 Individual (at each treatment stage) and cumulative (up to the end of SSF stage) mean removal efficiencies of basic water quality parameters in MSF pilot system at Puerto Mallarino.

Filtration		Period	Influent mean values			Individual mean efficiencies		
Stage	Rates (mh ⁻¹)		Turbidity (NTU)	Colour (PCU)	Faecal coliforms (CFU/100 ml)	Turbidity (%)	Colour (%)	Faecal coliforms (log. units)
DyGF stage								
DyGF	(0.9-1.4)	I	109	81	41,184	32	11	0.2
	(1.4-2.5)	II	59	54	31,800	41	15	0.6
	(1.4-2.5)	III	51	35	97,779	43	14	0.8
	(1.9-2.8)	IV	52	57	108,796	40	16	0.6
CGF stage								
UGFS	0.3	I	74	72	24,758	84	69	2.6
	0.45	II	35	46	8843	77	54	2.3
	0.6	III	29	30	16,823	77	53	2.4
	0.75	IV	31	48	26,226	75	63	2.3
UGFL	0.3	I	74	72	24,758	70	44	1.8
	0.45	II	35	46	8843	54	28	1.3
	0.6	III	29	30	16,823	55	30	1.4
	0.75	IV	31	48	26,226	61	46	1.3
SSF stage								
SSF 1	0.1	I	12	22	65	64	73	2.8
	0.1	II	8.1	21	45	75	67	2.7
	0.55	III	6.6	14	64	82	57	2.1
	0.15	IV	7.8	18	127	74	67	1.8
SSF 2	0.1	I	22	40	369	85	88	3.1
	0.1	II	16	33	452	79	70	2.2
	0.15	III	13	21	637	89	67	2.6
	0.15	IV	12	26	1226	77	69	2.0
Treatment lines		Period	Effluent mean values			Cumulative mean efficiencies		
DyGF + UGFS + SSF1		I	4.3	6	0.1	96	93	5.6
		II	2.0	7	0.1	97	87	5.5
		III	1.2	6	0.5	98	83	5.3
		IV	2.0	6	2.2	96	89	4.7
DyGF + UGFL + SSF2		I	3.2	5	0.8	97	94	5.1
		II	3.3	10	2.7	94	81	4.1
		III	1.4	7	1.7	97	80	4.8
		IV	2.8	8	10.7	95	86	4.0

Table 16.7 Individual (at each treatment stage) and cumulative (up to the SSF stage) mean removal efficiencies of basic water quality parameters in full-scale MSF plants.

Filtration		Filter bed length (m)	Influent mean values			Individual mean efficiencies		
Stage	Rates (mh ⁻¹)		Turbidity (NTU)	Colour (PCU)	Faecal coliforms (CFU/100 ml)	Turbidity (%)	Colour (%)	Faecal coliforms (log.units)
DyGF stage								
DyGF	0.9 – 1.6	0.3 – 0.6	3.8 – 24	15 – 30	2895 – 51,916	21 – 57	10 – 24	0.2 – 0.7
CGF Stage								
CGF	0.5 – 0.9	0.9 – 4.0	2.8 – 17	5 – 27	330 – 10,063	30 – 71	17 – 41	0.7 – 1.0
SSF Stage								
SSF	0.08 – 0.17	1.0 – 1.2	0.8 – 4.9	4 – 16	52 – 2008	50 – 87	25 – 75	1.7 – 3.3
Treatment plant			Effluent mean values			Cumulative mean efficiencies		
DyGF + CGF + SSF			0.4 – 0.9	3 – 6	0.7 -	79 – 96	40 – 87	2.6 – 4.7

MSF treatment can adapt itself to the type of raw water and the concentration of contamination. The systems give higher removal efficiencies for water that is higher in contamination. This implies that the barriers become more effective if the water to be treated has a higher risk and still can produce a water with a low sanitary risk level. MSF technology has a great potential to reduce the physical-chemical and bacteriological risk associated with surface water sources. However, the MSF technology is not a panacea and has its limitations, particularly with high levels of contamination, not always producing water of a quality that can be properly disinfected.

16.5 Cost considerations

Some components of a filtration system have the greatest impact (about 80%) on the construction cost. These include civil works, filter media, the excavation and the valves. The cost efficiency increases with the size of the system. Nevertheless, for this type of filtration systems the economy of scale is limited, which favours a relative short design period of some ten years.

The operation and maintenance cost of MSF systems is mainly determined by labour cost; in Colombia staff costs made up 85% of the total.

16.6 Selection of MSF alternatives

Different combinations of filtration stages are identified to treat raw water types. In general, filter bed lengths increase with the contamination levels in raw water types while filtration rates decrease. Unsurprisingly, capital and running costs of MSF plants increase with increasing contamination levels in their raw water types. Table 16.8 gives a selection guide based on the parameters faecal coliforms densities, turbidity and colour. All these MSF alternatives fulfil proposed water treatment objectives.

Table 16.8 An example of a selection guide for MSF alternatives fulfilling established treatment objectives for removing turbidity, faecal coliform bacteria and colour, based on experiences in the Andean Colombian Cauca Valley.

Faecal coliforms (CFU/100ml)	Y	Y4 Mean < 15,000 Max. < 45,000	DyGF ^{2.5} UGFS(3) ^{0.6} SSF ^{0.15}	DyGF ^{2.0} UGFS(3) ^{0.6} SSF ^{0.15}	DyGF ^{2.0} UGFS(3) ^{0.6} SSF ^{0.15}	DyGF ^{1.5} UGFS(3) ^{0.45} SSF ^{0.15}	DyGF ^{1.5} UGFS(3) ^{0.3} SSF ^{0.15}
		Y3 Mean < 5000 Max. < 15,000	DyGF ^{2.5} UGFS(2) ^{0.6} SSF ^{0.15}	DyGF ^{2.0} UGFS(2) ^{0.6} SSF ^{0.15}	DyGF ^{2.0} UGFS(3) ^{0.6} SSF ^{0.15}	DyGF ^{1.5} UGFS(3) ^{0.45} SSF ^{0.15}	DyGF ^{1.5} UGFS(3) ^{0.3} SSF ^{0.15}
		Y2 Mean < 1500 Max. < 5000	DyGF ^{2.5} UGFL ^{0.6} SSF ^{0.15}	DyGF ^{2.0} UGFL ^{0.6} SSF ^{0.15}	DyGF ^{2.0} UGFS(3) ^{0.6} SSF ^{0.15}	DyGF ^{1.5} UGFS(3) ^{0.45} SSF ^{0.15}	DyGF ^{1.5} UGFS(3) ^{0.3} SSF ^{0.15}
		Y1 Mean < 750 Max. < 2500	DyGF ^{2.5} UGFL ^{0.75} SSF ^{0.20}	DyGF ^{2.0} UGFL ^{0.6} SSF ^{0.15}	DyGF ^{2.0} UGFS(3) ^{0.6} SSF ^{0.15}	DyGF ^{1.5} UGFS(3) ^{0.45} SSF ^{0.15}	DyGF ^{1.5} UGFS(3) ^{0.3} SSF ^{0.15}
Turbidity (NTU)	X	Mean <	5	10	16	20	25
		P _{95%} <	15	30	50	60	70
		Max. <	50	100	150	225	300
Colour (PCU)	Z		X ₁	X ₂	X ₃	X ₄	X ₅
		Mean <	10	13	16	18	20
		Max. <	30	40	50	55	60
		Z ₁	Z ₂	Z ₃	Z ₄	Z ₅	

Explanation of the selection guide in table 16.8:

1. The number between brackets indicates the number of filtration steps in UGFS alternatives. The sub-index means filtration rates in mh^{-1} .
2. Raw water may be directly disinfected (without filtration) if turbidity and faecal coliform levels are below 5 NTU and 20 CFU/100 ml in 95% of samples respectively. These low contamination levels must be confirmed periodically with sanitary inspections and analyses in the watershed area.
3. DyGF + SSF (without CGF stage) could be applied if turbidity and faecal coliform levels are below 10 NTU and 20 CFU/100 ml in 95% of samples respectively. These low contamination levels must be confirmed periodically with sanitary inspections and analyses in the watershed area.
4. Turbidity treatment objectives (≤ 10 and 5 NTU in CGF and SSF effluents respectively) should be obtained with 95 percentile (P_{95}) turbidity values. It is expected that maximum (peak) turbidity values can be treated thanks to the protection capacity of the DyGF stage, combining flow reductions with higher removal efficiencies.
5. Faecal coliform treatment objectives (≤ 1000 and 10 CFU/100 ml in CGF and SSF effluents respectively) should be obtained with maximum faecal coliform levels. With medium faecal coliform levels in raw water sources SSF effluents should have effluents with mean values ≤ 3 CFU/100 ml before terminal disinfection).
6. Colour treatment objective (≤ 15 PCU in SSF effluents) should be obtained with maximum colour levels. This is a secondary treatment objective and should not compromise previous treatment objectives or terminal disinfection as a safety barrier.

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17 Rapid filtration

Luiz di Bernardo

17 Rapid filtration

17.1 Introduction

As explained in the previous chapter, filtration is the process whereby water is purified by passing it through a porous material (or *medium*). In rapid filtration sand is commonly used as the filter medium¹ 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 0.4-1.2 mm, and the filtration rate is much higher, generally between 5 and 15 m³/m².h (120-360 m³/m².day). Due to the coarse sand used, the pores of the filter bed are relatively large and the impurities contained in the raw water penetrate deep into the filter bed. Thus, the capacity of the filter bed to store deposited impurities is much more effectively utilised and even very turbid river water can be treated with rapid filtration. To clean a rapid filter bed it is not sufficient to scrape off the top layer. Cleaning of rapid filters is effected by backwashing. This involves directing a high-rate flow of water back through the filter bed whereby it expands and is scoured. The backwash water carries away the deposited material that was clogging the filter. Cleaning of a rapid filter can be carried out quickly; it need not take more than about half an hour. It can be done as frequently as required, if necessary each day. However, a limit does have to be placed on the frequency of backwashing because of the amount of the wash water used. Wash water has to come from the filtered water production. If the filtration runs fall below six hours, then there will be only a small net output of filtered water available for drinking water supply. Then some change in operation will be necessary, for example improving the pre-treatment in the sedimentation stage. In monsoon periods coarse pre-filtration may be necessary, as described in the previous chapter for protecting slow sand filters against high turbidity loads.

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 frequently provided as a pre-treatment to form insoluble compounds of iron and manganese (Fig. 17.2). See also chapter 13 on aeration.

1 Anthracite, crushed coconut shell, pumice, baked clay pellets, and other materials are also used especially in multiple-layer filter beds where one or more layers of such materials are placed on top of a (shallow) sand bed. For small community water supplies, more layers than two are not generally recommended as the advantages of multiple-layer filters are small compared with the additional design and operational complexities, in choosing the different media and maintaining their relative layer positions during backwashing. Anthracites from different parts of the world may have different densities. This will affect the backwashing regime, and may give rise to losses of anthracite over the wastewater weirs.

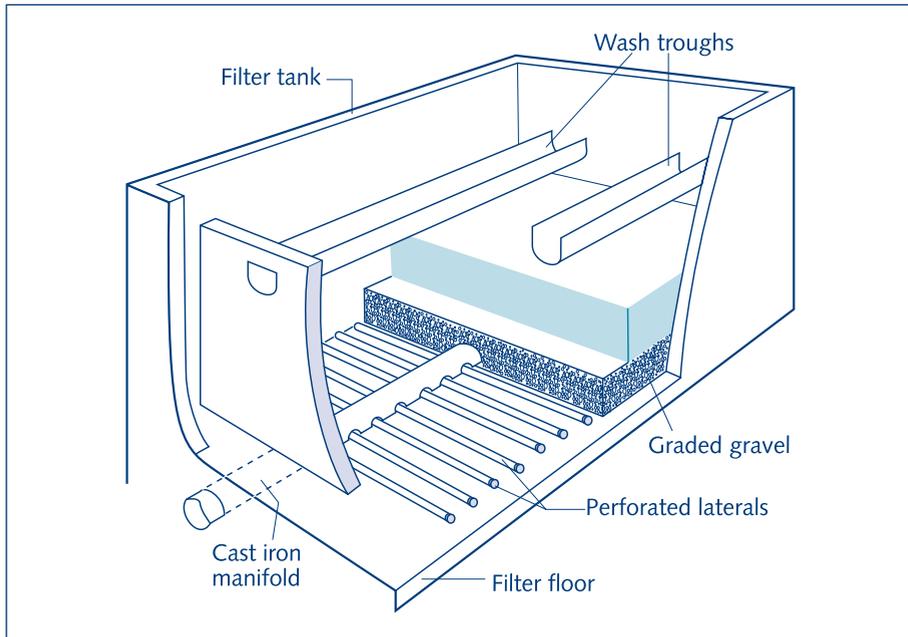


Fig. 17.1. Rapid filter (open-gravity type)

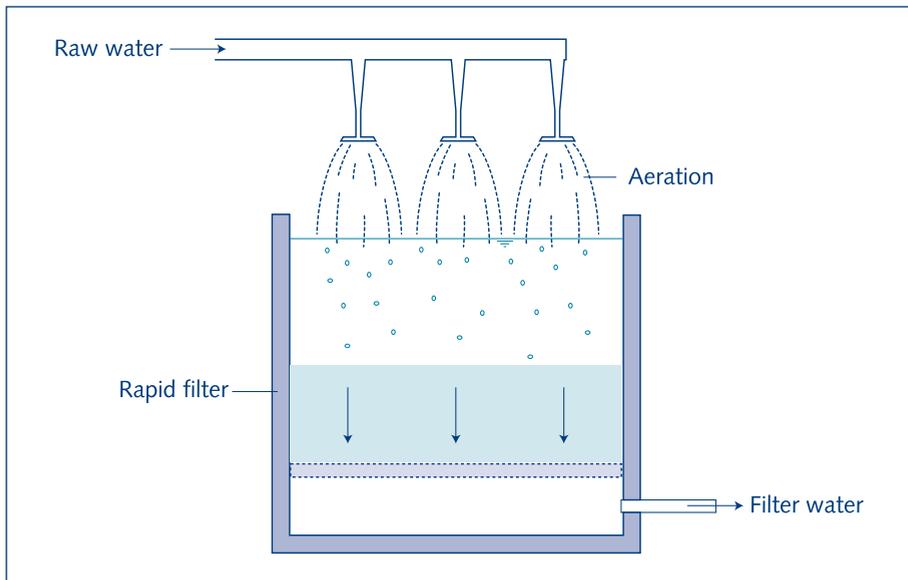


Fig. 17.2. Rapid filtration of pre-treated (aerated) water

For water with a low turbidity, as frequently found in lakes and sometimes in rivers, rapid filtration should be able to produce clear water, though it may still contain pathogenic micro-organisms. A final treatment such as chlorination is then necessary to obtain bacteriologically safe water.

In the treatment of river water with high turbidity, rapid filtration may be used as a pre-treatment to reduce the load on the following slow sand filters (Fig. 17.3), or it may be applied for treating water that has been clarified by coagulation, flocculation and sedimentation (Fig. 17.4). In such cases again a final chlorination is required.

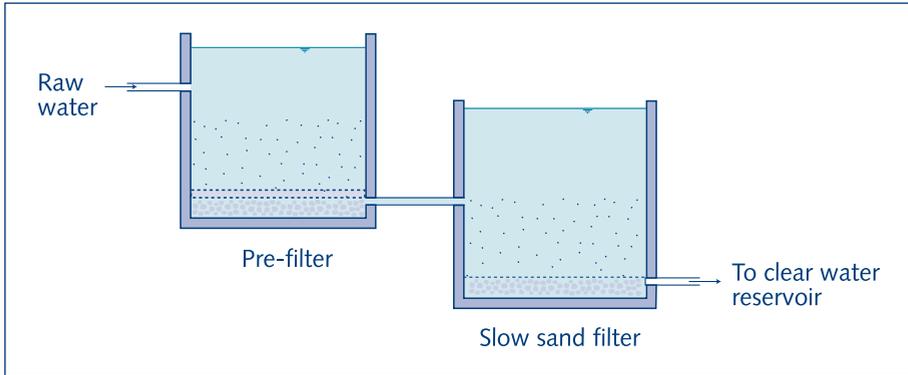


Fig. 17.3. Rapid filtration followed by slow sand filtration

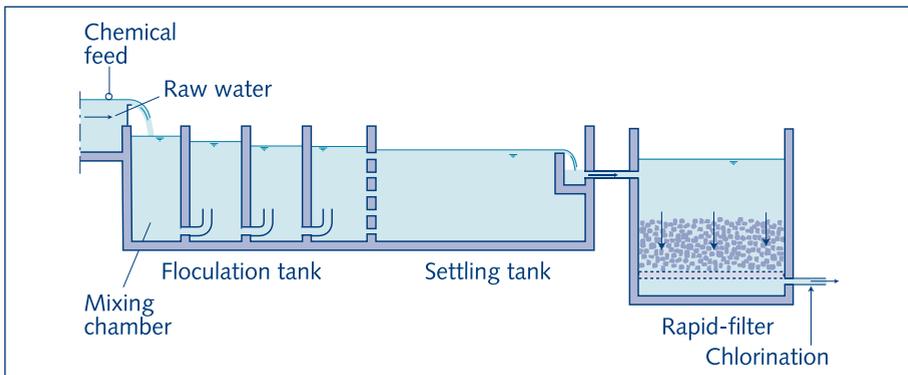


Fig. 17.4. Rapid filtration after coagulation and flocculation, and sedimentation

17.2 Types of rapid filters

Rapid filters are mostly built open with the water passing down the filter bed by gravity (Fig. 17.1). Rapid filters may be classified according to

- hydraulic head: gravity or pressurised
- flow direction: upflow or downflow
- filtration rate: constant or declining

The influent to the filters may be:

- clarified water, when coagulation, flocculation and sedimentation (or flotation) are performed before downflow filtration;
- coagulated water, in upflow direct filtration or in-line downflow direct filtration;

- flocculated water, when coagulation and flocculation steps exist upstream of the downflow direct filtration;
- pre-filtered water when upflow direct filtration is used upstream of the downflow filtration.

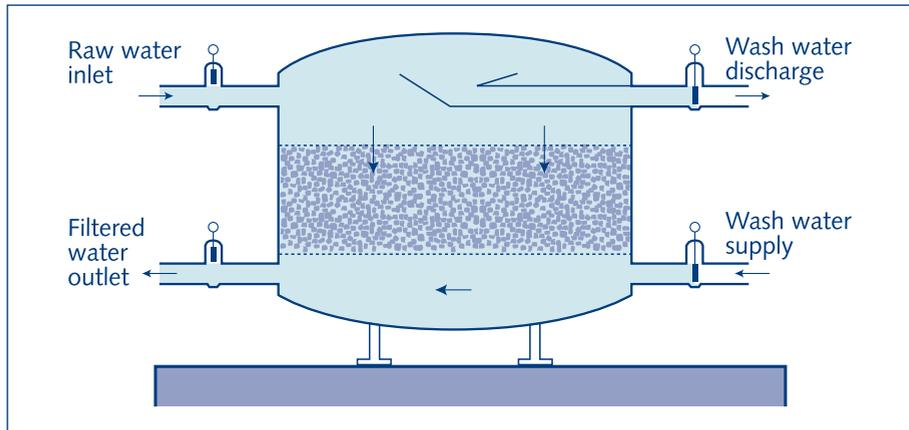


Fig. 17.5. Pressure filter

Pressure filters (Fig. 17.5) are of the same construction as gravity-type filters but the filter bed together with the filter bottom is enclosed in a watertight steel pressure vessel. The driving force for the filtration process here is the water pressure applied on the filter bed, which can be so high that almost any desired length of filter run is obtainable. Pressure filters are commercially available as complete units. They are not so easy to install, operate and maintain, particularly as it is not readily possible to inspect the condition of the media. For this reason they are not very well suited for application in small treatment plants in developing countries.

Upflow filters (Fig. 17.6) 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, with no great increase of the filter bed resistance, (*head loss*) due to the large pores. The overlaying fine layers have smaller pores but here also the filter resistance will increase only slowly as not many impurities are left to be filtered out.

In upflow filters, sand is used as the single filter medium. However, there are hygienic objections to using them as a final filtration stage in drinking water purification. The first of these is that the filtrate, being above the sand bed, is liable to contamination by birds (particularly water fowl who may like to swim in it) or from airborne dust. Covering the top, or operating the filters in a building, is an answer to this objection, but involves extra cost. The other objection is that backwashing the filter, which generates very dirty water, contaminates the same part of the filter unit as is used for delivering the clean filtered water. Special pipe work, such as a perforated filtrate pipe buried in the upper

part of the sand, can reduce this hazard to hygiene. This again would entail extra cost. Upflow filters are frequently used for the pre-treatment of water that is further purified by gravity-type rapid filters or by slow sand filters. In such cases, they can give excellent results and may be well suited for use in small treatment plants.

One drawback is that the allowable resistance (head loss) over an upflow filter should not be 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.

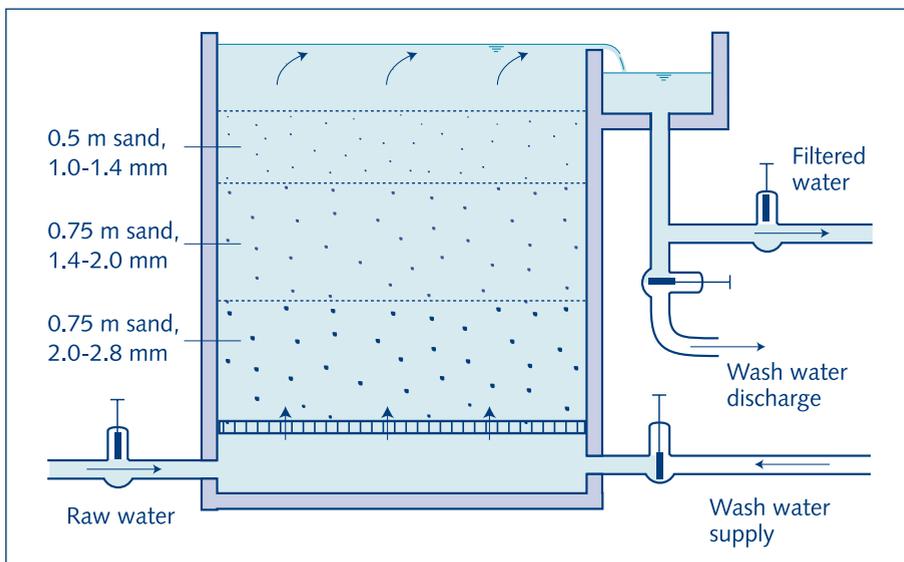


Fig. 17.6. Upflow filter

Upflow and downflow direct filtration

In direct filtration the influent of the rapid filter is coagulated, flocculated or pre-filtered water. For water with low turbidity (up to 25 NTU, with peaks up to 100 NTU), downflow (Fig. 17.2) or upflow (fig. 17.6) direct rapid filtration should be able to produce clear water. If raw water is of poor quality (turbidity up to 100 with peaks up to 200 NTU), double filtration may be used to produce a good effluent. Double filtration may consist of upflow direct filtration followed by downflow rapid filtration (Fig. 17.4). The double filter can also be constructed in one unit (Fig. 17.5). In all cases a final treatment such as chlorination is then necessary to obtain bacteriologically safe water.

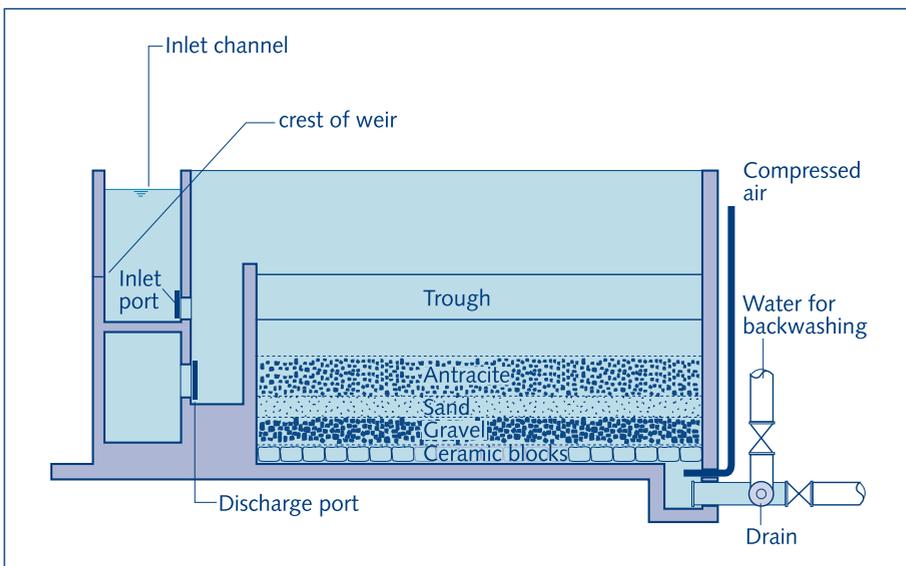


Fig. 17.7. Downflow rapid gravity filter receiving coagulated water (downflow direct filtration)

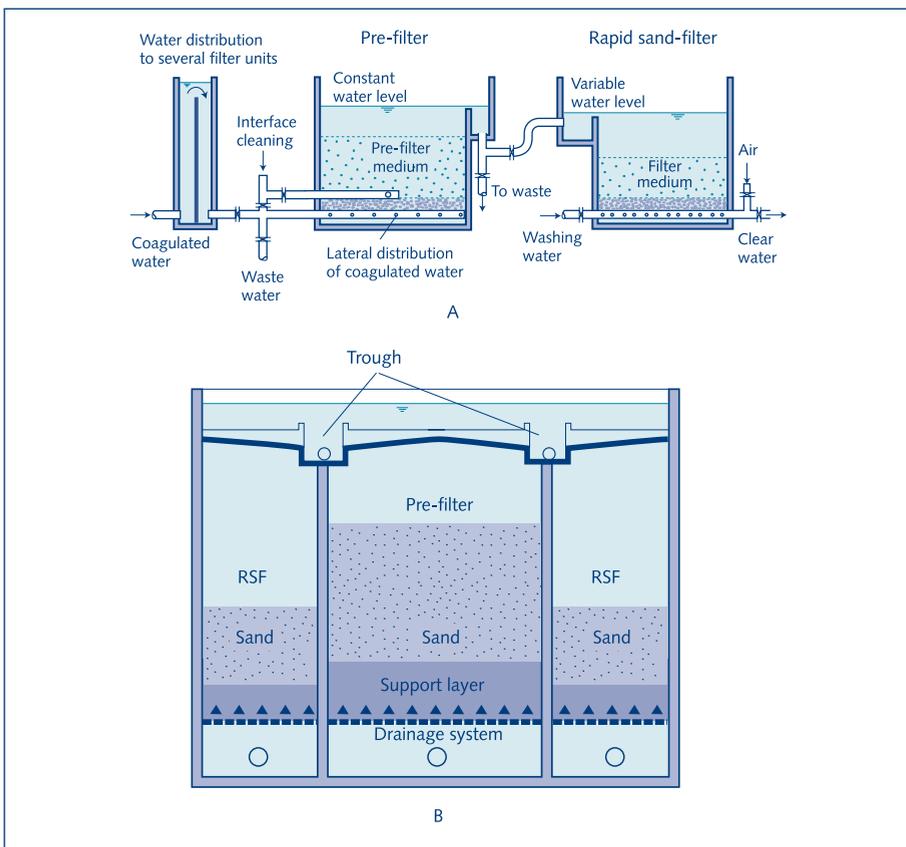


Fig. 17.8. Double filtration; downflow rapid gravity filter preceded by upflow coarse medium filter (a) as a separate unit; (b) built in same unit

Usually 30-50 % of head loss due to solids retention occurs at the gravel layer, while the remaining portion has been retained at the sand sub-layers next to the gravel-sand interface (thickness of about 0.4-0.6 m). The filter run can be extended by regular down-flushes in the operation of constant rate filtration. In this method the influent at the bottom is closed and the water in the filter is discharged by downflow. These down-flushes result in partial removal of solids. Gravel layer and sand sub-layers next to the gravel-sand interface are partially cleaned and a significant recovery of hydraulic head is achieved (Fig. 17.9).

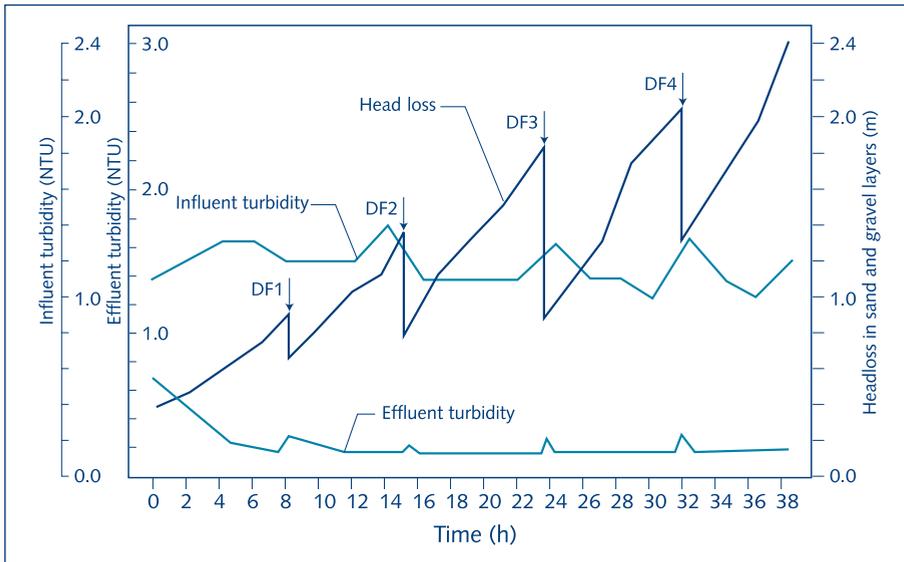


Fig. 17.9. Upflow filter operations with down-flushes during filter run

Multiple-media filters (Fig. 17.10) are gravity-type, downflow filters with the filter bed composed of several different materials, which are placed coarse-to-fine in the direction of flow. For small-sized rapid filters it is common to use only two materials in combination: 0.3-0.5 m of sand with an effective size of 0.4-0.7 mm as the under layer, topped by 0.5-0.7 m of anthracite (see also footnote on variability of anthracite, section 17.1), pumice or crushed coconut husks with an effective size of 1.0-1.6 mm. As a final treatment, multiple-layer filters can give excellent results and, when suitable materials are available locally, application in small treatment plants is a possibility. However, the stability of multiple-layer filters is difficult to design and maintain, particularly after backwashing.

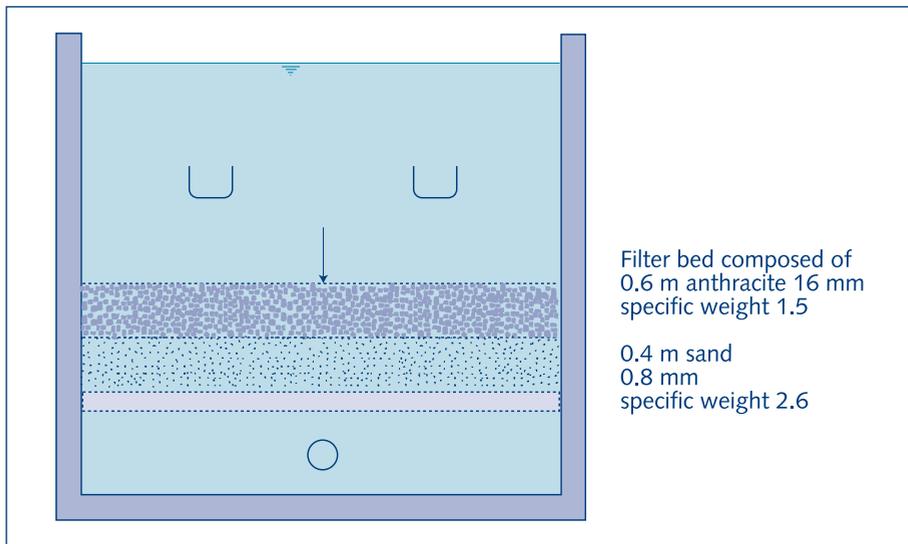


Fig. 17.10. Dual-media filter bed

17.3 Theoretical aspects

The overall removal of impurities from the water in rapid filtration is brought about by a combination of several different processes. The most important are straining, sedimentation, adsorption, and bacterial and biochemical processes. These are the same processes already described for slow sand filtration (in section 16.2). In rapid filtration, however, the filter bed material is much coarser and the filtration rate much higher (up to 50 times higher than in slow sand filtration). These factors completely alter the relative importance of the various purification processes.

The straining of impurities is not an important mechanism in rapid sand filtration due to the relatively large pores in the filter bed. Sedimentation is a significant mechanism, as in downflow filtration particles collect preferentially on the top of grains, forming caps; this is also due to the laminar flow regime. Thus, straining and sedimentation will retain far fewer impurities than in a slow sand filter. The upper filter bed layers in particular will be far less effective and there will be a deep penetration of impurities into the entire bed of a rapid filter.

By far the most important purification effect in rapid filtration is the adsorption of impurities. Although the surface of initially clean sand has a small negative electrostatic charge, this negative charge is neutralised very close to the particle by dissolved material in the water, and deposited positively charged material like aluminium or iron flocs. Thereafter, positively charged flocs encounter positively charged floc-covered sand surfaces, and electrostatic repulsion may be evident. The magnitude and effect of this is not only dependent on the electrostatic charges on the floc particle and sand surface,

but also the amount and nature of the salts dissolved in the water. The more minerals the water contains, the less is the range of effect of the electrostatic forces. Most significantly, forces of Van der Waals occur at very close approach (less than 1 micro metre) of the surfaces of the particle and the grain, due to the electronic nature of the atoms and molecules of the two approaching surfaces. Such forces always attract and account for the attachment (*adsorption*) of flocs and other particles with an electric charge to sand grains, and existing deposits with an opposite electric charge (see Ives, 1967 for further details).

In a slow sand filter the water stays in the filter bed for several hours, but with rapid filtration the water passes in only a few minutes. Accumulated organic deposits are frequently removed from a rapid filter when the filter is cleaned by backwashing. There is very little time and opportunity for any biodegradation of organic matter to develop, or for killing of pathogenic micro-organisms to take place. The limited degradation of organic matter need not be a serious drawback as the accumulated deposits will be washed out of the filter during backwashing. The poor bacteriological and biochemical activity of a rapid filter will generally be insufficient to produce bacteriologically safe water. Hence, further treatment such as slow sand filtration or chlorination will be necessary to produce water that is fit for drinking and domestic purposes.

17.4 Rapid filter operation and control

The operation of a rapid filter (gravity type) is shown schematically in figure 17.11.

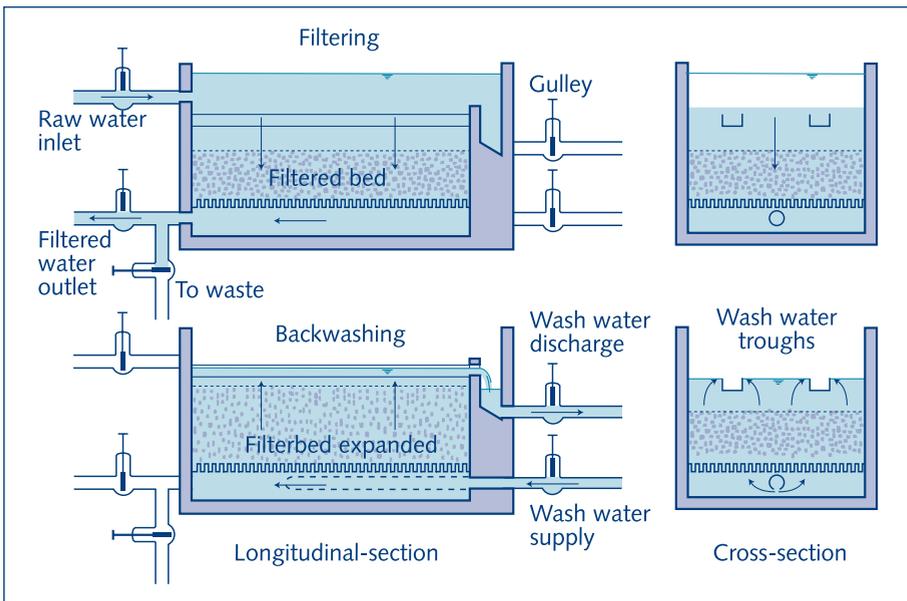


Fig. 17.11. Rapid filter (gravity type)

During filtration the water enters the filter through valve A, moves down towards the filter bed, flows through the filter bed, passes the underdrainage system (filter bottom) and flows out through valve B. The unit used to measure filtration rate is actually the approach velocity, which is the inflow rate (m^3/h) divided by the filtration area (m^2). The interstitial velocity in the bed is higher, as it is the filtration rate divided by the average porosity of the filter medium.

Due to gradual clogging of the pores the filter bed's resistance against the downward water flow will progressively increase. This will reduce the filtration rate unless it is compensated by a rising raw water level above the filter bed. In this example, the rapid filters are designed to operate with a constant raw water level, which requires that the filter is equipped with a filter rate control device in the influent or effluent line. These filter rate controllers provide an adjustable resistance to the water flow. They open gradually and automatically to compensate for the filter bed's increasing resistance and so keep the operating conditions of the rapid filter constant.

When, after a period of operation, the filter rate controller is fully opened, further clogging of the filter bed cannot be further compensated and the filtration rate will fall. The filter is then taken out of service for backwashing. For this, the valves A and B are closed, and valve D is opened to drain the remaining raw water out of the filter. A few minutes later valve E is opened to admit the wash water. The backwash rate should be high enough to expand the filter bed by about 20% so that the filter grains can be scoured, and the accumulated deposits carried away with the wash water. The wash water is collected in the wash water troughs from where it drains to waste. When the backwashing is completed, valves E and D are closed and valve A is re-opened, admitting raw water to begin a new filter run.

For fine filter bed material, the scouring action produced by the wash water during backwashing may not be sufficient to keep the filter bed clean in the long run. It is then desirable to provide an additional scour by using air and water in combination for backwashing. This, however, is much more complex than backwashing with water only, and is generally not to be recommended for small water treatment plants.

Filter control

Table 17.1 gives an overview of the four possible filter operation controls.

Option 2 with constant water level (available head), constant total resistance and constant filtration rate has been discussed. Figure 17.12 shows the head loss division over bed and controller for this case.

Table 17.1 Filter operation control

Option	Available head	Total resistance to filtration	Filtration rate	Remarks
1	Constant	Variable	Declining	-----
2	Constant	Constant	Constant	Inlet or outlet control devices are necessary
3	Variable	Variable	Constant	Filters are individually fed by free discharge weirs and the water level into each one follows the increase of head loss due to solids retention
4	Variable	Variable	Variable	Declining-rate filtration system; filter inlets are submerged; the cleanest filter in the battery works at the highest filtration rate and the dirtiest at the minimum filtration rate

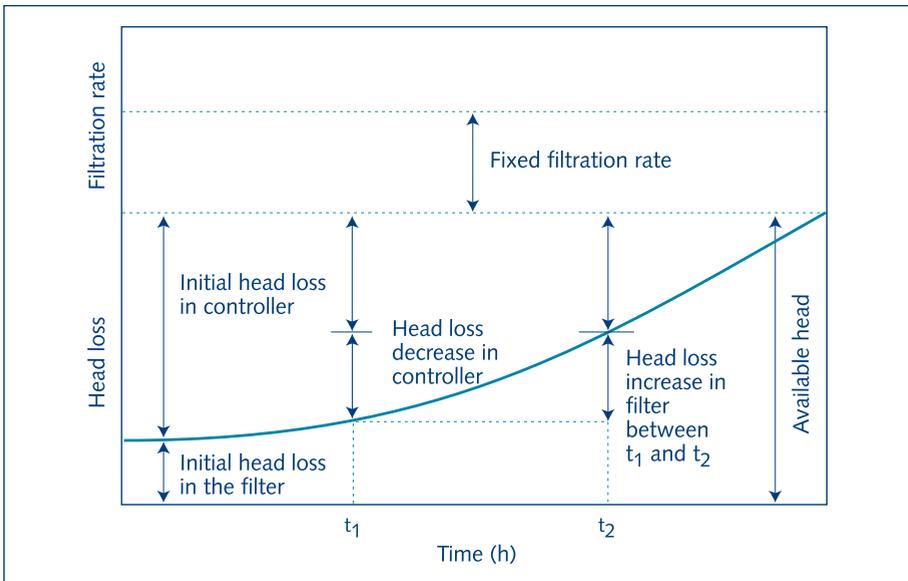


Fig. 17.12. Head loss in Constant Head – Constant Rate filter control

The filtration rate can be controlled jointly for all filter units by the raw water feeding rate. It can be readily adjusted to meet the demand for filtered water. In this arrangement there will be considerable variations of the raw water level in the filters,

which may be awkward. If so, another arrangement as shown in figure 17.14c may be preferred. Here a float-controlled valve is used to keep the raw water level in each filter constant. Individual rate controllers allow each filter unit to operate at its optimal filtration rate (Fig. 17.13). This advantage, however, is not very great and such rate controllers are generally very expensive and not easy to maintain.

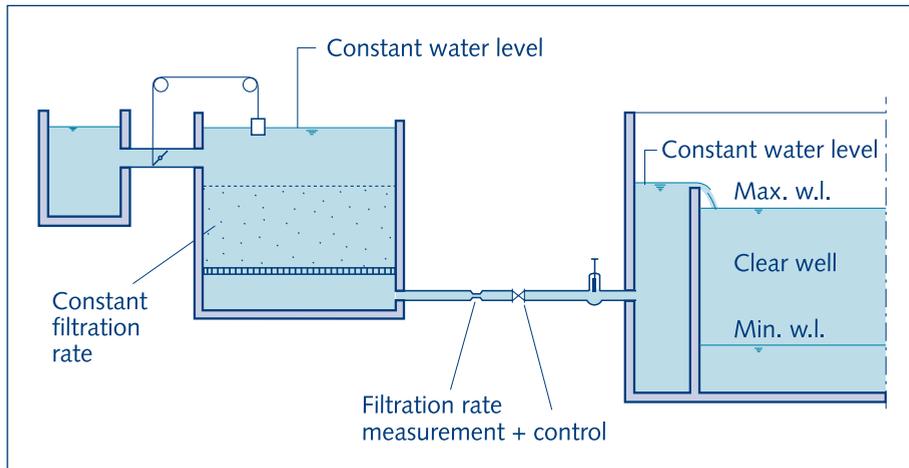


Fig. 17.13. Filter rate control

Filter control for option 3: variable water level – constant rate control

Filter control arrangements using an even distribution of the raw water (*flow splitting*) over the filter units or uniform withdrawal of the filtered water, are widely used in Europe and North America. Various methods can be employed. The one shown in figure 17.14b is probably the simplest as there are no moving parts at all. In this type the raw water enters the filter over a weir. The weir crest is at the same level for all filters. The raw water conduit feeding the filter units is generously sized so that the water will flow without any appreciable head loss. The water level in it will be practically the same at each entrance weir. Thus, the overflow rate at each weir will be the same, and the raw water feed to the filter units will be equally split. This method of control is known as *influent flow splitting*.

The water level in the filters of a bank must be different, otherwise the filters would all end the run at the same time. The necessary phasing can be accomplished during the plant start-up by backwashing each filter after some period of operation. For example, with four filters in a bank, filter F1 is backwashed after 4 hours of operation, filter F2 after 8 hours, filter F3 after 12 hours and filter F4, after 16 hours. Once this situation is reached, the operator will backwash filter F1 only when the available head is consumed (at the maximum level fixed by the designer) and so on. Since the filters are fed independently by using free discharge weirs, the water level in each one will vary independently from the others, so that the end of the run is visible for the operator. At

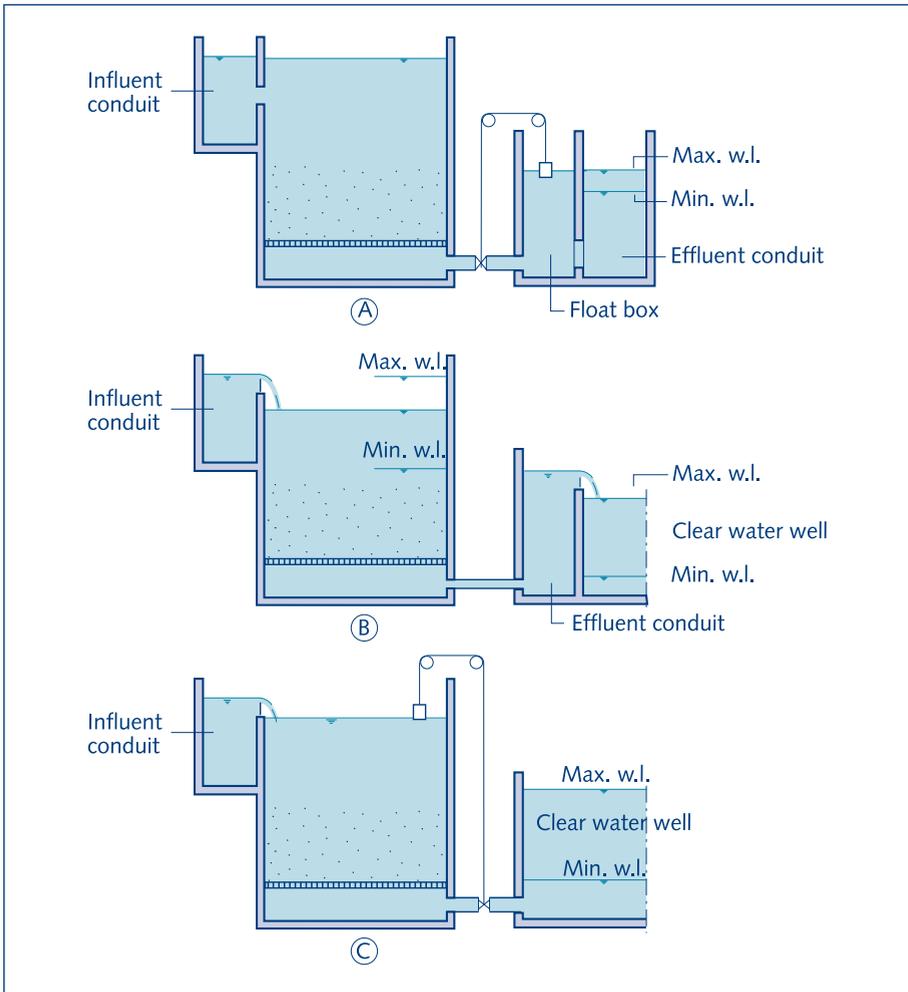


Fig. 17.14. Filter control systems

the beginning of the run, the medium is clean. To avoid the water level falling below the top of the sand bed, an outlet weir is recommended with the crest just above the level of the sand bed. This outlet weir can be placed in the individual box for each filter or in the common outlet channel. Compared to constant-rate constant level control, the influent flow splitting method has the following advantages:

- Filtration rate is maintained without the use of automatic controlling devices
- The influent volumetric flow rate is equally split by means of a simple hydraulic device, such as triangular or rectangular weirs
- When one filter is taken out of service for backwashing, the flow rate is uniformly distributed to the remaining filters in operation and the water level and the filtration rate in each one will gradually rise
- As soon as the clean filter is placed back to work, the decrease in filtration rate is also be gradual in the remaining filters

- Head loss in any filter is visual and depends only on the operator identifying the moment the dirtiest filter has to be backwashed by observation of the water levels in the units
- The position of the crest in the outlet weirs slightly higher than the top of the medium eliminates the occurrence of negative pressure
- The flow rate in each filter may be easily measured at the inlet weirs

Some disadvantages of this system have been claimed, such as the excessive height of the filter box and floc break-up due to the fall of water in the filter box. Without any doubt, the height of filter boxes is higher, but it should be considered against the substantial cost of acquisition of mechanical devices and the cost of operation and maintenance when constant-rate constant level control filters are used. In relation to the water falling at the inlet of the filters, it must be realised that a fall already exists in troughs used for settled water collection. The effect on performance of water falling upstream of the filters has been carefully investigated and no damage to the filtrate was observed.

Option 4: variable water level (head) and declining rate control

When no filtration rate controllers are used, filtration will take place at a declining-rate. The design of declining-rate filters is much simpler than for controlled-rate filters. Simple stop logs or gates can be used for filter control (Fig. 17.15).

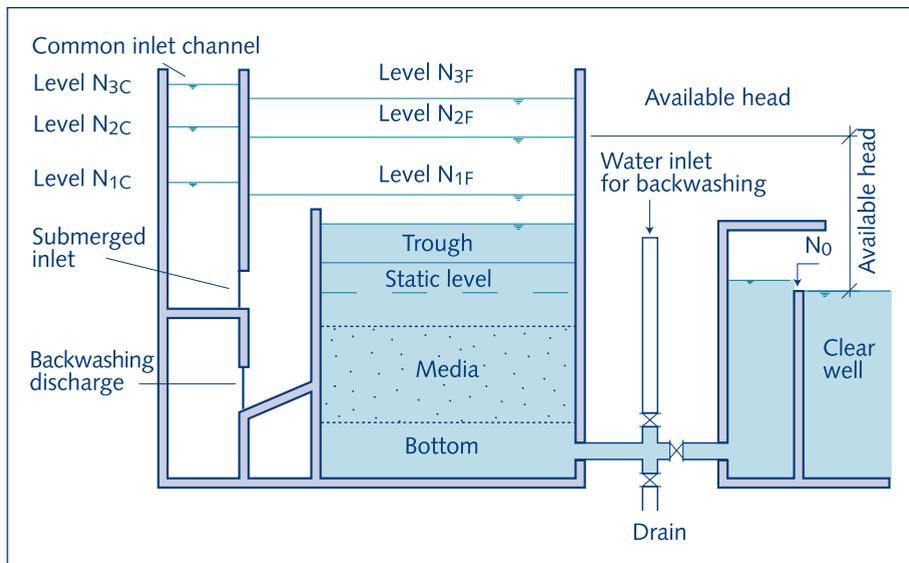


Fig. 17.15. Declining-rate filtration

All filters are in open connection with both raw and filtered water conduits. Consequently, all have approximately the same raw water level and filtered water level, so that all filters will operate under the same head. The filtration rate for the various filter units, however, will be different: highest in the filter just cleaned by backwashing and lowest for the one longest underway in its current filter run. For all filters jointly, the production will be determined by the supply of raw water, which should be high enough to meet the demand for filtered water. During filtration the filter beds gradually become clogged and the raw water level in all filters will rise due to the increased resistance against water flow. The filter unit that has been in operation for the longest period of time will reach the maximum allowable water level first, and then needs cleaning by backwashing.

After cleaning, this filter will have the lowest resistance against flow so that a considerable portion of the raw water supplied will pass through it. The load on the other filters is temporarily reduced. These units will show a fall of the influent water level but later the further clogging of the filter beds will cause the influent water level to rise again. When the maximum raw water level is reached in a second filter this one will be backwashed and so forth. The water level in the common inlet channel will vary depending on the operational stage of each of the filters; the supernatant water level is close to this level because the head loss at the inlet will be small. This water level in the common inlet channel is lowest when the backwashed filter is again put into operation, and highest just before the backwashed filter is resuming its operation. This is shown in figure 17.15.

If no special measures are taken, the filtration rate in a declining-rate filter just after cleaning can be very high, up to $25 \text{ m}^3/\text{m}^2\cdot\text{h}$, which is much higher than the average rate of $5\text{-}7 \text{ m}^3/\text{m}^2\cdot\text{h}$. When it is necessary to limit the filtration rate in order to safeguard the filtered water quality, an extra flow resistance device (e.g. an orifice) or a preset butterfly valve should be fitted in the effluent line. For pressure filters, declining-rate filtration is common practice. For gravity-type rapid filters, its application is gradually increasing in Great Britain, in Latin America and also, to a limited extent, in North America. Due to its simplicity, declining-rate filtration is certainly worth considering for small water treatment plants in developing countries.

Negative head

So-called negative head can occur when the pressure in the filter bed falls locally below atmospheric pressure. This may occur towards the end of the filter run and leads to air coming out of solution and appearing as bubbles that block the pores in the filter media. Erratic flow occurs, causing deposits to be scoured out of the filter pores and making the filtrate turbid. Some bubbles rise to the surface of the filter bed, creating channels through which unfiltered water may flow, again causing a penetration of turbidity into the filtrate. See also the remarks about "under-pressure" in chapter 16 section 16.2.1 on multi-stage filtration.

17.5 Design considerations

For the design of a rapid filter, four parameters need to be selected: the grain size of the filter material; the thickness of the filter bed; the depth of the supernatant water; and the rate of filtration. To the extent possible, these design factors should be based on experience obtained in existing plants that treat the same or comparable raw water. When such experience does not exist, the design should be based on the results obtained with a pilot plant operating experimental filters. The quality of the influent water (usually pre-treated water) greatly influences the range of the design parameters; in table 17.2, broad indicative ranges have been given.

Backwash arrangements

A rapid filter is cleaned by backwashing; that means directing a flow of clean water upwards through the filter bed for a period of a few minutes. Filtered water accumulated by pumping to an elevated tank can be used, or the effluent from the other (operating) filter units of the filtration plant can be used directly (*self-wash arrangements*). The velocity of the upward water flow should be high enough to produce a limited expansion of the filter bed, so that the accumulated deposits can be loosened and carried away with the wash water but without carrying sand or other filter media over the weir (Fig. 17.16). It is important to check media expansion if converting a sand filter to an anthracite/sand dual-media bed.

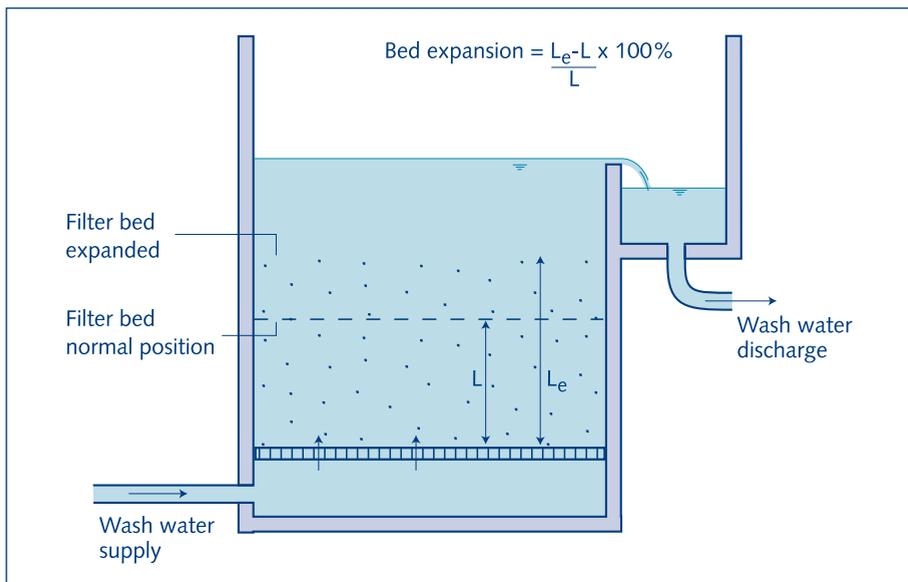


Fig. 17.16. Backwashing of rapid filter

For a filter bed of sand (specific weight: 2.65 g/cm^3) typical backwash rates giving about 20 % expansion are listed in table 17.3.

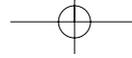


Table17.2 Indicative design ranges for rapid filtration

Design parameter	Downflow rapid sand filter using conventional sand	Downflow rapid sand filter using uniform sand	Upflow direct filtration	Double filtration: 1. upflow using gravel (2. as downflow sand filter)	Double filtration: 1. upflow using coarse sand (2. as downflow sand filter)
Grain size of filter medium	Grain size: 0.41-2.0 mm Effective size: 0.45-0.55 mm	Grain size: 0.84-1.68 mm Effective size: 1.0-1.2 mm	Grain size: 0.59-2.0 mm Effective size: 0.75-0.85 mm	Four sub-layers (each 40 cm) ranging from 32 mm-4.8 mm	Grain size: 1.0-3.2 mm Effective size: 1.19-1.41 mm
Filter bed thickness	0.6-0.8 m	0.8-1.2 m	1.6-2.0 m	1.5-1.8 m	1.2-1.6 m
Supernatant water depth	0.6-2.0 m *	0.8-2.5 m *	Not applicable		
Filtration rate	5-7.5 m/h	7.5-12.5 m/h	5-10 m/h	2.5-5 m/h	5-10 m/h
Support layer thickness	0.4-0.6 m **	0.3-0.5 m **	0.6-0.8 m	Not applicable	0.6-0.8 m

* Depends on the method of filter control and the filtration rate

** Depends on the drainage system

Table 17.3 Typical backwash rates

d (mm)	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.1	1.2
t (°C)	Backwash rate (m ³ /m ² .h)								
10°C	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

d = average grain size of filter sand (mm)

t = backwash water temperature (°C)

v = backwash rate (m³/m².h)

If the wash water is supplied by pumping, three (in very small installations two) pumps are normally used, one of which serves as the reserve unit. For high backwash rates and large filter bed areas, these pumps need to be of high capacity and their installation and operation is rather expensive. A wash water reservoir such as the one shown in figure 17.17 is then preferable; smaller pumps will be adequate to fill the reservoir during the intervals between successive backwashings. The reservoir generally should have a capacity between 3 and 6 m³ per m² of filter bed area and it should be placed about 4-6 m above the water level in the filter.

Again, three pumps are usually provided, one of which is the reserve unit. The total capacity of the two operating pumps should be about 10-20% of the wash water supply rate. A special wash water tank or reservoir is not necessary when the required wash water is taken from the filtered water reservoir. However, this may cause undesirable pressure fluctuations in the distribution system, due to the interrupted supply of water.

A simpler solution is to increase the depth of the water standing over the filter beds and to limit the maximum filter resistance. The filtered water will then be available at a head of some 1.5-2 m above the filter beds and that should be sufficient. The operating units of the filtration plant must supply enough water for the required backwash rate. For this reason a rapid filtration plant using this backwash arrangement should have at least six filter units. The wash water is admitted at the underside of the filter bed through the underdrain system (*filter bottom*). To divide the wash water evenly over the entire filter bed area, the underdrain system must provide a sufficient resistance against the passage of wash water (generally 0.6-1.0 m head of water). A frequently used underdrain system consists of laterals placed about 0.2 m apart and connected to a manifold (Fig. 17.19). The laterals have holes at the underside, with a diameter of about 10 mm. Rigid plastic pipes are generally used in this underdrain system.

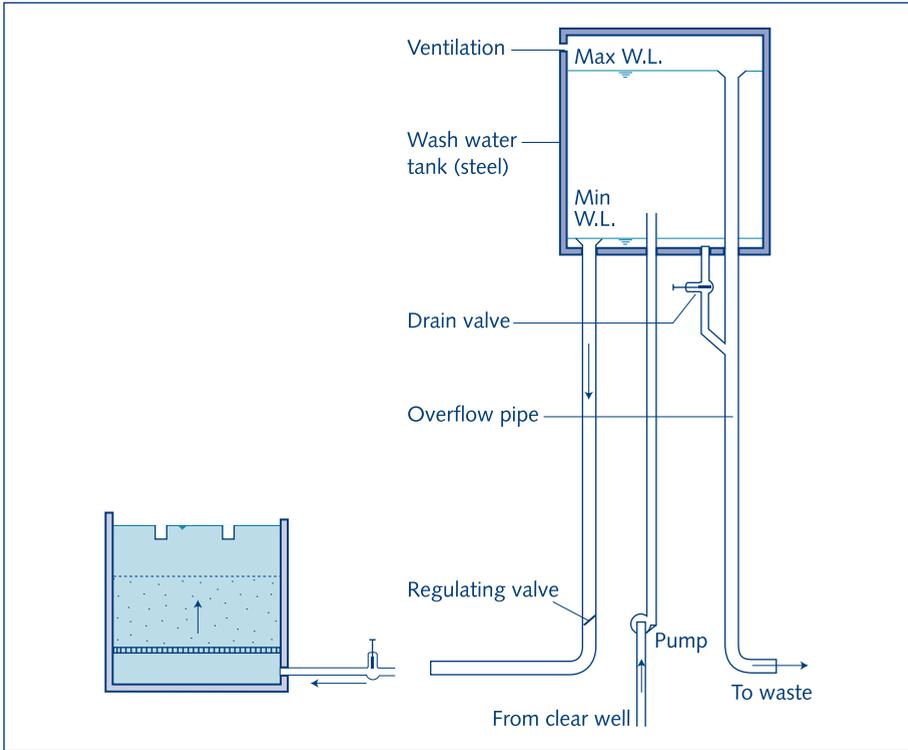


Fig. 17.17. Wash water tank arrangement

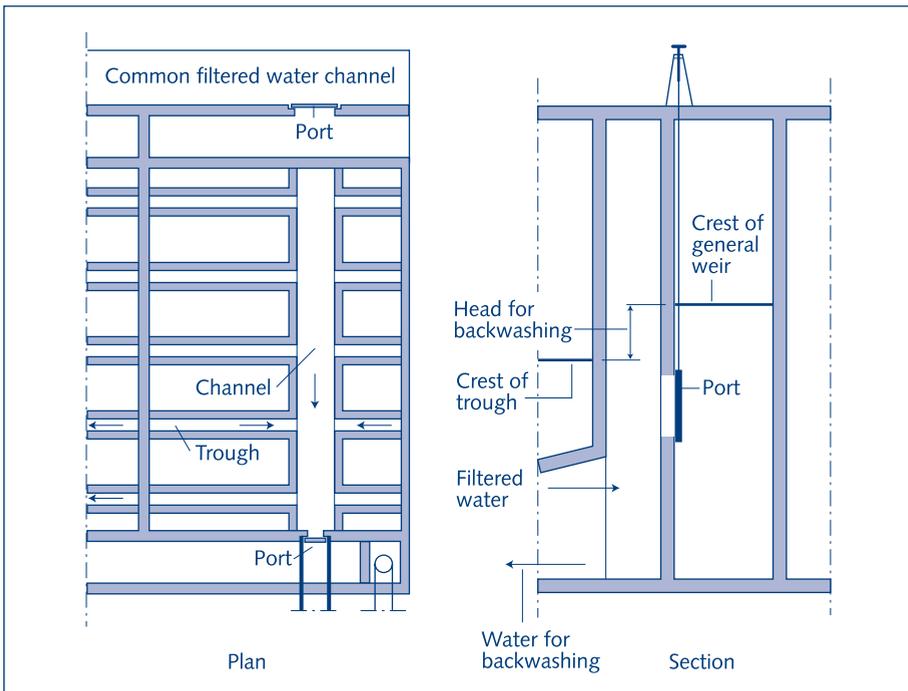


Fig. 17.18. Self-wash arrangements for (a) small plants and (b) medium-sized and large plants

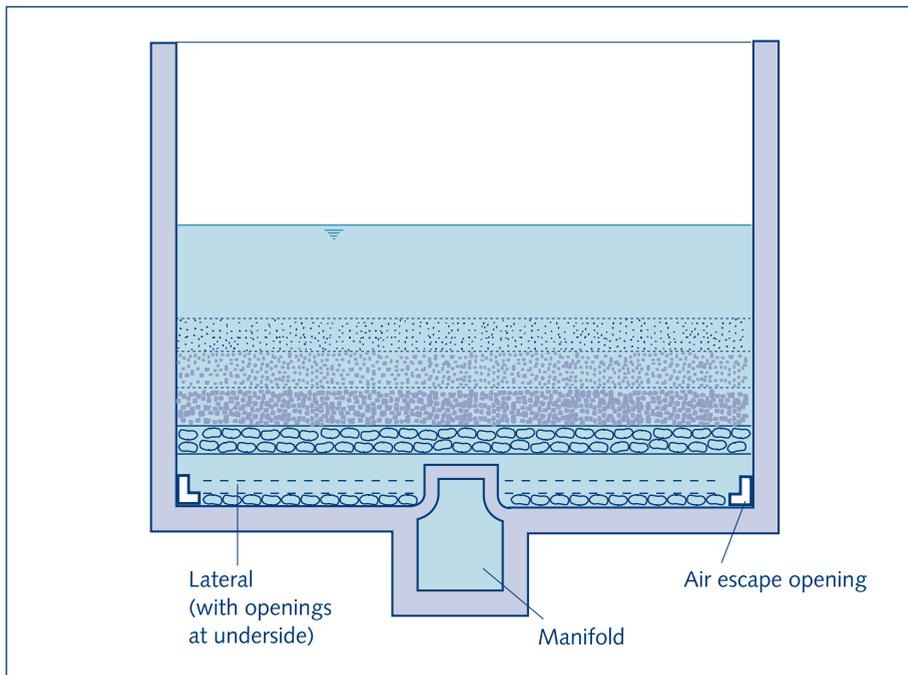


Fig. 17.19. Lateral underdrain systems

To prevent the filter material from entering the laterals through the holes, the filter bed should be supported by a layer of coarse material (e.g. gravel) that will not be dislodged by the backwash water jetting from the underdrain holes. For example, filter sand of 0.7-1.0 mm effective size would require four gravel layers; from top to bottom: 0.15 m x 2-2.8 mm, 0.1 m x 5.6-8 mm, 0.1 m x 16-23 mm and 0.2 m x 38-54 mm. The total gravel pack would be 0.55 m deep. Some other filter underdrain systems developed by the water industry are shown in the section "Construction".

After passing the filter bed, the wash water carrying the washed out impurities is collected and drained off via wash-water troughs. The distance the wash water will have to travel horizontally to a trough should be limited to about 1.5-2.5 m. The troughs are set with their top at 0.5-0.6 m above the unexpanded sand bed, and their cross-sectional area is derived from the consideration that at the discharge end of the trough the water depth will be the free discharge ("critical") depth (Fig. 17.20).

Table 17.4 gives wash water flow rates (Q) for combinations of depth of wash water flow (H) and width of wash water trough (b).

The wash water troughs can be placed in several ways. Figure 17.21 shows typical arrangements.

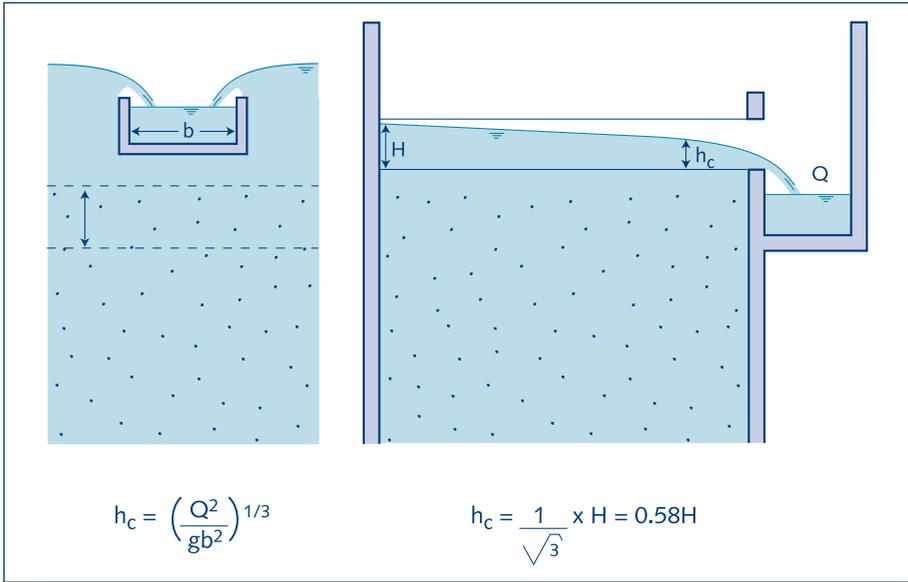


Fig. 17.20. Flow condition in wash water trough

Table 17.4 Wash water carrying capacity of troughs (l/s)

H = Depth of wash water flow in trough	Width of trough		
	0.25 m	0.35 m	0.45 m
0.25 m	43	60	78
0.35 m	72	100	129
0.45 m	104	146	188

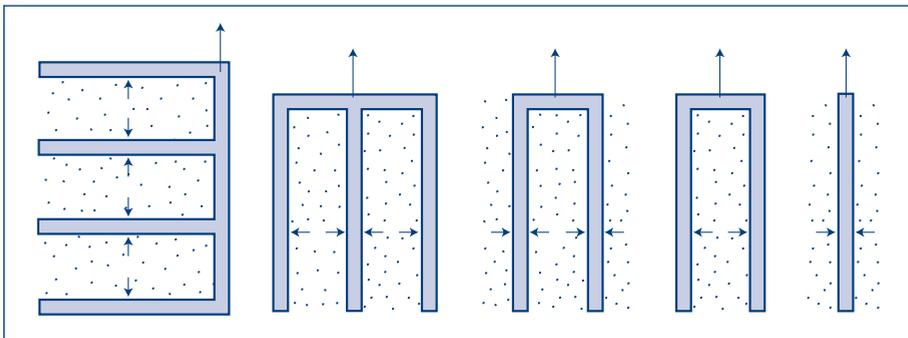


Fig. 17.21. Typical arrangements of wash water troughs

Particularly when fine sand is used with a grain size less than about 0.8 mm, the scouring force of the rising wash water may be inadequate to keep the filter grains clean in the long run. After some time they could become covered with a sticky layer of organic matter. This may cause problems such as mud balls and filter cracks.

These can be prevented by providing an additional air-wash scour. Filter cleaning now starts by backwashing with air at a 30-50 m/h rate, usually combined with a water wash at a 10-15 m/h rate. This should remove the coatings from the filter grains, and the loosened material is carried away by the following water wash. For backwashing with air a separate pipe system is necessary. An example is shown in figure 17.22. It should be noted that air-and-water backwashing generally is too complex an arrangement for small water treatment plants.

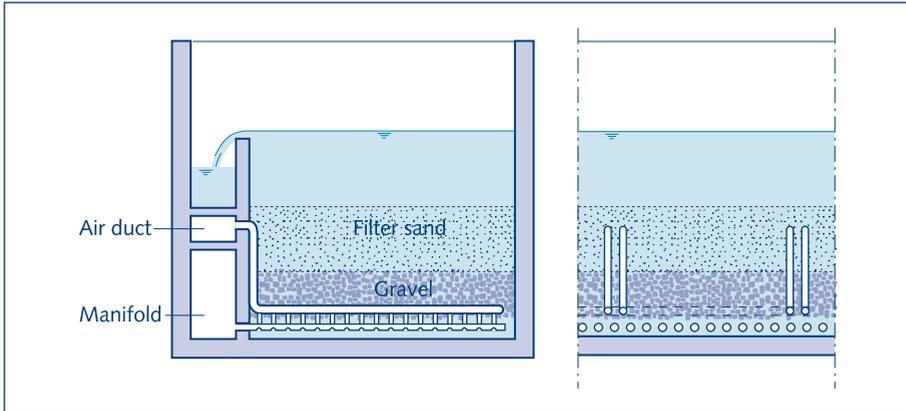


Fig. 17.22. Backwashing with air and water

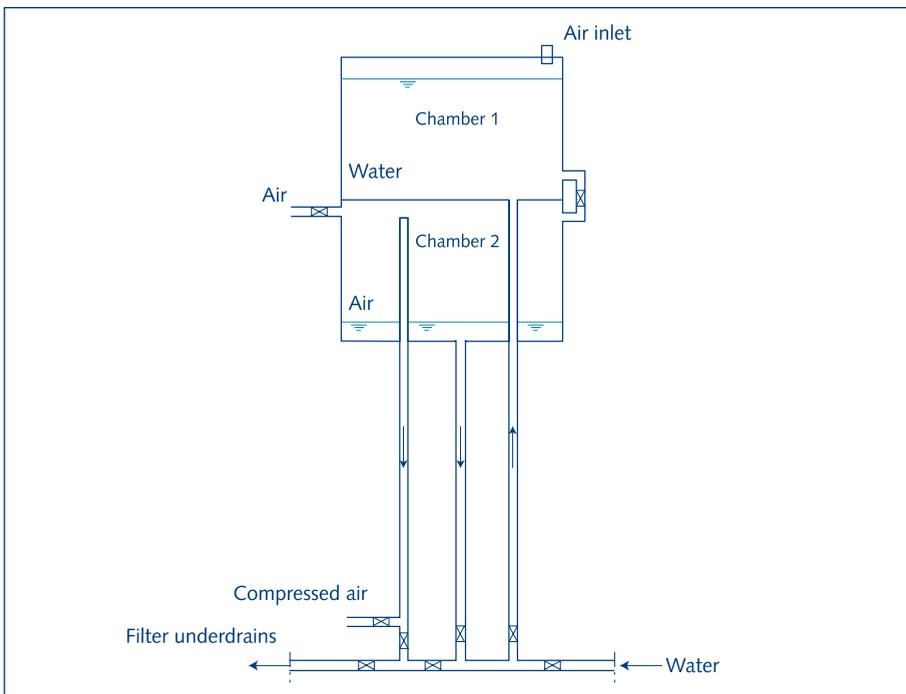


Fig. 17.23. Air-and-water backwash arrangement
Source: Sen, R.N. Indian Institute of Technology Kharagpur (India)
(applied at Metropolitan Water Board London, ca. 1930)

An interesting arrangement for feeding air and water for backwashing is shown in figure 17.24. Backwashing starts by allowing water from chamber 1 to flow into chamber 2. The air in chamber 2 is pressurized and admitted for scouring the filter. Then the water collected in chamber 2 is used to backwash the filter.

Rapid filtration plant layout

A rapid filtration plant consists of a number of filter units (minimum 2), each with an area A . When one filter is out of operation for cleaning, the remaining units must be able to provide the required capacity Q at the selected rate of filtration r . This is expressed in the formula: $Q = (n - 1) A \cdot r$

For small plants there is little choice regarding suitable combinations of n and A , but for larger plants the choice should be such that the cost of construction is minimized. As a tentative design step, the unit filter bed area (A) expressed in square metres may be taken as about 3.5 times the number of filter units n .

For economy in construction and operation, the filter units should be set in a compact group with the influent and effluent lines and any chemical feed lines as short as possible. The siting of the various units of a rapid filtration plant is a matter that warrants the closest attention of the design engineer. Allowance should be made for a future expansion of the plant. An example is shown in figure 17.24. Common facilities such as wash water pumps and tanks, and chemical solution feeders are best placed in a service building, which also should contain the office, laboratory and storerooms, and chemical handling, storage, and sanitary facilities.

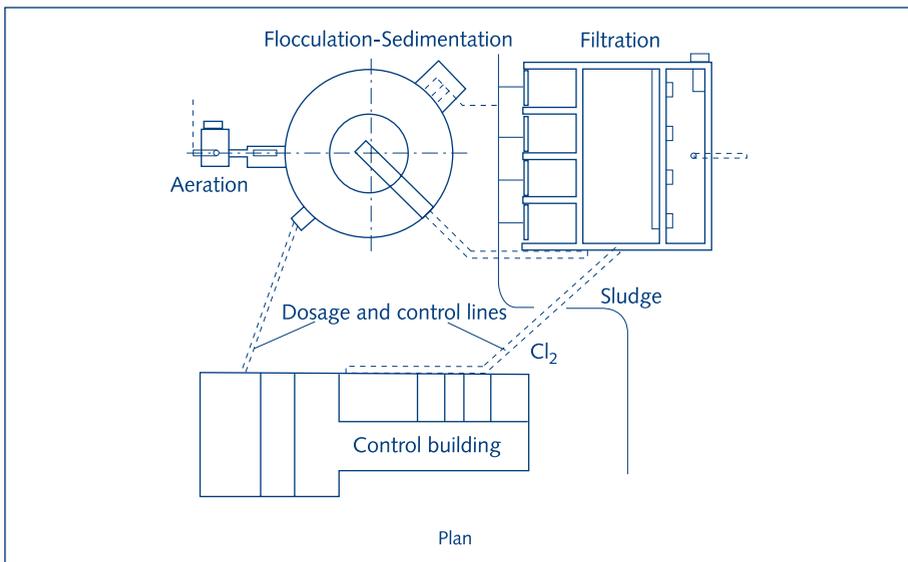


Fig. 17.24. Rapid filtration plant layout

Many designs place the service building in the centre, while in the wings the various filter units are arranged on one or two sides of a two level corridor, the upper level being the operating floor and the lower level the pipe gallery.

17.6 Construction

As explained in the preceding sections, a rapid filter consists of a tank containing the underdrain system, the filter bed and the supernatant water. The filter tank is generally made of reinforced concrete, rectangular and with vertical walls. The design of the concrete structure follows common rules with the added difficulty that the water retaining structures must be watertight. An ample concrete cover should be provided to protect the reinforcement bars against corrosion.

All bars should be placed far enough apart to allow the concrete to surround them completely. Loading stresses should be kept to a minimum. Any stresses developing in the concrete due to drying, shrinkage, temperature changes and differences in soil subsidence should be limited as far as possible by sub-dividing the building into a number of independent sections connected with watertight expansion joints. The concrete mix's cement content and the placing of the mix should aim at full water tightness and as little drying shrinkage during hardening as possible. A plaster finish should never be used. A good finish can be obtained by using smooth shuttering, for instance made of laminated wood. To prevent short-circuiting of the water flow along the walls of the filter box, the inside shuttering next to the filter bed should be made of unplanned planks placed horizontally. Whenever possible, the filters should be set above the highest groundwater table, if necessary on elevated land.

Numerous underdrain systems (popularly known as filter bottoms) have been developed in the past. But unfortunately many are either too expensive or unable to ensure an even distribution of the wash water over the full underside of the filter bed. The simple system that was described earlier, using perforated laterals, can be so constructed that a good wash water distribution is obtained. It has the added advantage that it may be made of locally available materials using local skills. Another good solution is the false bottom and strainer underdrain system. It consists of prefabricated concrete slabs, about $0.6 \times 0.6 \text{ m}^2$, placed on and anchored to short concrete columns.

The slabs are provided with holes, about 60 per square metre, in which the strainers are set. The slits in these strainers are narrow, about 0.5 mm, giving a sufficient resistance against the passage of the wash water for an even distribution of the water. This underdrain system allows the filter sand to be placed directly on the filter bottom with the strainers, and no supporting gravel layers are needed.

The work of a rapid filter is done by the filter bed and considerable attention should be given to its composition. Sand as filter material has proven to give excellent results. It is cheap and generally available and for these reasons widely used. For single-medium filter beds there is no reason to use other filter materials except in very special cases. To prevent a hydraulic classification during backwashing that would bring the fine grains to the top and the coarse grains to the bottom of the filter bed, filter sand that is as uniform as possible in size should be used. It should have a coefficient of uniformity less than 1.7 and preferably as low as 1.3. The requirements for grading filter sand are best given as maximum and minimum percentages of material that pass various sieves of standard mesh sizes. For a graphical specification, a diagram can be plotted as in figure 17.25.

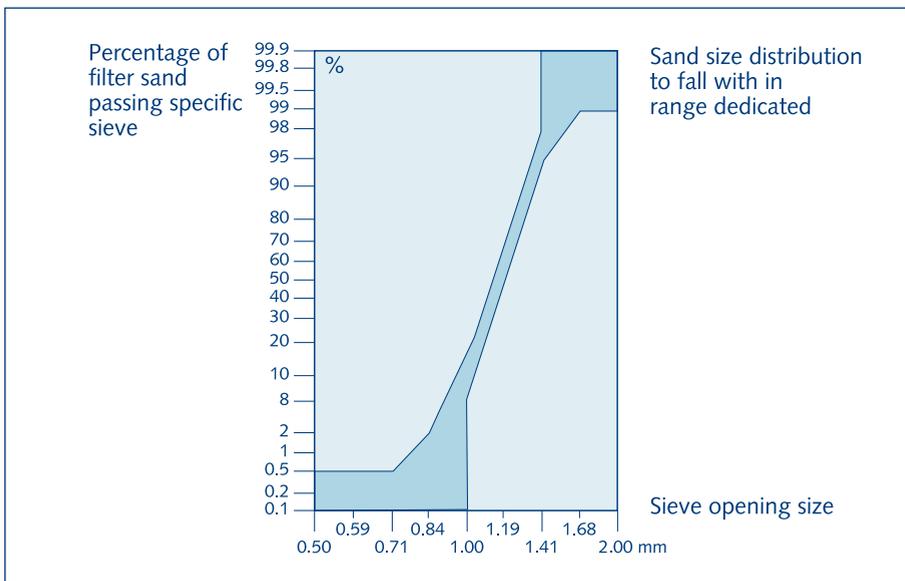


Fig. 17.25. Specification of filter sand for pre-treatment of river water

17.7 Village-scale rapid filtration

Because of their complex design and construction, and the need for expert operation, rapid filters are not very well suited for application in village-scale water treatment plants. This is especially true for their use as final filters in the treatment of turbid river water. The bacteriological safety of the filtered water then has to be secured by post-chlorination with all its associated difficulties. It would be better to use slow sand filters, which give a bacteriologically safe filtrate, but these may suffer from rapid clogging caused by the turbidity present in the raw water.

Suspended matter can be removed from raw water through various processes such as storage, coagulation and flocculation, and sedimentation. However, only rapid filters are able to produce clear water constantly with a turbidity of less than 5 NTU. This will

ensure the smooth operation of any following slow sand filters. There should be few objections against such an application of rapid filters. The use of rapid filtration for the removal of iron and manganese from groundwater also presents few problems, as the health hazard of possible contamination of the treated water will be small.

Assuming a water use of 40 litres/person.day, the required water filtration capacity for 10,000 people would be 400 m³/day or 40 m³/h for a 10-hour daily operating period. With a filtration rate of 5 m/h this calls for 8 m² filter bed area, which may be provided in three circular filters of 2 m diameter each (one filter as reserve). The underdrain system would probably best be made of perforated laterals (see section 17.4), covered with graded layers of gravel, broken stones or hard bricks chipped to the desired size. When coarse sand is available it should be graded using suitable sieves. Grading limits would be 0.8-1.2 mm for pre-filters; 1.0-1.5 mm for iron and manganese removing filters. For pre-filters the sand bed thickness should be taken at 1.0 m and for iron and manganese removing filters at 1.5 m. In the event that sand cannot be obtained, similar materials may be used, such as crushed stones, 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 backwashed for about half an hour to clean the filter material. The depth of supernatant water may be fixed between 1.5 and 2 m. The filter box will then have a total depth of 3.5-4 m.

The greatest difficulty encountered in village-scale rapid filtration is the backwashing process. It is uneconomical to use a wash water pump. In the example presented earlier a capacity of 100-200 m³/h would be needed, in duplicate, to allow for mechanical failures. Compared with the plant capacity of 40 m³/h, this is an enormous pump involving a considerable investment and high operating costs. With an elevated wash water reservoir of 20 m³ volume the pump capacity can be reduced to 10 m³/h, but the costs of the tank should be taken into account. For villages with low buildings, the pressure in the distribution system generally does not need to be more than 6 m. In these cases, a good solution will be to use an elevated service reservoir for backwashing the filters. No separate pumps would then be needed.

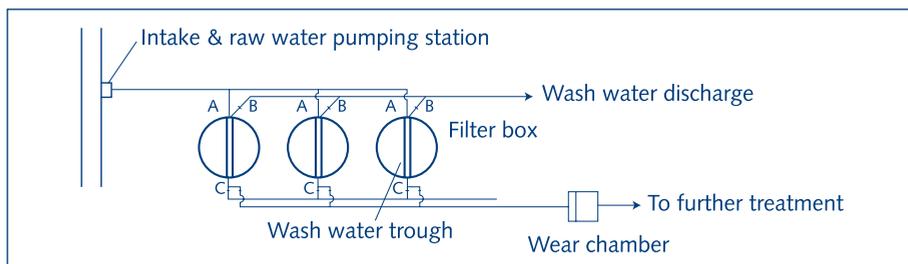


Fig. 17.26. General layout of a rapid filtration plant

The layout of the rapid filtration plant as described above is shown in figure 17.29. The raw water enters the filter through valve A and falls into the wash water trough to disperse the flow energy. The branch pipes into which valves A are set have a small diameter, giving sufficient flow resistance (e.g. 0.5 m of head) to assure an even distribution of the raw water over the individual filter units. The filtered water is discharged through valve D and passes over a weir placed in the weir chamber. The top of the weir is set so high that the lowest raw water level in the filter tank will be at least 0.2 m above the filter bed. Due to clogging, the level of the supernatant water will rise until it reaches the water pressure level in the supply pipe; no more water will enter the filter. The filter should then be cleaned by feeding the wash water through it and discharging it through valve B. The dirty wash water should be clarified by sedimentation after which it may be discharged back into the river, some distance downstream of the raw water intake.

17.8 Roughing filtration

Roughing filtration is often a desirable process to be used when the turbidity of the inflow to slow sand filters exceeds 25mg/l (approximately 20 NTU). Such pre-treatment protects the slow sand filters from becoming clogged in a few days, which is possible, particularly in monsoon periods. A bypass pipe can isolate roughing filters when they are not needed, so that they can be switched into service only when necessary.

Sometimes a more limited treatment than rapid filtration using a sand bed can be adequate for treating the raw water. This can be obtained by using gravel or plant fibres as filter material. In the chapter on multi-stage filtration theory, design and construction details have been given for roughing filtration using horizontal flow, downflow and upflow methods.

Coconut fibres have been used for filter material in filter units similar to sand filters. The filter bed is only 0.3-0.5 m thick and the depth of supernatant water about 1 m. The filter is operated at rates of 0.5-1 m/h, which gives a length of filter run of several weeks. To clean the filter it is first drained, after which the coconut fibres are taken out and discarded. The filter is repacked with new material that has previously been soaked in water for 24 hours to remove as much organic matter as possible. Coconut fibre filters appear to be able to cope with considerable fluctuations in their loading while producing an effluent of almost constant quality. There is a remarkably constant behaviour of the coconut fibre filters. The overall turbidity removal varies between 60 and 80%.

Another alternative is the pebble matrix filter, which consists of a deep bed (about 1-1.2 m) of pebbles, about 50 mm in size. About 0.8 m of the lower depth of the pebble bed is filled in with sand. The turbid water entering the top first encounters a 0.2-0.4 m depth of

pebbles and then passes through the sand within the pores of the pebbles. This partially clarifies the water, and as any sand-filled pore becomes clogged the flow diverts to passing over the adjacent pebble surface. The water transfers from pebble to pebble down through the filter, entering sand in a lower, less clogged pore. Pilot experiments have shown that turbid water containing up to 5,000 mg/l clay suspension can be clarified to less than 25 mg/l at flow rates of 0.7 m/h over a 20 hour run with a head loss of 1.5 m. Cleaning is accomplished by repeated drainage, and refilling from below with raw water, usually about three times sequentially. After about a week of such drainage cleaning, a thorough backwash is needed using drinking water supply. This should agitate and fluidise the sand in the pebble pores, with some expansion up into the pebble only layer. It is important that this expansion does not force the sand over the top of the pebble layer, as it is then difficult to put it back into the pebble pores below.

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18 Desalination technology

*Maria D. Kennedy, Ingrida Bremere and
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18 Desalination technology

18.1 The role of desalination in solving water scarcity problems

Currently about 470 million people live in regions with severe water shortages. The worst affected regions include northern China, northern Africa and the Middle East, northern India, parts of Mexico and the western United States. By 2025, the number of people living in water stressed countries is projected to climb to 3 billion – more than a six-fold increase (Cosgrove and Rijsberman, 2000).

Prudent use of available water resources can make them last longer, but countries with a per capita availability of less than 1000 m³/year (i.e. the water scarce countries) will need technological solutions such as seawater desalination and reuse of reclaimed wastewater to augment fresh water supplies in the future. Desalination, along with water reuse and water importation, can provide a means of increasing the supply of fresh water. Some experts predict that desalinated seawater will become an important water source in this century, as 70% of the world's population lives within 50 miles of the sea. Given the almost unlimited availability of seawater, desalination could provide sustainable water supply to many municipalities and industry (Postel, 2000).

Driven by scarcity, competition for water between municipal users and irrigation may increase dramatically (Macoun, 2000). For domestic water supply in large cities, desalination will become a realistic alternative to conventional drinking water treatment, when costs are comparable. Industrial development demands large amounts of clean water and desalination may be the technology of choice to develop industry in regions of water scarcity in Asia, Africa and South America.

Desalination in the world's regions

By the end of 1999 a total desalination capacity of about 26 million m³/d was installed or contracted worldwide, counting only plants with a capacity of more than 100 m³/d (Wangnick, 2000). The leaders and pioneers of desalination are found in the Middle East region, followed by North America and Europe (table 18.1). Roughly half of the current desalination capacity is met by desalination of seawater while brackish water accounts for a quarter of the capacity. More than 80% of the total seawater desalting capacity is produced by some form of distillation process, i.e. multi-stage flash (MSF), multi-effect (ME) or vapour compression (VC) systems. However, in recent years a preference for membrane plants has developed outside the Middle East. About 50% of the newly installed seawater desalination capacity is based on reverse osmosis technology. The remarkable growth in seawater reverse osmosis is due to the lower energy consumption, lower specific investment costs, shorter plant construction time and easy capacity extension of membrane systems compared with distillation systems.

Table 18.1 Desalination in the world's regions

World's region	Desalination in 2000 Total capacity million m ³ /d (%)	Seawater million m ³ /d (%)
Australia & Pacific Islands	0.1 (0.4)	Negligible
Asia	3.2 (13.3)	1.2 (8.5)
The Middle East	11.3 (47.1)	9.5 (67.4)
Africa	1.2 (5.0)	0.8 (5.7)
Europe	3.1 (12.9)	1.7 (12.1)
North America	4.3 (17.9)	0.3 (2.1)
Central America & Caribbean	0.6 (2.5)	0.5 (3.5)
South America	0.2 (0.9)	< 0.1 (0.7)
TOTAL:	24.0 (100)	14.1 (100)

Source: Wangnick, 2000

18.2 Desalination methods

Definition of saline water

Saline water is water that contains a significant amount of total dissolved solids (TDS). It is divided into three categories. Freshwater generally covers water with a TDS up to 1000 mg/l, brackish water from 1000 to 10,000 and seawater above 35,000 mg/l. In certain cases brackish water may contain 10,000 to 35,000 mg/l TDS and it is then referred to as "difficult" brackish water (Buros,1980). Saline water also contains small amounts of organic matter and dissolved gases but the majority of dissolved materials are inorganic salts.

Desalination methods

Desalination technology can basically be divided in two types: thermal desalting technology and membrane desalting technology. Thermal desalting technology comprises multi-stage flash distillation (MSF), multi-effect distillation (MED) and vapour compression (VC), while membrane technology includes electrodialysis (ED) and reverse osmosis (RO). In all types of desalting technology, saline water is separated into two streams: a freshwater stream with a low salt concentration and a brine or concentrate stream with a high salt concentration. Both types of technology require energy to operate.

While both distillation (MSF, MED or VC) and membrane processes (RO) are widely used for seawater desalination, RO is also applied for brackish and low salinity water. ED is only suitable for fresh or brackish water. The energy consumption of membrane processes depends on the salinity of the raw water, whereas in distillation processes the (thermal) energy consumption is constant and does not depend on feed water salinity.

Energy consumption in RO and ED for brackish and low salinity water is much lower than in distillation processes. Recent innovations in seawater RO have reduced the energy consumption further. However, it is difficult to make a general statement that in seawater desalination one thermal or membrane process is better than another without in-depth information on site conditions and the specific application. In general, thermal systems are robust and have high tolerance for variable feedwater quality, while membrane systems have lower capital and energy costs but are sensitive to fouling.

Table 18.2 Technology choices for seawater and brackish water desalination

Feedwater TDS (mg/l)	Type of water	Most suitable technology
< 1000	Freshwater	Membrane technology (RO/ED)
1000 – 10,000	Brackish Water	Membrane technology
> 35,000	Seawater	(RO/ED) Distillation (MSF/MED/VC) & membrane technology (RO)

18.3 Thermal desalting processes

Thermal desalting processes rely on the use of thermal energy to heat seawater to its boiling temperature. Water and dissolved gases volatilise and thus evaporate on continuous heating, while salts do not. Water vapour produced during evaporation is condensed on a cold surface and pure water is produced. Producing water by this process requires a large amount of energy, equivalent to about 9 USD/m³ (at a crude oil price of 21 USD per barrel). These costs make the process very unattractive. That is why multi-effect distillation has been developed. In this process the heat of condensing steam is used to evaporate seawater. By repeating this process ten times or more, more distillate can be produced with the same amount of energy.

Multi-effect distillation (MED)

Although the installed capacity of MED is relatively small compared with MSF, a growth of 17% was recorded at the end of the 1990s. The complexity of MED plants and scaling problems hindered their real breakthrough. Since these problems have been solved by innovative design concepts and the use of very effective antiscalants, MED is gaining ground.

A schematic outline of an MED plant is presented in figure 18.1 and a plant in Mindelo, Cape Verde, is presented in figure 18.2. In the MED process, distillation takes place in a series of chambers (or effects) operating at progressively lower pressures, thus ensuring

that the temperature at which seawater boils is lowered in each subsequent effect. The heat exchanger tubes in the first effect are heated by steam from a boiler or from a steam turbine in a power plant. Cold seawater is either sprayed or otherwise distributed over the surface of the evaporator tubes in a thin film to promote rapid boiling and evaporation. Steam flowing through the tubes condenses (inside the tubes) into pure water. The seawater film, on the outside of the tubes, boils as it absorbs heat from the steam inside the tubes. The steam from the seawater is introduced into the heat exchanger bundle (tubes) in the next effect (Buros, 2000). This process is usually repeated in 8-16 effects.

Scale formation needs to be avoided, since this phenomenon reduces the heat transfer, which in turn results in lower production capacity and higher energy consumption. Scale formation is controlled by (i) the degree to which seawater is concentrated in the plant; (ii) the top temperature of the plant (today 65°-80°C is commonly applied); or (iii) adding sulphuric acid and/or antiscalants. In the early days of MED the plants suffered severely from scaling. This was reason for the development of the MSF principle. MSF plants were much less vulnerable to scaling. Nowadays MED plants have solved these problems adequately.

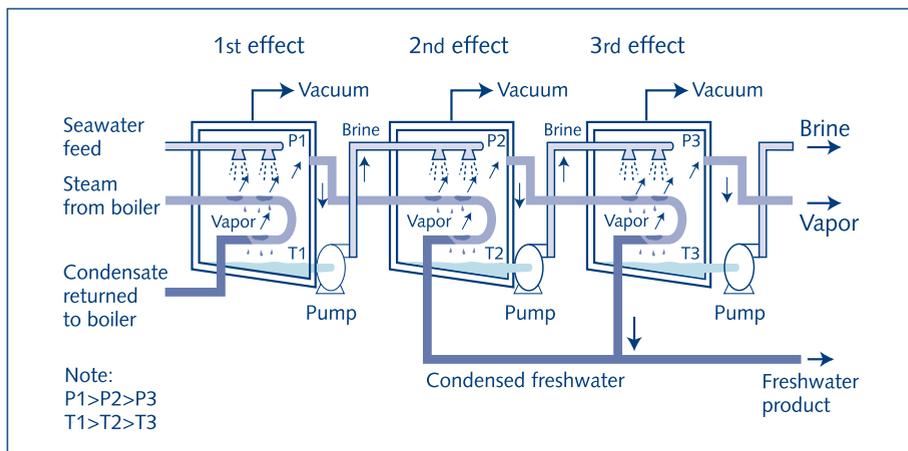


Fig. 18.1. Schematic diagram of a multi-effect distillation (MED) unit
 Source: Buros, O.K., 1980

In the MED process up to 15 tonnes of distillate can be produced per tonne of steam. To achieve this high thermal efficiency or *performance ratio* a relative large number of effects (chambers) and large heat exchangers are needed, resulting in higher investment costs.

Multi-stage flash distillation (MSF)

In the MSF process water is heated up to 110°C (max.), and flows subsequently through chambers (stages) of decreasing pressure (Fig. 18.3). As a result, the water flashes off to produce vapour. The vapour is condensed through a heat exchange with feed water. In

this way the evaporation (condensation) heat is recovered. In practice about nine tonnes of distillate is produced with one tonne of steam. Scaling problems were drastically less than with MED since water was evaporated in chambers and not at the hot surface of the heat exchangers. Higher performance ratios can be achieved when more chambers (stages) and more heat exchanger surface area are installed. Scaling is controlled by addition of sulphuric acid and/or antiscalant.



Fig. 18.2. Multi-effect distillation plant with vapour compression (MED +VC) in Mindelo, Cape Verde (capacity: 2400m³/day) (photograph by Jan C. Schippers)

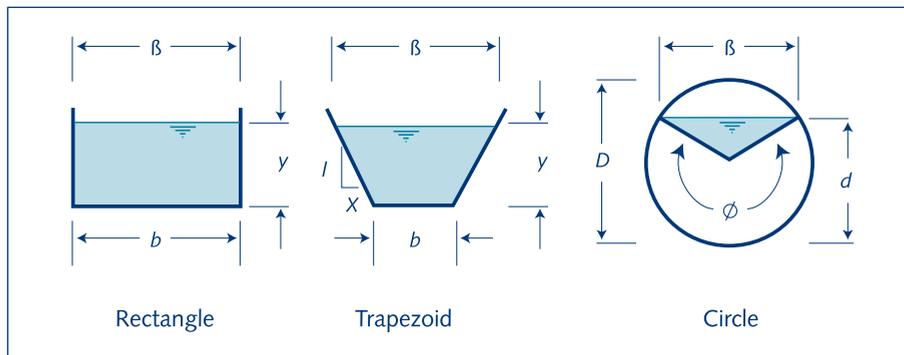


Fig. 18.3. Schematic diagram of a multi-stage flash (MSF) unit
Source: Buros, O.K., *The U.S.A.I.D. Desalination Manual*, 1980

MSF distillation systems were introduced in the early 1960s and dominated the seawater desalination market for several decades. However, gradually MED and in particular seawater RO are increasing their market share. An MSF plant in Las Palmas, Gran Canaria (Spain) is shown in figure 18.4. MSF and MED technology are commonly combined with power generation (dual purpose plants). This combination reduces investment and energy costs because no separate steam boiler is needed and low-grade steam from the steam turbines can be utilised.



Fig. 18.4. Multi stage flash (MSF) distillation plant, Las Palmas, Gran Canaria (capacity: $2 \times 9,000 \text{ m}^3/\text{day}$; commissioned in 1980) (photograph by Jose M. Veza)

Vapour compression distillation (VC)

Two types of vapour compression distillation are currently in use: mechanical vapour compression (MVC) and thermal vapour compression (TVC). The commonly applied mechanical vapour compression makes use of electrical power only, so no external steam is needed. The heat to evaporate the water comes from the compression of the vapour.

In an MVC system (Fig. 18.5), seawater, preheated in a heat exchanger by the outgoing streams of concentrate and fresh water, is sprayed onto the heat exchanger tubes. The seawater boils and partially vaporises. The vapour produced is drawn up into the compressor where it is compressed, a process that raises the saturation temperature. The vapour condenses inside the heat exchanger tubes and releases its condensation heat to evaporate the preheated and recycled seawater outside. The heart of any vapour compression system is the compressor. Mechanical vapour compressors are simple and robust, but the electrical energy consumption is relatively high. The capacity of vapour compression plants is limited by the size of the compressors. These units are typically used for small-scale application in remote locations. A mechanical vapour compression plant in Praia, Cape Verde is shown in figure 18.6. The specific energy consumption of the MVC is between 6.5 and 11 kWh/m^3 .

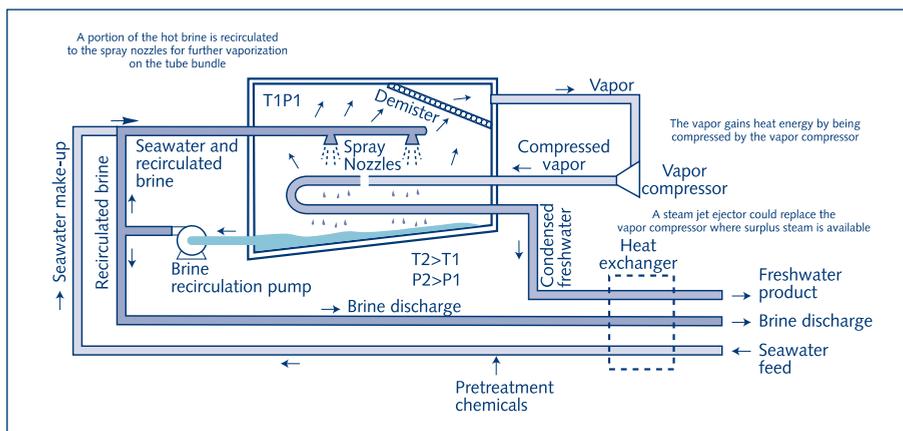


Fig. 18.5. Schematic diagram of a vapour compression (VC) unit
Source: Buros, O.K., 1980



Fig. 18.6. Mechanical vapour compression (MVC) plant in Praia, Cape Verde (capacity: $2 \times 1250 \text{ m}^3/\text{day}$) (photograph by Jan C. Schippers)

18.4 Membrane desalting processes

Electrodialysis

Electrodialysis (ED) became a commercial process in the 1970s and currently many small and medium-sized plants are operating all around the world. ED is considered to be a very suitable process for desalting brackish water. A major advantage of ED over reverse osmosis is the high recovery rate that can be achieved (up to 95%), which strongly reduces the volume of concentrate/brine that has to be disposed. For this reason, ED is very suitable for the desalination of inland brackish water sources, where the disposal of brine can be a serious environmental issue (IWACO, 2000).

The principle of electrodialysis

Electrodialysis is a process in which solutions are desalted by an electric current. Salts in water dissociate into positively and negatively charged ions. When electrodes connected to an external source of direct current (e.g. a battery) are placed in a container of salt water, electrical current is carried through the solution by the charged species (ions). The ions in solution tend to migrate to the electrode of opposite charge (i.e. positively charged ions (cations) such as Na^+ , Mg^{2+} migrate to the negatively charged electrode and vice versa). On its own this process is not very effective, due to back diffusion of the ions into the bulk. To overcome this problem, semi-permeable membranes are placed between the electrodes (Fig. 18.7).

When the raw water contains suspended/colloidal matter, then pre-treatment is applied to prevent clogging of the membranes.

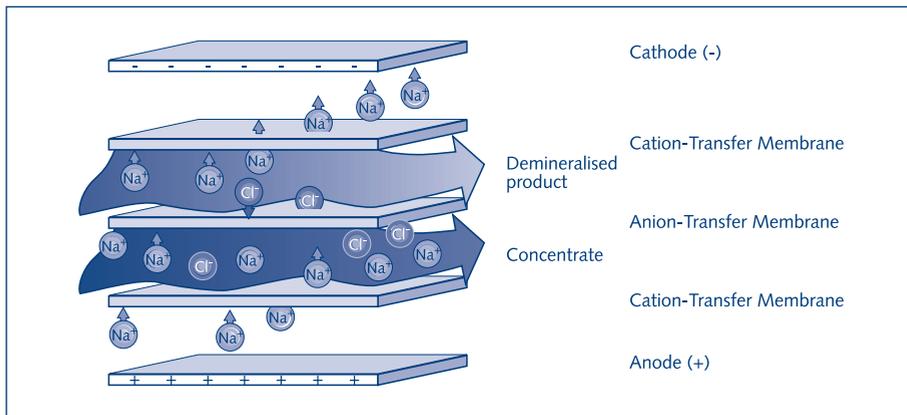


Fig. 18.7. The electrodialysis process
Source: www.ionics.com

Electrodialysis reversal (EDR)

One of the problems in all desalting processes is that the membrane and other active surfaces tend to become “scaled” over time due to the precipitation of sparingly soluble salts present in the feed water. The electrodialysis reversal (EDR) process was developed to prevent scaling and to reduce the addition of acid and antiscalant in electrolysis systems. By reversing the electrical current and exchanging the fresh (product) water and the concentrate (brine) streams within the membrane stack several times per hour, fouling and scaling constituents that build up on the ED membranes in one cycle are washed out in the next cycle. The reversal process is useful in breaking up and flushing out scale, slime and other deposits before they harden on the membrane. EDR is less vulnerable to fouling and scaling than RO.

Reverse osmosis

Reverse osmosis membrane technology is a more recent development than thermal distillation processes. Today reverse osmosis represents the fastest growing segment of the desalination market. For the first time, in 1998, more membrane plants (particularly RO) were contracted/built than distillation units and in 2000, 65% of all new plants were membrane systems (Pankratz, 2000). The strong growth in the membrane market is due to the lower energy consumption, lower specific investments costs, shorter plant construction time and easy capacity extension of membrane systems compared with distillation systems.

Principle of reverse osmosis

The heart of any RO system is a semi-permeable membrane that allows the fluid that is being purified to pass through it, while rejecting a high percentage of unwanted constituents. Reverse osmosis membranes are capable of rejecting bacteria, salts, sugars, proteins, particles and dyes. In the case of desalination of seawater and brackish water, the membrane is permeable to water but not to molecules of dissolved salt. An osmotic

pressure difference exists when solutions of different concentration are separated by a membrane permeable to the solvent (water) but not to the solute (salt). The pure water passes through the membrane to the salt water side in an attempt to dilute the salt water. The diffusion of pure water through a membrane continues until equilibrium is reached – a process known as *osmosis*.

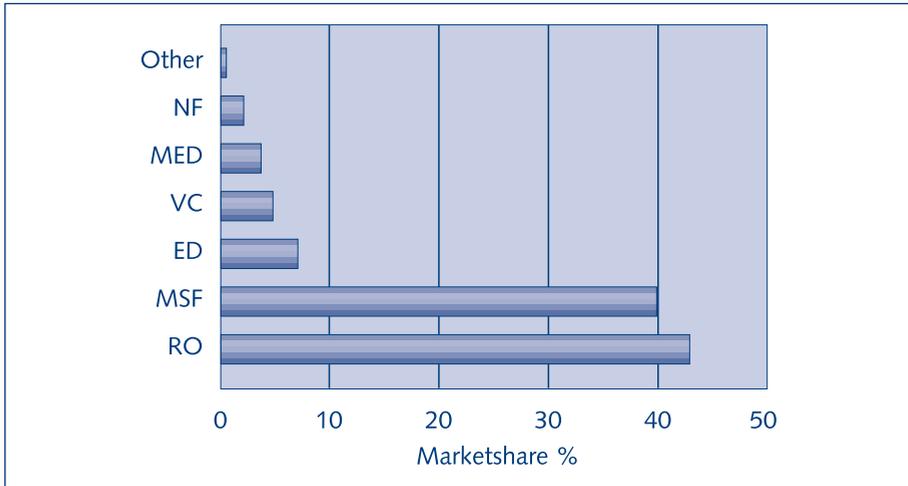


Fig. 18.8. Market share of thermal and membrane processes for desalination plants in operation
Source: Pankratz, T., 2000

Since the aim of most desalination systems is to recover fresh water from salt water, the natural osmotic flow has to be reversed by forcing the salt water through the membrane in the reverse direction. This can be achieved by applying a pressure to the salt water as it is fed to the system, creating a condition known as reverse osmosis. In reverse osmosis systems the dynamic pressure difference must be greater than the osmotic pressure in order to reverse the flow and force the water from the seawater side through the membrane to the pure water side. The permeate water flow through the membrane is proportional to the difference between the applied dynamic pressure difference across the membrane and the osmotic pressure difference, and proportional to the membrane permeability coefficient. Pressures range from 5-25 bar in the case of brackish water desalination and 50-90 bar for seawater.

Pre-treatment requirements in reverse osmosis systems

Pre-treatment of feed water is necessary to avoid fouling and/or scaling, which result in a lower permeability of the membrane and higher pressures required. When the applied pressure exceeds a maximum allowable level, chemical cleaning is needed. For greater operational efficiency, suspended and colloidal material must be removed and the water pre-treated to prevent particulate fouling, precipitation of salts (scaling) and growth of micro-organisms (biological fouling) on the membrane surface. Typically pre-treatment consists of filtration to remove suspended solids and dosing with acid or acid/antiscalant

to prevent precipitation. The need to remove suspended/colloidal matter, algae and biodegradable organic matter (biofouling) depends on the location.

Membrane devices and systems

The main component of any RO system is the membrane. RO desalination membranes are made from a variety of polymeric materials, e.g. aromatic polyamides, cellulose acetate and cellulose acetate derivatives. RO membranes are manufactured in two different element configurations - spiral-wound (fig 18.10) and hollow fibre.

A pump is used to pressurise the feed water; the operating pressure of the feed pump must be at least the osmotic pressure of the water. In practice the feed pressure is usually 2-3 times greater than the osmotic pressure. Typical feed pressures range from 8-25 bar for brackish water and 50-90 bar for seawater (Fig. 18.11).

An important parameter in the design of a reverse osmosis system is the recovery rate, defined as the ratio of permeate flow to feed flow. RO membranes plug instantly unless they are run in *cross flow* mode. Unlike traditional filtration, all the influent does not pass through the media. Rather it is split into a *permeate* (filtrate) stream and a *concentrate* stream, the latter flowing parallel to the membrane. Thus, cross flow splits the feed stream into two effluents: a purified stream and a stream more concentrated in solutes too large to pass through the pores of the RO membrane. During operation the feed solution becomes more and more concentrated and the osmotic pressure (and the salt concentration) increases as pure water permeates the membranes. In addition, the flow velocity at the feed side of the membrane is reduced due to the permeation of (pure) water through the membrane.

The possibility of membrane fouling and scaling is increased when the solute (salt) concentration increases and the flow on the feed side of the membrane is then reduced. Scaling of reverse osmosis membranes can occur during operation, particularly if the design recovery rate is set too high. Knowing the solubility of a given compound (salt), the maximum safe recovery rate (R) can be calculated to avoid scaling of the membrane during operation. The most likely components to scale RO membranes during operation are salts with a very low solubility, e.g. CaCO_3 , BaSO_4 , CaSO_4 and CaF_2 . Besides scaling, fouling of the membrane by colloids and other particles is also exacerbated by high recovery rates, as rejected material (colloids, etc.) may accumulate on or close to the membrane surface and eventually foul the membrane.

Generally, all membrane processes show a decrease in membrane flux (15-20% over a three-year period) and an increase in solute (salt) passage with time. To combat the unavoidable flux decline, RO systems are designed so that the operating pressure can be increased during operation to maintain the design flux of the system. The operating

pressure is usually increased (over a period of 1-3 months) until the operating pressure reaches the maximum design pressure of the system. At this point water production is stopped and part of the RO installation is cleaned using chemical cleaning solutions to restore the permeability of the membranes. This procedure is repeated until the entire installation is cleaned.

Table 18.3 Design parameters of seawater and brackish water reverse osmosis plants

Design parameter	Units	Typical value for dea water RO plant	Typical value for brackish water RO plant
Flux (Jw)	l/m ² .hr	9-16	20-31
Salt rejection (R)	%	98-99.8	93-98
Recovery rate (Y)	%	30-60	75-90
Operating pressure (P)	Bar	8-25	80-90

Source: IWACO, 2000



Fig. 18.11. Reverse osmosis (RO) racks and pump turbine system in La Aldea, Gran Canaria (capacity: 5000 m³/day; commissioned in 2001) (photograph by Jose M. Veza)

18.5 Energy consumption in desalination systems

Thermal desalination processes produce very high quality fresh water, but large amounts of thermal energy in the form of steam, and electrical energy (2-4 kWh/m³) are required in these plants. Thermal distillation plants are usually coupled to power plants where steam, used to drive turbines generating electricity, is available to provide thermal energy to evaporate seawater in the desalination process. The combination of distillation (MSF or MED) and power generation (dual purpose plant) is attractive from an energy point of view, as it can yield substantial savings in energy, provided there is a market for the electricity. In addition, no separate boiler is needed to produce steam, which reduces the investment costs.

From table 18.3 it can be seen that thermal energy consumption is roughly 16 times lower in a dual purpose (co-generation) plant than in a single stage MSF or MED plant. Nevertheless, the total energy consumption of distillation systems is still high compared with RO systems because thermal energy is not required in RO systems and the electrical energy requirement (3-6.5 kWh/m³) is similar to that of distillation systems (table 18.3). Significant improvements in energy recovery and in RO membranes have decreased the energy consumption to about 2.5-3 kWh/m³ in new RO plants and further reductions to about 2 kWh/m³ are expected in the future.

Table 18.4 Thermal and electrical energy consumption in distillation and membrane systems

Energy	Distillation technology		Membrane Technology (RO)
	MSF	MED	
Energy – Heat, MJ/m³			
• Single stage/effect system	2600	2600	-
• *Multi stage/effect system	290	290	-
• Co-generation power production	160	160	-
Energy – Electrical power (kWh/m³)	3.6	2.3	6.5-3.01

* Performance ratio = 9 in MSF & MED

The energy consumption of thermal and membrane desalination systems is compared in table 18.4. The electrical energy requirement of RO systems is converted into thermal energy in order to compare the energy consumption of RO with the MSF and MED processes. The thermal energy consumption in a dual purpose (co-generation) plant was used, which is lower than for a single or multiple stage distillation plant (table 18.3). Assumptions were made regarding the price of energy and the efficiency of converting electrical energy into heat energy (see footnote).

Table 18.5 A comparison of energy consumption of distillation and membrane (RO) systems

Energy	Distillation technology		Membrane Technology (RO)
	MSF	MED	
Energy – heat, MJ/m³			
• co-generation	160	160	heat energy not required
• electrical energy	25	16	45-21
Total energy consumption (MJ/m³)	185	176	45-21
Cost primary energy (USD/m³)	0.65	0.61	0.16-0.07

Assumptions: 1 kWh = 7 MJ (50% efficiency), Energy price: 3.5 USD /GJ (ca. 21 USD barrel of oil; 1 barrel = 160 L)

18.6 Desalinated water costs

The difference between the cost of desalinated water and that of conventional supplies narrowed dramatically in the 1990s. The drastic price drop caused the desalination market to explode – a 49% growth was recorded in seawater desalination in a three-year period at the end of the Millennium. Prices quoted for desalinated brackish water range from 0.2 to 0.35 USD/m³. (Wilf and Klinko, 1998) (Wade et al., 1999). While prices quoted for desalinated seawater are significantly higher, in the mid-1990s they have decreased to 0.5-0.8 USD/m³ for large membrane desalination systems (capacity ca. 40,000 m³/day) and 0.7-1.2 USD/m³ for large distillation systems. The costs are higher for small membrane desalination systems (e.g. < 100 m³/day) and range from 1.5 to 2.8 USD/m³ with and without energy recovery, respectively (Desalination & Water Reuse, Editorial, 1999).

Desalinated water costs are made up of energy, depreciation, monitoring and maintenance (particularly high for distillation systems), membrane replacement (reverse osmosis systems only), spare parts and process chemicals (Buros, 1980). Energy and

depreciation (capital costs) are responsible for about 70-80% of the costs of thermal and membrane distillation systems (table 18.5).

The recent decrease in the selling price of desalinated water is due in no small part to the advent of large-scale privatised desalination projects where financing and/or operating tasks are delegated to private enterprises (Pankratz, 2000). Technological improvements in energy recovery have dramatically reduced energy utilisation (from 5-2.5 kWh/m³) in seawater reverse osmosis (SWRO) systems. Membrane costs have also fallen in recent years while salt rejection and productivity (flux) have increased. Other factors such as lower specific investment costs, shorter plant construction time and easy extension of plant capacity of membrane systems have also helped to reduce costs. A new advancement in thermal desalination technology is the use of low-grade waste heat from industry to provide the thermal energy for desalination in Multi-effect distillation (MED) systems. In future, alternative energy sources (wind, solar) and hybrid plants using both thermal and membrane technology may increase the efficiency and flexibility of thermal plants and further reduce costs.

Table 18.6 Operational costs of distillation and membrane desalination systems (price level of 2000)

Cost component	MSF (USD/m ³) ¹	MED (USD/m ³) ²	RO (USD/m ³) ³
Energy – heat	0.24	0.24	-
– power	0.11	0.07	0.25
Operation & maintenance	0.13	0.13	0.06
Spare parts	0.08	0.08	-
Membranes	-	-	0.05
Chemicals	0.02	0.02	0.06
Capital	0.62	0.60	0.32
TOTAL COST (USD/m³)	1.20,	1.14	0.74

1 Electrical energy = 0.03 USD/kWh, Thermal energy = 1.5 USD/GJ, Plant lifetime = 25 years, Interest rate = 8%

2 Electrical energy = 0.03 USD/kWh, Thermal energy = 1.5 USD/GJ, Plant lifetime = 25 years, Interest rate = 8%

3 Energy requirement = 4.2 kWh/m³, Electrical energy = 0.06 USD/kWh, Plant Recovery = 45%

18.7 Small community water supply applications

Appropriate technology modifications using the principles of thermal and membrane desalting technologies have been developed in several countries. However, the small treatment plants are usually very inefficient in operation. Capital investments are very high, while production volumes are low. The economies of scale and the use of dual purpose plants, as applied in larger thermal desalting installations, do not often apply in rural and small town situations.

Nevertheless, small desalination package plants or simple desalination technologies may be considered in situations where sufficient fresh water cannot be found and water has to be supplied by trunk lines or tankers from far away. In such situations, desalinated water may be cheaper than what people presently pay for their scarce fresh water. To make the technology and service sustainable, the private sector will usually be involved in the investment, management and distribution. Before deciding to opt for desalination technology, all other water source options should be reviewed for long-term feasibility and sustainability. These possible options should include piped water from outside the area; tankering; rainwater harvesting; fog collection; using deep, fossil water; and mixing fresh water with saline water.

Membrane desalting technologies are not common in small community water supplies because of the required mechanical parts (pumps) and sensitive membranes. Recent membrane improvements and reduced unit prices may mean that the more reliable technology will become attractive for small systems. Hybrid RO systems have been developed using photovoltaic arrays or wind energy to drive the pumps and solar energy to heat up the water to reach a higher efficiency.

Thermal desalting systems are commonly based on solar power to raise the temperature of the saline water. Several different technologies have been developed, including

- Solar stills
- Hybrid solar stills
- Wood-fuelled stills

Solar stills

The Mexican Still is the most common solar still (Fig. 18.12). Solar energy is concentrated through (stainless steel) reflectors and magnifying glasses mounted around a distillation tank. This causes the saline water to boil (Ryan, 1996). It is a durable system, easy to construct but with a low production efficiency. The production ranges from 4-17 l/m². Investment costs are relatively high and, as the still is covered by glass sheets (better than plastic), it is prone to breakage.

In many countries with arid and semi-arid regions, modifications have been made to this principle, resulting in creative but not always efficient technical solutions.

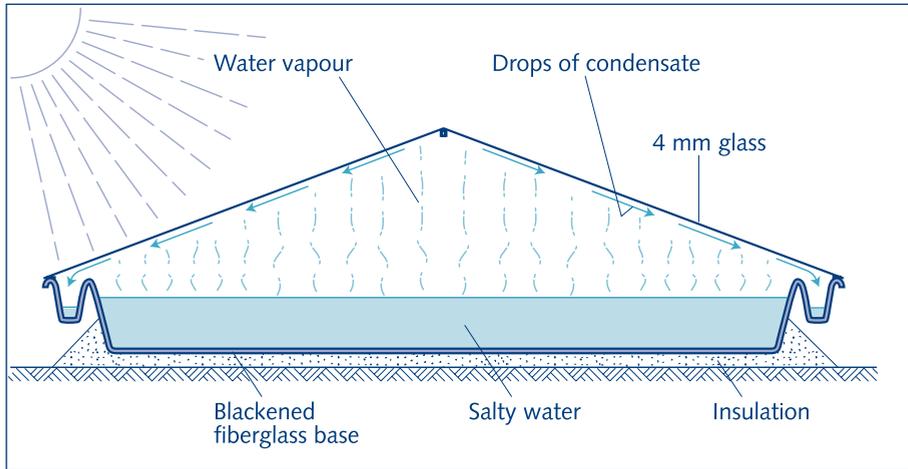


Fig. 18.12. Mexican Still
Source: Yates, Woto and Thage, 1990

Hybrid solar stills

Hybrid solar stills make use of the sun's energy to heat up the water and have a distillation tank in which the pressure is lowered through vacuum pumps driven by wind energy, photovoltaic arrays, diesel engines or electric motors.

Wood-fuelled stills

The most direct and simple but also the most inefficient desalting technique is the wood-fuelled still. Figure 18.13 shows three systems producing different volumes of water per hour. The Kudu-horn system has a production of about 0.7 l/h while the Ghanzi still produces about 33 l/h. (Yates et al., 1990). A major environmental concern is that saline water is common in arid and semi-arid areas where the availability of firewood is very limited.

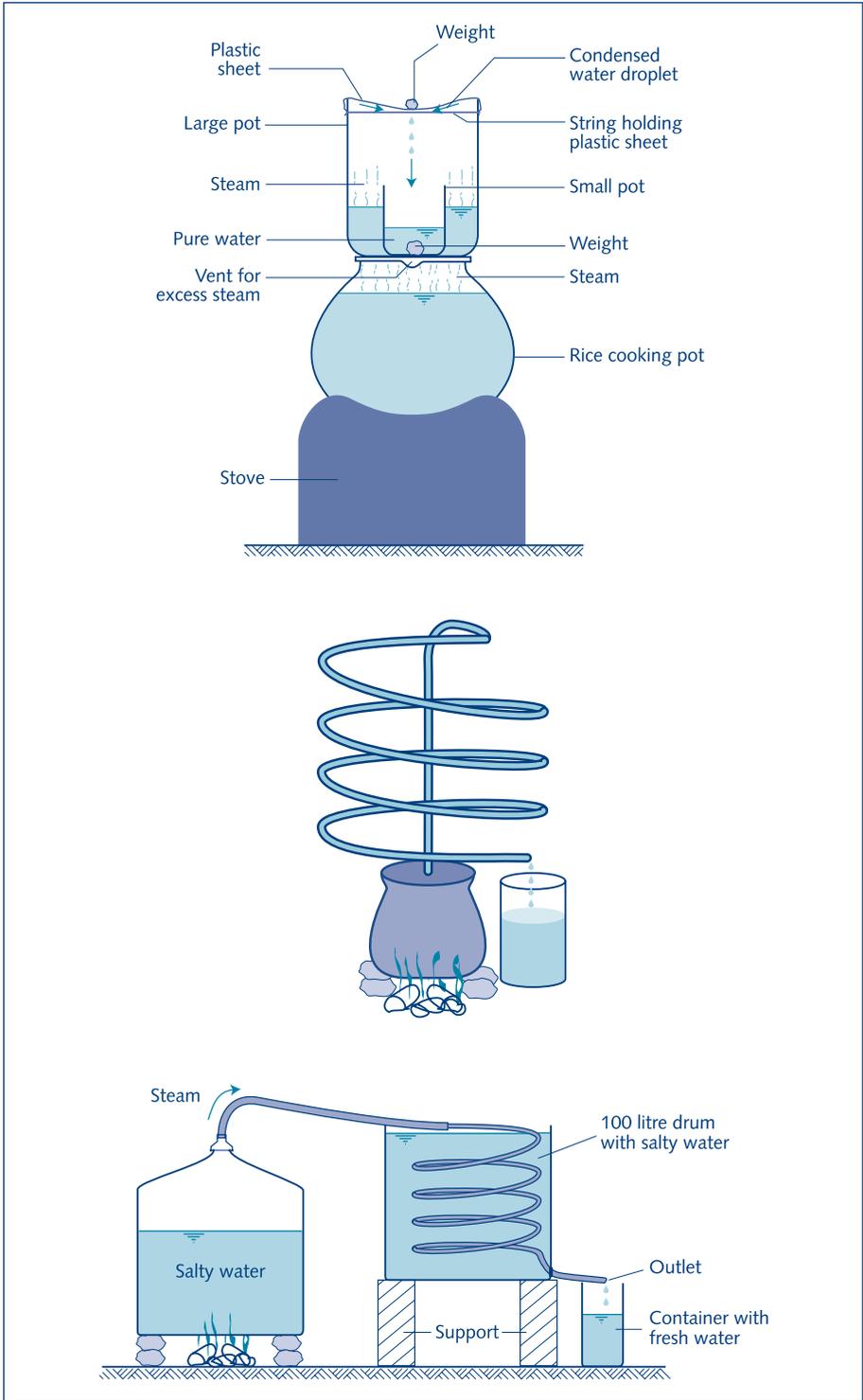


Fig. 18.13. Three wood-fueled stills
 Sources: Ryan, 1996 and Yates et al., 1990

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Web sites

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University of Hawaii: <http://www.hawaii.edu>

Desalination Directory, International Science Services: <http://www.desline.com>

The Middle East Research Center: <http://medrc.org.om>

IONICS: <http://www.ionics.com>

DOW: <http://www.dow.com>

19 Disinfection

Felipe Solsona

19 Disinfection

19.1 Introduction

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 and flocculation, and filtration reduce the bacterial content of water to varying degrees. However, these processes cannot assure that the water they produce is bacteriologically safe. Final disinfection will be needed. Disinfection means the destruction, or at least the complete inactivation, of harmful micro-organisms present in the water. It is considered the last barrier in water treatment and in cases where no other methods of treatment are available, disinfection may be resorted to as a single treatment against bacterial contamination of drinking water.

The following factors influence the disinfection of water:

- The nature and number of the micro-organisms to be destroyed. Certain organisms like parasites and viruses may not be destroyed or completely inactivated by disinfection.
- The type and concentration of the disinfectant used. Higher concentrations are correlated to higher efficiencies.
- The temperature of the water to be disinfected. The higher the temperature the more rapid the disinfection will be.
- The time of contact. The disinfection effect becomes more complete when the disinfectant remains in contact with the water longer.
- The nature of water to be disinfected. If the water contains particulate matter, especially of a colloidal and organic nature (turbidity), the disinfection process generally is hampered due to the "protection" of the micro-organisms by the turbidity particles.
- The pH of the water. Chlorine, for example, will have better disinfection power if working at pH below 7, as the chlorine compound that will prevail is HClO. At higher pH the chlorine compound present is ClO⁻, which has a lesser bactericidal power.
- Mixing. Good mixing ensures proper dispersal of the disinfectant throughout the water, and so promotes the disinfection process.

In small communities there are two possible ways of disinfecting water for human consumption. If the population is scattered, disinfection can be applied at household level. In communities with a higher population density, a "central" water disinfection system is more efficient.

As a classification of different disinfection methods, physical and chemical disinfection are discussed below.

19.2 Physical disinfection

At family level the two principal physical disinfection methods used are boiling of the water and solar disinfection. Ultraviolet radiation has been gaining rising acceptance for small community systems in developed countries, because of the reliability of the components and the declining costs.

Boiling

In some areas of the world this method may be expensive for the user (too much fuel consumption and work for women). Consumers usually do not like the taste of boiled water and it also takes a long time for the water to cool. Nevertheless, it is highly effective as a household treatment, as it destroys pathogenic micro-organisms such as viruses, bacteria, cercariae, cysts and ova. Boiling is normally carried out after education campaigns. In emergency situations, boiling of water may be used as a temporary measure. To enhance feasibility, promotion may focus on boiling water only for groups with the highest risks, such as infants and young children.

Solar disinfection

Solar disinfection (SODIS¹) works on a different principle to that of boiling. SODIS uses pasteurisation, which is based on the time/temperature relationship, to destroy pathogenic germs that may be present in the water. It has been observed that heating water above 62.8°C for 30 minutes or 71.7°C for 15 seconds is sufficient to remove waterborne bacteria, rotaviruses and enteroviruses from contaminated water. In addition, cysts of *Giardia lamblia* are inactivated during 10 minutes at 56°C.

Popular SODIS is performed by beaming sunshine onto transparent water containers with exposure times of several hours. This technique is very appealing, as it does not depend on conventional energy, is very simple, and uses either bottles or low-cost equipment. It is environmentally friendly and people accept it without difficulty.

Nevertheless, SODIS has never reached peak popularity, as there are too many variables that influence the efficiency and eventual safety of the treated water. Parameters that may interfere with a perfect disinfection include geographical latitude and altitude; season; number of hours of exposure; time of the day; clouds; temperature; type, volume and material of vessels containing the water; water turbidity and colour.

1 www.sodis.ch

The World Health Organization considers SODIS a valid option, but as a “lesser and experimental method”. Even so, for areas where there are no other means available to disinfect water, the method can substantially improve the bacteriological quality of water. The best results will be obtained when the measure is promoted and monitored by health officials or trained personnel (from a community-based or non-governmental organization – CBO or NGO).

Figure 19.1 shows both the batch and continuous solar disinfection systems. The continuous one comprises an exposure vessel or *reactor* and a tank where treated water exchanges heat with raw water (obviously without mixing).

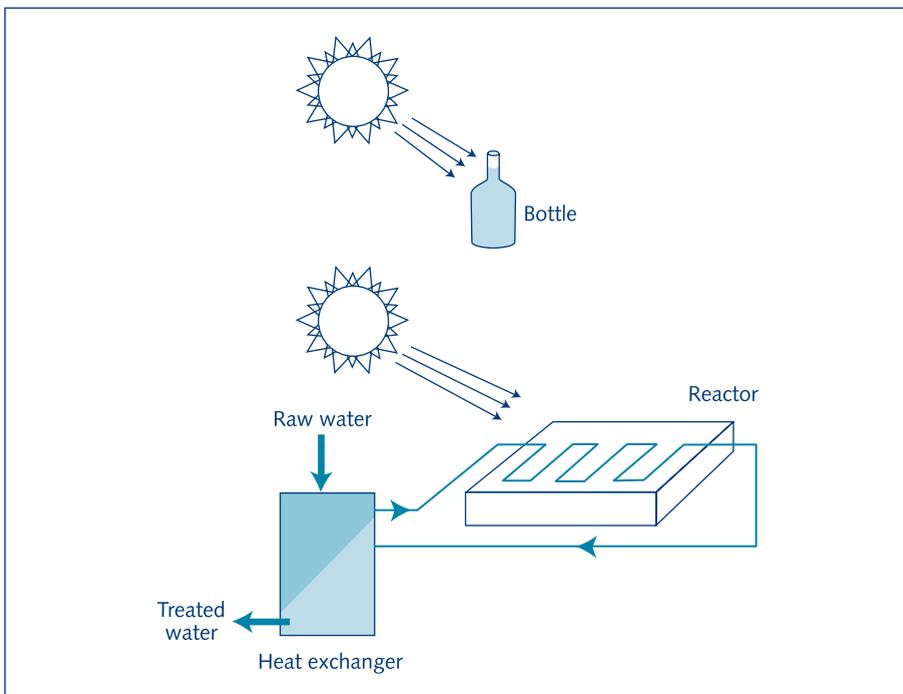


Fig. 19.1. Batch and continuous solar disinfection

Ultraviolet radiation

Even though it is not popular in the Third World, the most practical physical method that can be used for water disinfection in a “central” water treatment facility at small community level is ultraviolet radiation.

In ultraviolet disinfection, short wave radiation strikes water flows at close distance. The UV radiation (or *UV light*) is produced by low-pressure mercury vapour lamps that resemble the popular fluorescent lamps. In fact, large companies that make standard fluorescent lamps also manufacture the UV lamps; consequently, lamps, ballasts and starters for the UV systems can be bought “off the shelf”.

Although it has been well known for decades, it is only recently that the technique of UV radiation has been widely acknowledged. The reasons are that the industry has achieved a great level of quality, durability and reliability in the production of lamps and that the equipment and operation costs have been drastically reduced.

The disinfection mechanism is the disrupting effect the short waves have on the genetic material (DNA) of micro-organisms and viruses, killing them in a very short time.

The most important parameters determining the efficiency of disinfection are:

- Wavelength
- Condition of the water
- Intensity of radiation
- Type of micro-organism
- Exposure time

The UV wavelengths range from 100-400 nm, but they are differentiated and called

- UV-A: 315 – 400 nm
- UV-B: 280 – 315 nm
- UV-C: 100 – 280 nm

The germicidal portion is the UV-C. The commercially available low pressure mercury vapour lamps emit at 254 nm, a wavelength that is strongly active for disinfection.

A typical UV equipment setup is shown in figure 19.2. The lamps can be either in direct contact with the water or cased in protective shields made of quartz.

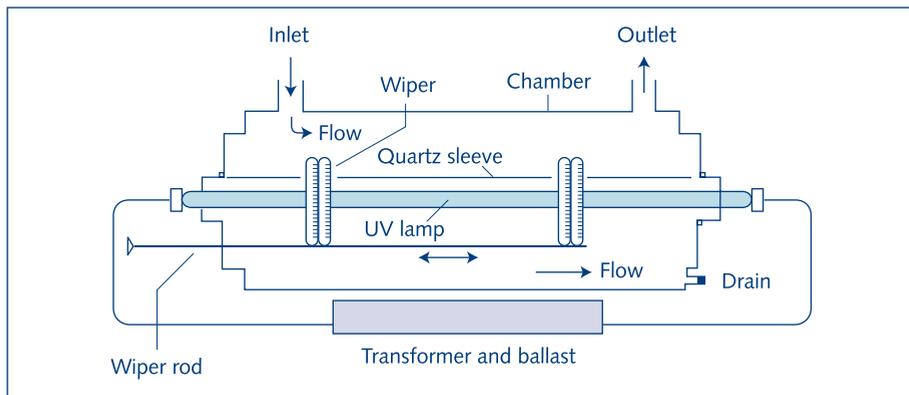


Fig. 19.2. A typical UV disinfection system

Deposits of calcium carbonates, silt, organic material or iron onto the bulbs or the sleeves can reduce the output and the germicidal power. Now, almost all systems have sleeve wipers that reduce the problem.

A modern UV disinfection system may include the following:

- A non-corrosive chamber that hosts the system
- UV lamps
- Mechanical wipers, ultrasonic cleaners or any other self-cleaning mechanism
- Sensors connected to alarm systems for monitoring UV intensity
- Safety shut-off in case of high or low flow rates, low lamp intensity or elevated temperatures of the lamp or system components
- Lamp-out monitors
- Electronic ballasts or current stabiliser

The condition of the water is linked with the effective intensity of radiation. If the water is clear, the intensity will be higher. If there are too many UV-absorbing elements like turbidity, organic matter, iron and/or manganese, these will shield the micro-organisms from the lethal rays. A clear (or as clear as possible) water is thus important to ensure a powerful effect. To have an effective disinfection process whereby the entire water body is properly reached by the rays, the water depth should not be more than 75 mm.

Ultraviolet radiation is measured in milli-Watt seconds per square centimetre (mWs/cm²) or micro-Watt seconds per square centimetre (μWs/cm²).

Obviously, different micro-organisms will have different resistance against the radiation. The dosage of UV light required to destroy most common micro-organisms (coliform, pseudomonas, etc.) is between 6000–10,000 μWs/cm². Standards for UV dosage in different countries range between 16,000 and 38,000 μWs/cm². As with any other disinfectant, the time of exposure is vital to ensure a good performance. It is not easy to determine the right contact time with accuracy (as this depends on the type of flow), but the period of time should be related to the needed dosage. Normal exposures are in the order of 10-20 seconds.

An advantage of the method is its simplicity of operation and maintenance. There is no need to buy and store any chemicals. It leaves no bad taste or odour in the water and carries no risk of overdose. It effectively destroys bacteria and viruses.

There are also limitations. The efficacy of UV light for the inactivation of *Giardia lamblia* or *Cryptosporidium* cysts has long been doubted by microbiologists. It is unsuitable for water with high levels of suspended solids, turbidity, colour or soluble organic matter. In those cases, pre-treatment of the water is needed. Also, unlike chlorination, UV radiation does not produce any residual that would protect the water against new contamination and that could also serve for control and monitoring purposes.

Ultraviolet light for disinfection is used in small communities in several developed countries but is rarely applied in developing ones. This may change in the coming years.

In brief:

- Pros**
- Simple process
 - Efficient process
 - No chemicals involved
 - Does not modify aesthetic characteristics of water
 - Can be managed by unskilled personnel
- Cons**
- Intermediate to high cost of equipment when compared with chlorine solution feeders
 - Water should be very clear. Needs electric power
 - There is no residual
- O & M tips**
- Checking should be done to ensure that there is no scaling on protective sleeves

19.3 Chemical disinfectants

Several chemicals, acting as strong oxidants, can destroy micro-organisms. Hydrogen peroxide and other metallic peroxides, lime, potassium and calcium permanganate, iodine, bromine, ozone and chlorine and its related compounds all fall into this category. Clean metals like copper, silver, mercury and zinc also disinfect, basing their action on a mechanism that is probably related to the absorption of the metallic ions by the organism, which in some way affects the chemistry of its cell structure.

It is not only important to have the potential to destroy germs. A good chemical disinfectant for use in developing countries should also possess the following important characteristics:

- Quick and effective in killing pathogenic micro-organisms present in the water
- Readily soluble in water in concentrations required for the disinfection
- Capable of providing a residual
- Not imparting bad taste, odour or colour to the water
- Not toxic to human and animal life
- Easy to detect and measure in water
- Not producing disinfection by-products (DBPs)
- Easy to handle, transport, apply and control
- Simple or “appropriate technology” devices for dosing
- Readily available in far away locations
- Low cost

Unfortunately there is not one disinfectant that complies with all of those conditions. Almost all of them fall into a category that could be called “far from complying”. Only a few may be called “almost complying”.

A brief description is given here for the most popular included in the first group: ozone, iodine, bromine, potassium permanganate and metallic ions. More detail is provided for the ones in the second group: chlorine and its compounds.

Ozone

Ozone, being a very strong oxidant, is effective in destroying organic matter and in eliminating compounds that give objectionable taste or colour to water. Nowadays it is used in several important water facilities in the industrialised countries. Like ultraviolet rays, ozone normally leaves no measurable residual, which could serve for monitoring the process or that may protect against new contamination of the water after its disinfection.

Disinfection by ozone, like chlorine, leads to the formation of both inorganic and organic DBP. The most frequently found are bromates, bromoform, bromoacetic acid, aldehydes, ketones and carboxylic acids, which are carcinogenic.

The high installation and operation costs, the need for continuous supply of power and the need for a proper operation and maintenance, do not make the use of ozone a recommended practice for small systems in developing countries.

Iodine

Iodine has attractive properties as a disinfectant. It has an effective bactericide and virucide power over a wide range of pH. In fact, iodine, unlike other halogens, becomes a more effective virucide as the pH increases. Iodine has been widely used for individual water supplies and for small batches of water. From farmers to hikers the use of iodine is popular either in solution, drops or tablets.

In spite of its attractive properties as a disinfectant, iodine has not gained widespread use in water treatment. One reason is that it is more costly (about ten times more expensive) than chlorine. Another may be that the use of iodine for water disinfection over extended periods of time has been seriously debated, as it may have physiological effects on iodine-sensitive people. The high volatility of iodine in aqueous solution is also a factor against its use except in emergency situations.

Bromine

Compared with chlorine and iodine, bromine is a more effective amoebic cysticide throughout the pH range. Its residual is more persistent than that of chlorine and, once in solution, it can be dosed easily with a diaphragm pump. On the other hand, bromine is not easily found everywhere it is a bit dangerous to manipulate, and it is more costly than chlorine. Little experience exists for its use as disinfectant for drinking water. For these reasons, bromine is not recommended for small water supplies.

Potassium permanganate

This is a powerful oxidising agent, and has been found to be effective against cholera vibrios but not for other pathogens. It leaves stains in the container and hence it is not a very satisfactory disinfectant for community water supplies.

19.4 Metallic ions

Several metallic ions (gold, silver, copper, mercury, etc) have germicidal properties called oligodynamia. Most of them, though, have drawbacks: gold is expensive; copper is good as an algicide but not so good as a bactericide; mercury is toxic. Silver seems to be the only one to have relatively good characteristics for water disinfection. It is not very toxic to human beings and the doses used in water treatment are very low, in the range of 20 - 75 micrograms/litre. Silver is added to treated water by dosing it from solutions or by direct electrolysis of silver or silver coated electrodes in the running water. Residual silver is not likely to decay easily, and it does not produce taste, odours, colour or DBPs.

Even though the bactericidal power of silver is important, it is not so quick as that of other disinfectants. It has also been found to be not a very good virucide. Besides, organic matter or other salts present in the water may hinder its activity.

Finally, silver disinfection treatment costs can be tens or even hundreds of times more expensive than low-cost disinfectants.

19.5 Chlorine and its compounds

Water disinfection by chlorination, massively introduced worldwide in the early 20th century, was perhaps the most important technological development in the history of water treatment. Filtration and chlorine disinfection of drinking water have contributed significantly to the increase in life expectancy in developed countries during the 20th century. In the USA, at the beginning of the new millennium, over 98% of water systems disinfected their waters with chlorine or chlorine-based compounds.

Even though chlorine and chlorine related substances are not perfect disinfectants, they have a number of characteristics that are highly valuable:

- They have a broad-spectrum germicidal potency and show a good persistence in water distribution systems. This means that they present residual properties that can be easily measured and monitored in networks or after the water has been treated and/or delivered to the users.
- Equipment needed for dosage is simple, reliable and low-cost. At village level, a number of "appropriate technology" feeders have proved to be easy to use, functional and accepted by local operators.

- Chlorine or chlorine-based products are easily found even in remote locations in developing countries.
- It is very economic and cost effective.

The most popular substances in the chlorine family are chlorine; chlorinated lime; high-concentration hypochlorites and sodium hypochlorite. They present different chlorine concentrations or *active chlorine*; which is a measure of their strength.

Active chlorine is the percentage by weight of molecular chlorine that would be rendered by a molecule of the compound. If, for example, a certain solution contains 10% of active chlorine, this is equivalent to 10 g of chlorine gas being bubbled (and totally absorbed) in 100 ml (100 g) of water.

Chlorine

Chlorine is a greenish yellow toxic gas found in nature only in the combined state, chiefly with sodium as common salt. It has a characteristic penetrating and irritating odour, is heavier than air and can be compressed to form a clear amber-coloured liquid. Liquid chlorine is heavier than water. It vaporises under normal atmospheric temperature and pressure. Commercially, chlorine is manufactured by the electrolysis of brine, with caustic soda and hydrogen as by-products. As a dry gas, chlorine is non-corrosive but in the presence of moisture it becomes highly corrosive to all metals except silver, titanium, gold and lead. Chlorine is slightly soluble in water, approximately 1 percent by weight at 10°C.

Chlorinated lime (Bleaching powder)

Before the advent of liquid chlorine, chlorination was mostly accomplished by the use of chlorinated lime. It is a loose combination of slaked lime and chlorine gas, with the approximate composition $\text{CaCl}_2 \cdot \text{Ca}(\text{OH})_2 \cdot \text{H}_2\text{O} + \text{Ca}(\text{OCl})_2 \cdot 2\text{Ca}(\text{OH})_2$. When added to water, it decomposes to give hypochlorous acid, HOCl. When fresh, chlorinated lime has active chlorine content of 33-37%. Chlorinated lime is unstable. Exposure to air, light and moisture makes the chlorine content fall rapidly. The compound should be stored in a dark, cool and dry place; in closed, corrosion-resistant containers.

High-concentration hypochlorites

These are not only twice as strong as chlorinated lime (60-70% available active chlorine content) but retain their original strength for more than a year under normal storage conditions. They may be obtained in small volume packages or in bulk. They are available in granular or tablet form.

Sodium hypochlorite

As a solution, sodium hypochlorite (NaOCl) usually contains 10-15% active chlorine in the commercial product. Household bleach solutions of sodium hypochlorite usually

contain only 3-5% available chlorine. It is the characteristics of chlorine and its compounds that have dictated the methods of handling and application in water disinfection practice.

19.6 Pre- and post-chlorination and disinfection by-products

In a water facility, chlorination is normally performed at the end of the treatment, after the filtration stage. This is sometimes called post-chlorination.

Pre-chlorination is sometimes applied prior to any other treatment. This is done for the purpose of controlling algae, taste and odour. In this case and when the raw water carries some organic materials (called *precursors*) it may give place to the production of disinfection by-products (DBPs). The most characteristic constituents of the DBPs are the trihalomethanes (THMs).

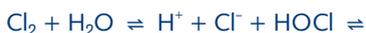
There has been some concern about THMs as some of them are carcinogenic. Though this is true, the risk of having widespread outbreaks of diarrhoeas and other water-related diseases due to the lack of disinfection largely outweighs the risk of having some cases of cancer. The WHO and the USEPA strongly recommend not jeopardising the microbiological safety of water in order to prevent eventual cases of cancer. Volume 1 of the *WHO Guidelines for Drinking Water Quality* states: "An efficient disinfection should never be compromised". Furthermore, the International Agency for Research on Cancer (IARC) in 1991 evaluated every available major scientific analysis of the potential health effects of chlorinated water, and concluded "chlorinated water is not a classifiable human carcinogen".

19.7 Chlorination in practice

Chemistry

Chlorination of drinking water is carried out in practice through the bubbling of chlorine gas or through the dissolving of chlorine compounds.

In the case of chlorine gas, the reaction that takes place is:



In the case of hypochlorites, the reaction that takes place is:



Or in the case of chlorinated lime (the portion that takes part in the reaction is $\text{Ca}(\text{OCl})_2$):



The hydrochloric acid (HCl) and the sodium and calcium hydroxides formed in the different reactions do not take part in the disinfection process.

The disinfectant species, the hypochlorous acid (HOCl); dissociates in water as follows:
 $\text{HOCl} \rightleftharpoons \text{H}^+ + \text{OCl}^-$

The pH of the water will govern the relative quantities of HOCl and OCl^- .

Both hypochlorous acid (HOCl) and hypochlorite ion (OCl^-) are present to some degree when the pH of the water is between 6 and 9 (the usual range for natural and drinking water). When the pH value of the chlorinated water is 7.5, 50% of the chlorine concentration present will be undissociated hypochlorous acid and 50% will be the hypochlorite ion. The different percentages of HOCl and OCl^- at different pH values can be seen in figure 19.3.

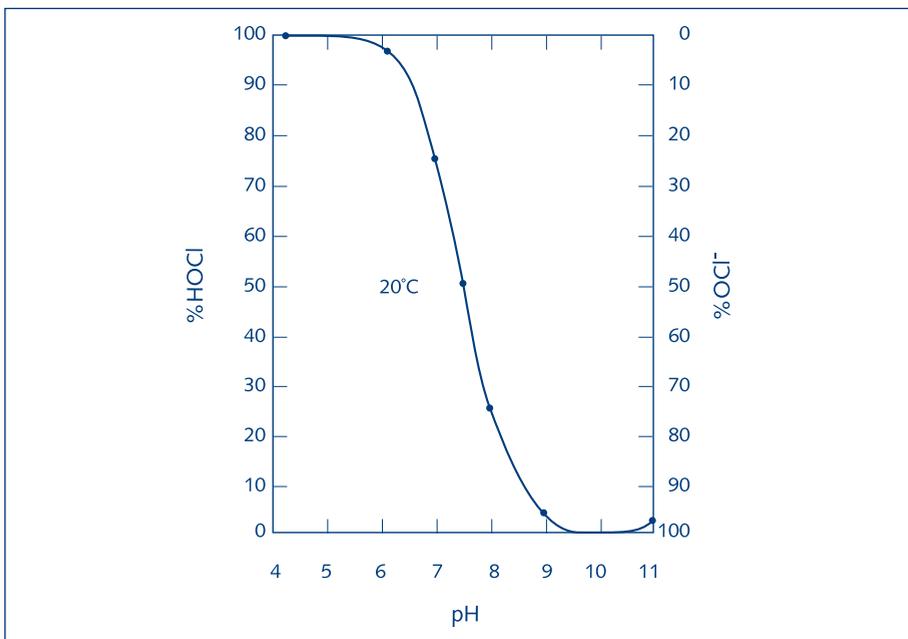


Fig. 19.3. Dissociation of hypochlorous acid versus pH

The different concentrations of the two species make a considerable difference to the bactericidal property of the chlorine, as these two compounds display different germicidal properties. In fact, the HOCl efficiency is at least 80 times greater than that of the OCl^- .

Therefore, in monitoring chlorine in water, the pH also needs to be monitored, as this will give an idea of the real bactericidal potential of the present disinfectant species.

Dose/Demand/Residual

The amount of chlorine added to the water is referred to as the *dose*, and is usually measured as the number of milligrams added to each litre of water (mg/l). The amount of chlorine destroyed in the reaction with the substances in the water is called the *demand*. The amount of chlorine (either free or combined) that remains after a certain contact time is known as the *residual chlorine*.

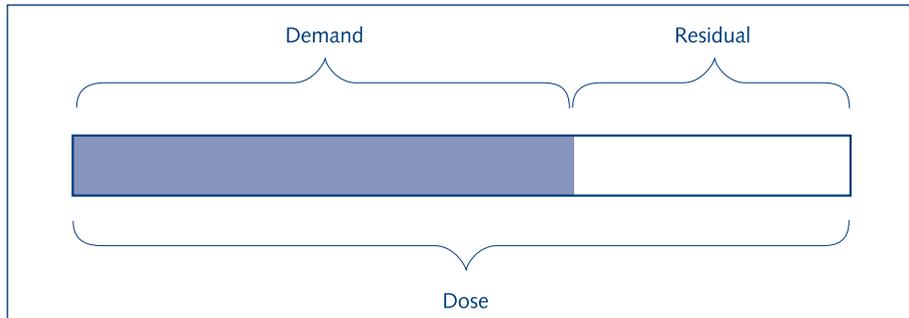


Fig. 19.4. Relation between dose/demand/residual

The graphic (figure 19.4) illustrates that, if the dose is correctly applied, the water will take up the demand in order to achieve full disinfection, and there will still be a residual left that will cope with any post-contamination (for example in the distribution network). The residual is also important as a check on the effectiveness of the dosing, as monitoring the residual will test whether the disinfection treatment has been complete or not.

When chlorine in the form of one of these compounds is added to water, a certain period of time is required for the chlorine to react with the micro-organisms and compounds in the water. This time is called the *contact time*, and a minimum of 30 minutes is usually recommended. The presence of the residual chlorine should be determined only after the specified retention time. If a 30 minutes retention time was set, then the monitoring should be done after that time has elapsed. This is what is called the C x T concept (concentration after a certain contact time).

WHO recommends the following conditions for a proper disinfection:

- Residual chlorine: ≥ 0.5 mg/l
- Contact time ≥ 30 minutes
- pH: < 8
- Turbidity: < 5 NTU; but ideally < 1 NTU

Several countries do not have, in their drinking water standards, an upper limit for residual chlorine in a distribution system. The WHO gives a guideline value of 5 mg/l. It is important that this value is not exceeded, as sometimes may happen in the first connections in the distribution network.

Determination of residual chlorine

Several methods are available to measure residual chlorine in water. Two of the simpler methods are presented here.

a) Diethyl-para-phenylenediamine method (DPD)

Free available chlorine reacts instantly with N-diethyl-para-phenylene-diamine producing a red coloration provided iodine is absent. Standard solutions of DPD-potassium permanganate are used to produce colours of various intensities. In this way, DPD can be used as a colorimetric method to indicate the concentration of residual chlorine. The colour produced by this method is more stable than that in the orthotolidine method.

b) Orthotolidine method

Orthotolidine, an aromatic compound, is oxidised in an acid solution by chlorine, chlorazines and other oxidants to produce a yellow coloured complex, the intensity of which is directly proportional to the amount of oxidants present. The method is suitable for the routine determination of chlorine residuals not exceeding 10 mg/l. The presence of natural colour, turbidity and nitrate interferes with the colour development. Due to the fact that orthotolidine has been demonstrated to be carcinogenic it is advised to handle the chemical with caution.

Application of chlorination

Household level

The most convenient chlorine compound for chlorination use in the home is sodium hypochlorite. Households may collect the disinfectant in liquid form from central distribution points and are taught to use it to disinfect the water in their own containers. Hypochlorite distribution projects have shown a good level of acceptability and success. The chemical can be brought from outside the area or it can be produced locally, for example in a clinic or school with on-site NaOCl generators.

These generators are electrolytic cells that produce a weak NaOCl solution from NaCl brine. The electric power can be obtained either from a normal source or from photovoltaic cells. The production and power consumption for these cells are between 1 and 2 g of chlorine per amp-hour. The concentration of the solution is between 5 and 7 g chlorine/litre (0.5-0.7%). They work on a batch system and some maintenance is needed to prevent electrode scaling.

This type of project can be very successful provided there is good information/education support. Also continuous monitoring from health officials or a community-based NGO is extremely important to ensure success.

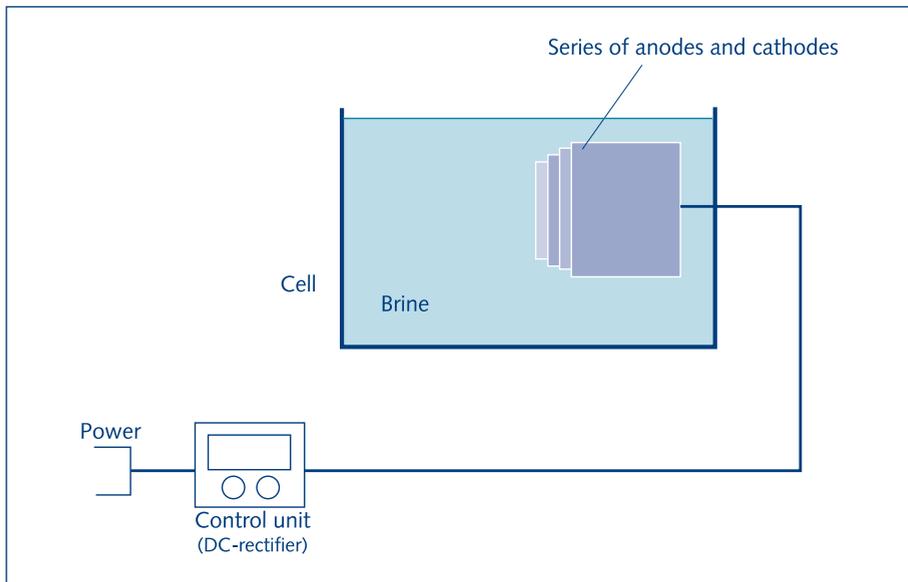


Fig. 19.5. On-site sodium hypochlorite generator

- Pros**
- Simple process
 - Good for spread rural population and also for small rural systems
 - Can be managed by unskilled personnel
 - Photovoltaic cells can provide the electric power
- Cons**
- Produces low concentration hypochlorite solution
 - Intermediate cost of equipment
 - Need for electric power
 - There is need for community education when used at household level
 - It should be monitored
- O & M tips**
- Care and proper maintenance should be in place to control the electrode scaling

Disinfection of small water bodies (dug wells, small reservoirs, etc.)

Dug wells are used either as individual or communal sources of drinking water. There are a number of devices that have been used for on-site disinfection (pot chlorinators, porous pot chlorinators, double pot systems, the Chinese plastic bag chlorinator, etc). However, proper disinfection of these sources is not easy because it is very difficult to have a constant disinfectant feeding rate.

The methods mentioned give users the false expectation of drinking safe water. When dealing with open dug wells it is better to adopt a disinfection program at household level, like the hypochlorite system described previously.

Disinfection of drinking water at community level

It is very important that a small community water system is designed for operation and management by the local people. This means that technology choice should be adapted to the local reality and to the cultural acceptability of the users.

There are a number of options available for disinfecting rural water supplies. Each has its pros and cons and issues that need to be discussed with the community members before a final choice is made.

Chlorine gas. Disinfection by gaseous chlorine is very economical and is the universally used technology around the world. More than 90% of the world population drinks water disinfected by chlorine gas.

The most commonly used gas system consists of a cylinder with the gas, a regulator with a rotameter (feed rate indicator) and an injector. The system operates under the vacuum created by water flowing past a venturi. A mixture of water and gas is injected at the application point, where the gas diffuses and dissolves.

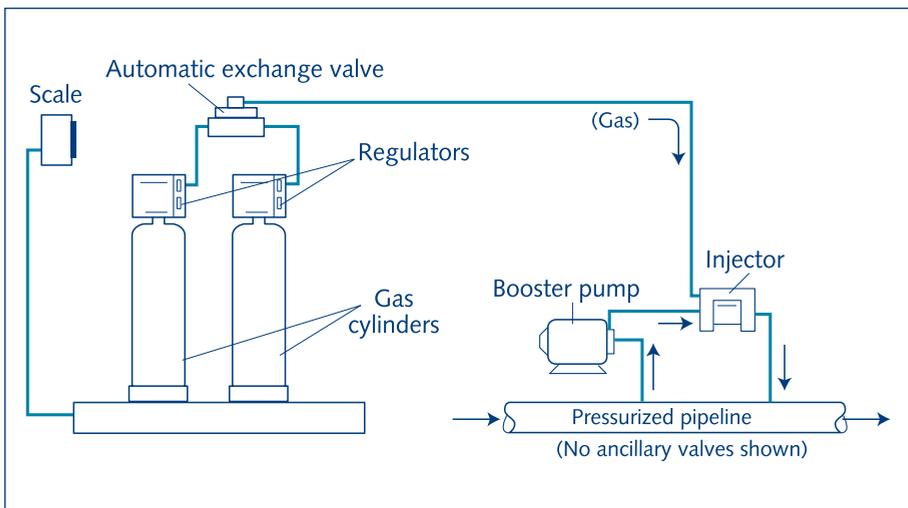


Fig. 19.6. A chlorine gas system

Even though the system is relatively simple, to ensure that the operation is safe and precise the personnel need proper training and several environmental safety precautions need to be in place (such as well-ventilated operating rooms, leak detectors, alarms, self-contained breathing apparatus, scales for gas cylinders, anchoring for the cylinders and safe gas transport systems).

In general the costs of any chlorination method are quite low. However, in many countries the operating costs of chlorination using chlorine gas are about 25-50% of the

cost of the equivalent solution of hypochlorite. But the capital investment costs required for gas chlorination, together with the precautions and training needs, make chlorine gas unfeasible for very small water supplies. As a rule of thumb it is considered that the gas should be used for communities with flows of water exceeding 500 m³/day. Today chlorine gas systems are reliable, but for conditions prevailing in small communities, hypochlorite disinfection may be more reliable and simpler.

- | | |
|-----------------------|--|
| Pros | <ul style="list-style-type: none"> • Widespread technology • Chlorine gas is produced in almost every country • Cheapest chemical • Most widely used in the world |
| Cons | <ul style="list-style-type: none"> • Costly system for very small villages • Needs ancillary equipment • Personnel need training • Can be dangerous if not properly operated • Not recommended for systems treating less than 500 m³/day |
| O & M tips | <ul style="list-style-type: none"> • Care should be taken on leaks • Personnel should be strict on safety regulations and always use protective equipment |

Chlorine solutions. All the other chlorine-based chemicals are liquid or can be dissolved and used as a solution. This is the most popular way to disinfect in the rural environment and small communities. It is simple, easy, and low-cost appropriate technology devices can be used.

In the feeding of the chlorine solution, different dosing systems can be applied. These dosing systems can be subdivided into atmospheric pressure and positive pressure systems.

Under the heading of atmospheric pressure, the most popular methods are the ones using the *constant head* principle and the *erosion system*. Devices used include wheel feeders, suction feeders, or just feeding by hand (batch method usually used when a community tank is filled and then open for consumption).

A *constant head system* retains a stock solution at a fixed depth in a tank from which it is fed through a regulating valve to the water to be disinfected. An important precaution to be taken is to ensure that the stock chlorine solution does not have precipitates that may clog the valves. It is suggested always to install a small filter (comparable to the gasoline/diesel filters) upstream of the regulating valves.

These systems are used to dose chlorine solutions in channels or in tanks.

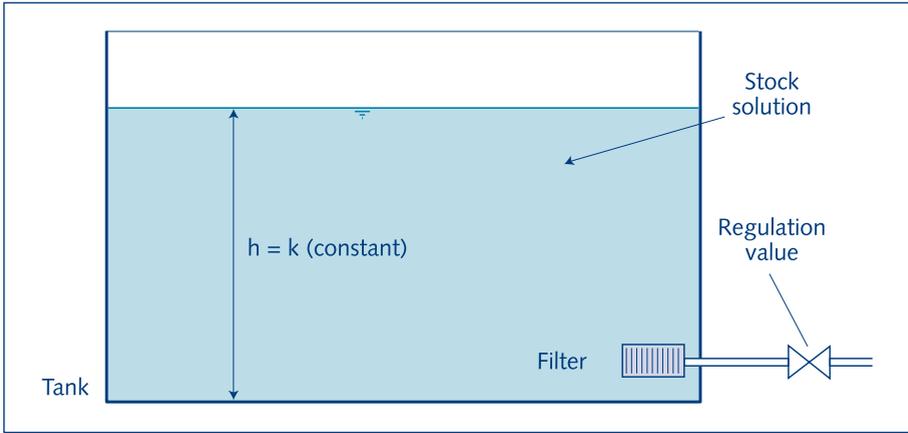


Fig. 19.7. Constant head system

There are a many such devices. Three of the most popular are shown here.

1. *Float valve in box system*

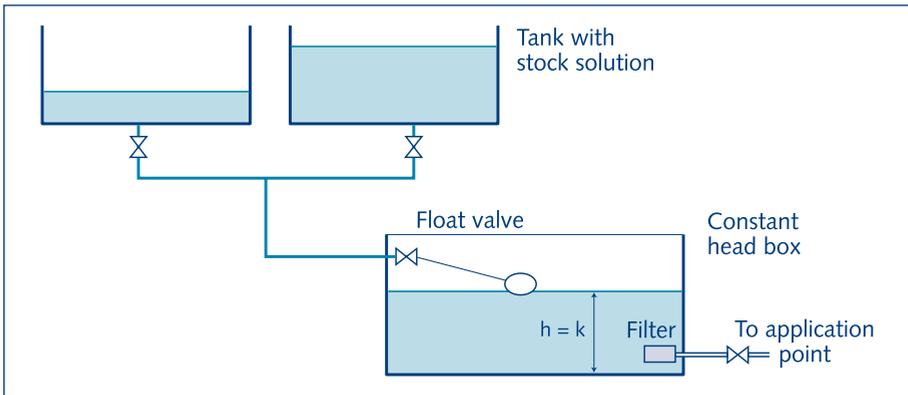


Fig. 19.8. Float valve in box

The heart of this system is a float valve, the same kind as used in toilet cisterns. One or two tanks hold the stock solution to be fed, and the float valve is placed in a small box. The system, although very simple, is cheap and accurate.

- | | |
|-----------------------|---|
| Pros | <ul style="list-style-type: none"> • Extremely simple principle • Very cheap • Can be manufactured locally • Reliable • Does not need electric power |
| Cons | <ul style="list-style-type: none"> • Error around 10% • Material may corrode |
| O & M tips | <ul style="list-style-type: none"> • Keep small orifices clean • Use filter to eliminate particulate matter or sediments |

2. Floating tube with hole system

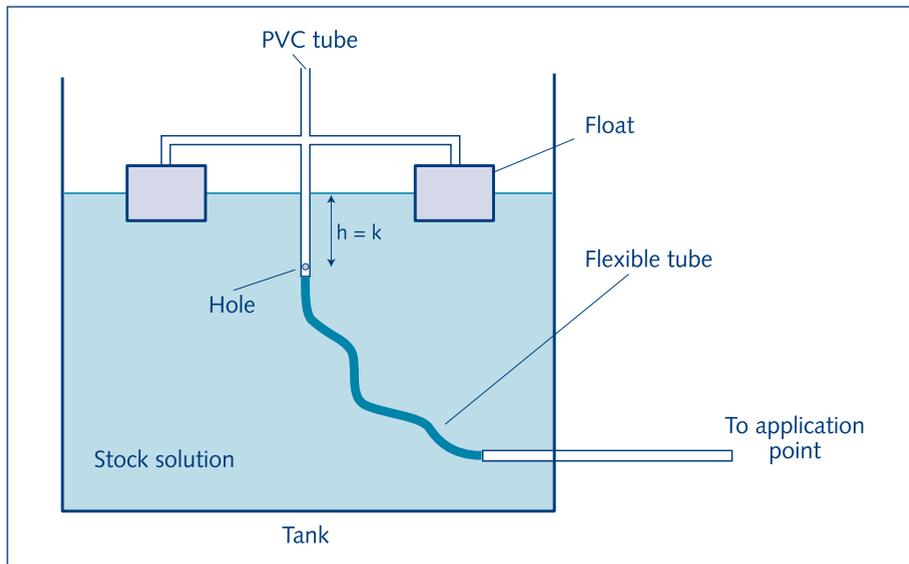


Fig. 19.9. Floating tube with hole

This, too, has been widely used in several different arrays. The basic element is a PVC tube with one or more holes. The tube is fixed to any kind of floating device and the hole/s should be placed some centimetres below the solution level. The tube is adjusted to such a level that the exact volume per second enters into the delivery tube and flows down to the application point.

- | | |
|-----------------------|--|
| Pros | <ul style="list-style-type: none"> • Extremely simple. • Very cheap. • Can be manufactured locally. • Popular. • Does not need electric power |
| Cons | <ul style="list-style-type: none"> • Depending on the way the system was built the dosing error may be up to 20% |
| O & M tips | <ul style="list-style-type: none"> • Keep small orifices clean. Use filter to eliminate particulate matter or sediments |

3. Bottle/glass system (developed in Argentina)

This system consists of a tank with the stock solution, a dosing element and a regulating valve.

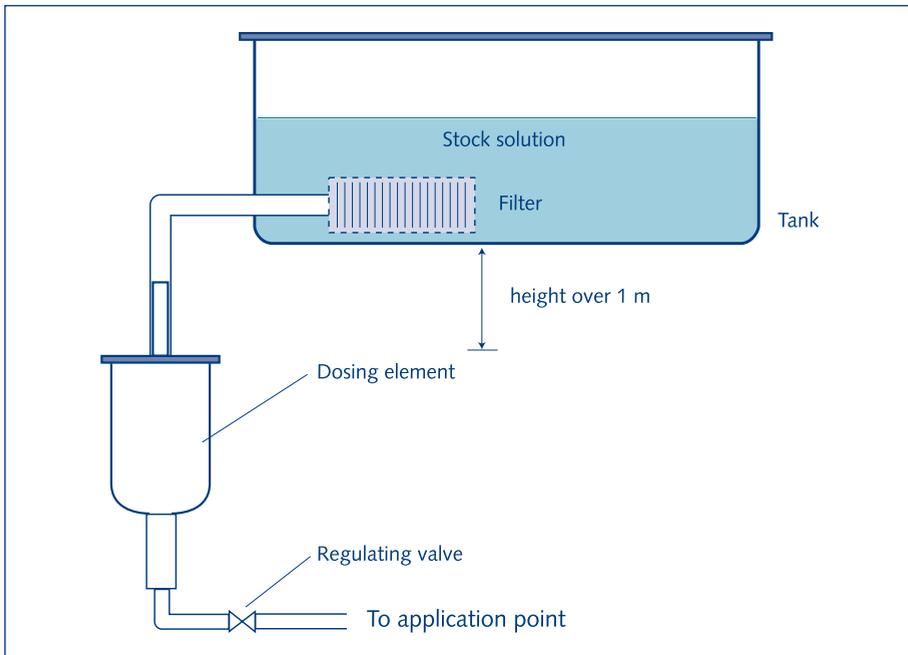


Fig. 19.10. Bottle/glass dosing element

The tank should be installed a metre (or more) above the level where the dosing element is placed. The dosing element is a simple system composed of a container with a floating device. It is made of a cylindrical plastic or glass bottle with smooth walls and with any volume between 0.5 and 1 litre. The bottle's base should be removed and the bottle inverted (the neck facing down).

On the upper part (area of the removed base) a small cover made of wood or plastic is glued with epoxy putty. This cover has two holes. In the central one, a $\frac{1}{4}$ " plastic tube or a piece of a discarded pen is introduced, protruding about 1 cm. This tube should be firmly welded or glued to the cover and its upper and lower edges should be levelled and smoothed. The second hole allows the air to flow freely.

An inverted plastic beaker is placed inside the bottle. On the external part of the base, a piece of soft rubber is glued. Because air is trapped in the beaker, it operates as a floating device to regulate the flow to the bottle and the liquid level in the bottle. The flow to the water tank is regulated with a simple valve.

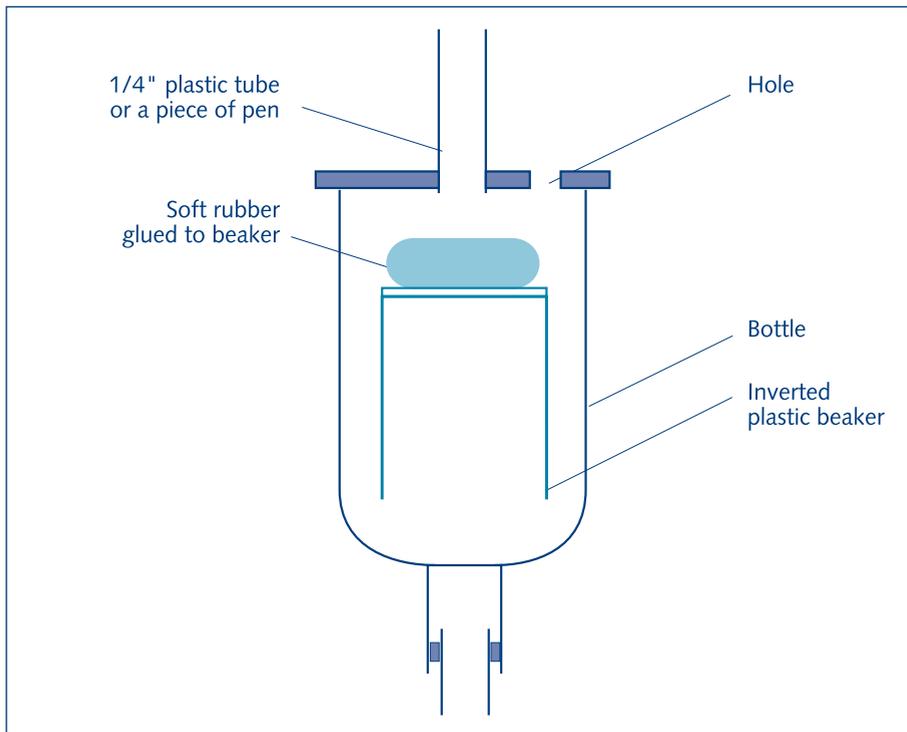


Fig. 19.11. Bottle/beaker dosing element (detail)

- Pros**
- Extremely simple
 - Very cheap
 - Can be manufactured locally
 - Ideal for small communities
 - Dosing error less than 10%
 - Does not need electric power
- Cons**
- Should be kept clean.
- O & M tips**
- Use filter to eliminate particulate matter or sediments

Erosion systems use tablets of high concentration hypochlorites. The system is commercially available and is inexpensive and durable. The systems are very easy to operate and maintain. The tablets can be bought from distributors or made locally. The tablets are safer and easier to handle and store than liquid or gas chlorine compounds.

Tablet erosion feeders take advantage of the rate of solubility of the hypochlorite tablets in running water. Tablets gradually dissolve at a predetermined rate as water flows around them, to provide the required chlorine dosage. As tablets are dissolved, they are replaced with new tablets fed by gravity into the chamber. The discharge from the feeder is a concentrated chlorine solution that is fed into a tank, an open channel or reservoir.

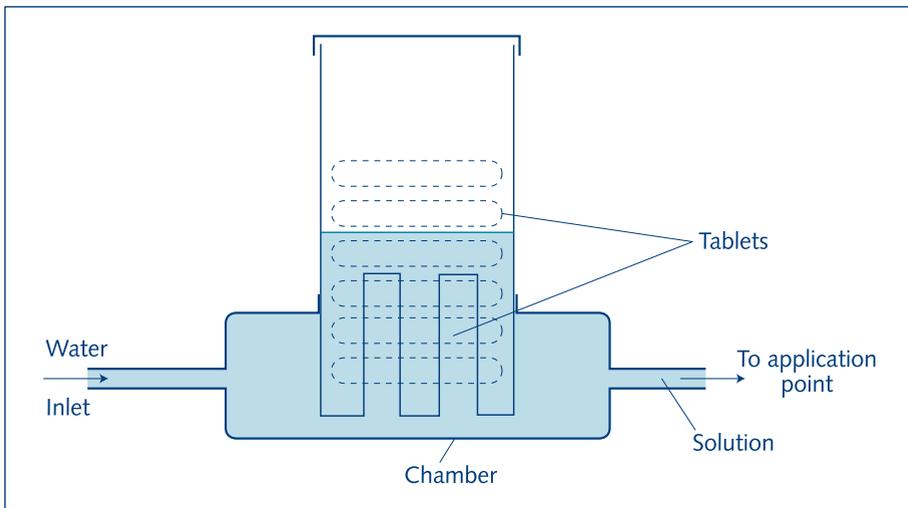


Fig. 19.12. Erosion feeder

- Pros**
- Extremely simple. Ideal for small communities
 - One of the best solutions for dosing at the entrance of a tank
 - Does not need electric power
- Cons**
- Intermediate cost
 - Dosing errors around 10%
 - Needs tablets
 - In some feeders the tablets (if locally produced) tend to get stuck or form caverns and do not fall in the dissolution chamber
- O & M tips**
- Make sure to use the proper tablets

Positive pressure feeders work on the principle that the chlorine solution is pressurised above atmospheric pressure and subsequently injected into a water pipeline. The most important positive pressure system is the highly popular diaphragm metering pump. These pumps are equipped with a chamber that has two one-way valves, one at the inlet and one at the outlet. The solution is drawn into the chamber through the inlet valve as the diaphragm opens, and is forced out of the chamber through the outlet valve as the diaphragm closes. An electric motor drives the diaphragm.

The task of the pump is to elevate the solution by means of a series of strokes. The application point may be a channel or reservoir (atmospheric pressure), but also a pipeline with running water (positive pressure).

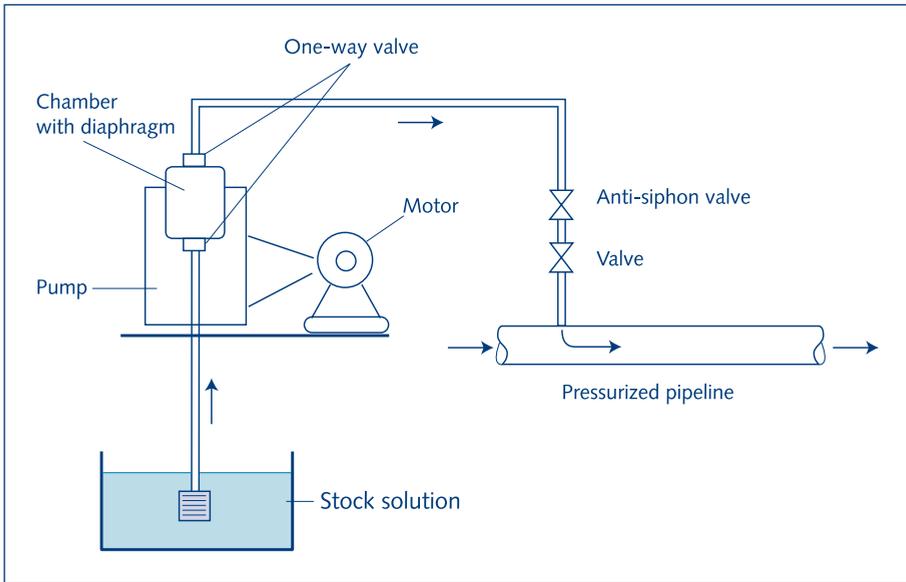


Fig. 19.13. Diaphragm pump system

- Pros**
- Highly reliable
 - Very popular
 - Simple to operate
 - One of the few systems to work under pressure
- Cons**
- Personnel should be trained in its operation and maintenance
 - Intermediate to high cost for a rural system
 - Needs electric power
- O & M tips**
- Give continuous and proper maintenance
 - Check the anti-siphon valves

A second way to inject a chlorine solution in a pressurised pipeline is shown in figure 19.14.

This system is very simple and economical, but if the mixing of the injected stock solution in the pipeline is not good, it may cause damage to the pump turbines.

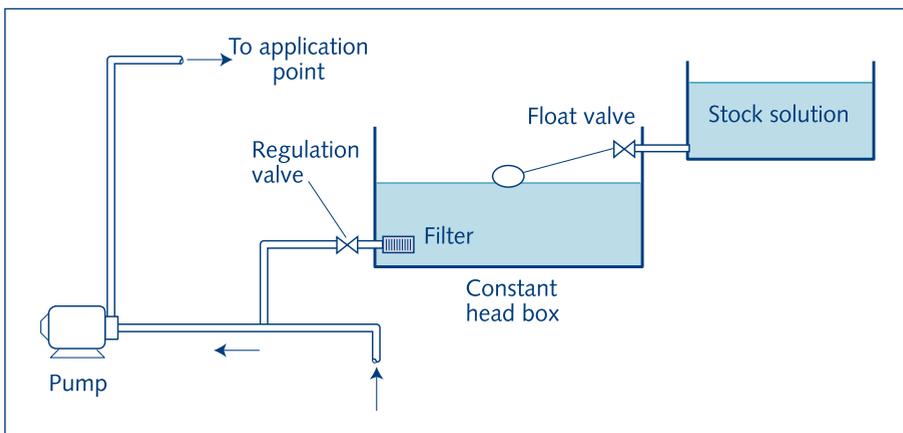


Fig. 19.14. Chlorination for pumped supplies

- Pros**
- Very simple
 - The cheapest solution for a feed under pressure
- Cons**
- It should be monitored
 - Sometimes there is corrosion in the pump rotor due to the chlorine
- O & M tips**
- Use filter to eliminate particulate matter or sediments

Disinfection of new tanks, pipes and wells

New tanks. All new tanks and reservoirs should be disinfected before they are brought into service. Similarly, tanks that have been out of service for repair or cleaning should also be disinfected before they are put back in service. Prior to disinfection, walls and bottom of tanks should be cleaned by sweeping and scrubbing to remove all dirt and loose material.

One of the disinfection methods used for a new tank is to fill it to the overflow level with clean water to which enough chlorine is added to produce a concentration of 50-100 mg/l. The chlorine solution is introduced into the water as early as possible during the filling operation in order to ensure thorough mixing and contact with all surfaces to be disinfected. After the tank is filled, it is allowed to stand, preferably for 24 hours but not for less than 6 hours. The water should then be drained out and the tank refilled for regular supply.

A second method, which is quite satisfactory and practical under rural conditions, is the direct application of a strong solution (200 mg/l) to the inner surfaces of the tank. The surface should remain in contact with the strong solution for at least 30 minutes before the tank is filled with water.

New mains and pipes. Whatever precautions are taken, distribution mains and pipelines are likely to be contaminated during laying. Therefore, they must be disinfected before they are brought into use. Distribution systems also need to be disinfected from time to time when they are contaminated by main fractures or floods.

Every pipeline should be cleaned by swabbing and flushing in order to remove all foreign matter. Immediately before use, the packing and jointing material should be cleaned and disinfected by immersion in a 50 mg/l chlorine solution for at least 30 minutes.

A practical means of applying chlorine solution for disinfection of rural water supply systems is to flush out each section to be disinfected. The intake valve is shut off and the section is allowed to drain. Then the discharge hydrant or valve is shut off and the section is isolated from the rest of the system. The disinfecting solution is fed through a funnel or a hose into a hydrant or opening made especially for this purpose at the highest part of the pipeline. Since air valves are usually placed at these high points, removing an air valve is often a convenient way to provide a point of entry.

Wells. New, rehabilitated or redeveloped wells should be disinfected before they are used for drinking water supply. Also wells that are suspected of being contaminated (e.g. in case of continuing poor well water quality or cholera outbreak, etc.) need to be disinfected. Better than regular disinfection is to find and correct the source and cause of the well contamination.

Before disinfecting the well water body, the walls of the lined well are cleaned by jetting chlorinated water (concentration 20 mg/l) and brushing to remove all dirt. Protect skin and eyes as chlorinated water is aggressive, and provide good ventilation or work short periods as chlorine gases are harmful.

Disinfection is done by estimating the volume of the well water body, and then adding sufficient chlorine to get a concentration of 50 mg/l. A contact time of 12-24 hours is needed. The chlorinated water should then be pumped out. Then the well is ready again for drinking water use.

19.8 Disinfection of water supply in emergency situations

A more detailed overview of water supply and disinfection in emergency situations is given in chapter 24. Long-term measures for the provision of safe water supply, aided by personal hygiene and health education, will greatly help to protect and promote public health. However, natural disasters like cyclones, earthquakes and floods do occur and sometimes result in complete disruption of water supplies.

While efforts are being conducted to put the systems back into operation, top priority also has to be given to providing the affected population with safe drinking water. While there is no single measure that is the panacea for all situations, the following may be useful to ensure a safe water supply depending upon local conditions and available resources.

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. In an emergency situation, if quantity is important, quality is mandatory. To achieve bacteriological safety, proper disinfection should be ensured.

Health officers are often confronted with a peculiar situation. It is not lack of disinfectants, but an excess that brings the problem. After a disaster strikes an area, it is usually flooded with a great variety of disinfectants. Normally these are chlorine-based compounds, but of different compositions and concentrations. It is then essential to have the knowledge to handle them properly.

Two suggestions

Firstly, it is important that the population never prepare their own stock solutions from high concentration hypochlorites. The user should be given a disinfectant solution ready to be used in a batch system (for disinfection of a family tank or container). Secondly, an ideal stock solution to be used in emergency situations is the one holding a concentration of 5000 mg of active chlorine/litre.

Health officers can prepare the stock solutions from any chlorine-based product using the following formula.

$$\frac{V_{\text{water}} \times C_{\text{stock}}}{C_{\text{product}} \times 10} = W_{\text{product}}$$

Where:

V_{water} = Volume of stock solution that will be prepared, in litres

C_{stock} = Concentration of stock solution intended (if as suggested, it is intended a 5000 mg/l concentration, then the value for C_{stock} should be = 5000)

C_{product} = Chlorine concentration in the product as specified by the manufacturer (in the formula, only the number should be placed, for example 65 when the chlorine concentration in the product is 65%)

10 = Factor in order that the result be given in grams of the product

W_{product} = weight (in grams) of product to dissolve

The disinfection dose that will be suggested to the population should be 5 mg/l in the moments of the extreme emergency, and then 2 mg/l under less stressing conditions. The minimum contact time should be 30 minutes.

The appropriate dilutions that people should prepare with the stock solution they are given is presented in the following table.

Table 19.1 Appropriate dilutions with stock solution

Volume of water to disinfect (litres)	Volume of stock solution (of 5000 mg/l) to add for a 5 mg/l final concentration	Volume of stock solution (of 5000 mg/l) to add for a 2 mg/l final concentration
1	20 drops = 1 ml	8 drops
5	100 drops = 5 ml	40 drops = 2 ml
10	10 ml	4 ml
20	20 ml	8 ml
100	100 ml	40 ml
200	200 ml	80 ml
1000	1 litre	400 ml

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CEPIS-PAHO/WHO: <http://www.cepis.ops-oms.org/>

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<http://www.disaster.info.desastres.net/>

EPA-Office of water: <http://www.epa.gov/>

International Water and Sanitation Centre IRC: <http://www.irc.nl/>

Loughborough University (Water, Engineering and Development Centre – WEDC):
<http://www.lboro.ac.uk/>

Minnesota Rural Water Association: <http://www.mrwa.com/>

The International Ultraviolet Association: <http://www.iuva.org/>

The University of New Hampshire: <http://www.unh.edu/>

Tripod (The World Water Project Ultraviolet Disinfection): <http://www.tripod.lycos.com/>

Water Health International. INC.: <http://www.waterhealth.com/>

Water Supply and Sanitation Collaborative Council: <http://www.wsscc.org/>

WHO: <http://www.who.int/>

SANDEC on solar disinfection: <http://www.sodis.ch>

20 Water transmission

Nemanja Trifunovic

20 Water transmission

20.1 Introduction

Water transmission frequently forms part of small community water supply systems; in that respect they do not differ from large schemes. The water needs to be transported from the source to the treatment plant, if there is one, and onward to the area of distribution. Depending on the topography and local conditions the water may be conveyed through free-flow conduits (figure 20.1), closed conduits (figure 20.2) or a combination of both (figure 20.3). The water conveyance will be either under gravity or by pumping.

Free-flow conduits are generally laid at a uniform slope that closely follows the hydraulic grade line¹. Examples of such conduits are canals, aqueducts, tunnels or partially filled pipes. If a pipe or tunnel is completely full, the hydraulic gradient and not the slope of the conduit will govern the flow. The hydraulic laws of closed conduit flows, also commonly called pressurised flows, apply in this case. Pressurised pipelines can be laid up- and downhill as needed, as long as they remain at sufficient distance below the hydraulic grade line, i.e. a certain minimum pressure is maintained in the pipe.

Free-flow conduits have a limited application in water supply practice in view of the danger that the water will get contaminated. They are never appropriate for the conveyance of treated water, but may well be used for transmission of raw water.

For community water supply purposes, pressurised pipelines are the most common means of water transmission. Whether for free flow or under pressure, water transmission conduits generally require a considerable capital investment. A careful consideration of all technical options and their costs and discussion with the community groups that will support and manage the system are therefore necessary when selecting the best solution in a particular case.

Routes always need to be checked with community members as well to make use of local knowledge and ensure cultural acceptability (technically desirable routes may, for example, run through a burial site or be unacceptable for other local reasons).

1 Its altitude and the water pressure in it define the piezometric head of each flow cross-section. The hydraulic grade line connects elevations of the piezometric heads and represents the potential energy of the flow. The slope of the hydraulic grade line is called the hydraulic gradient. For free-flow conduits it is the slope of the water surface. For closed conduits the hydraulic grade line slopes according to the head loss.

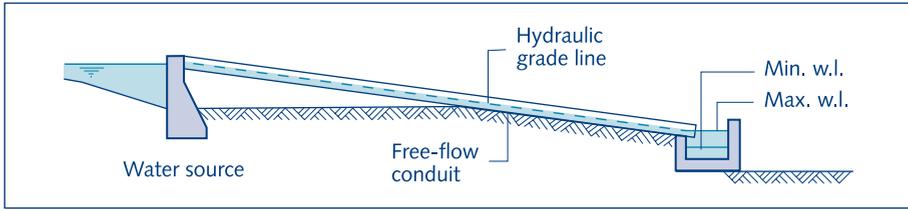


Fig. 20.1. Free-flow conduit

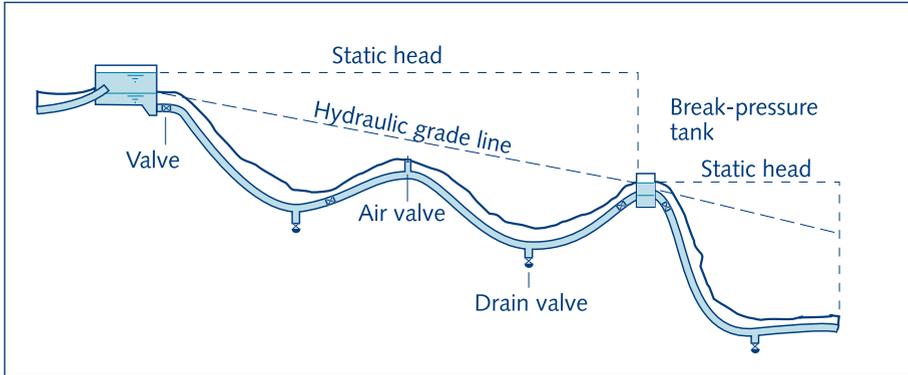


Fig. 20.2. Closed conduit

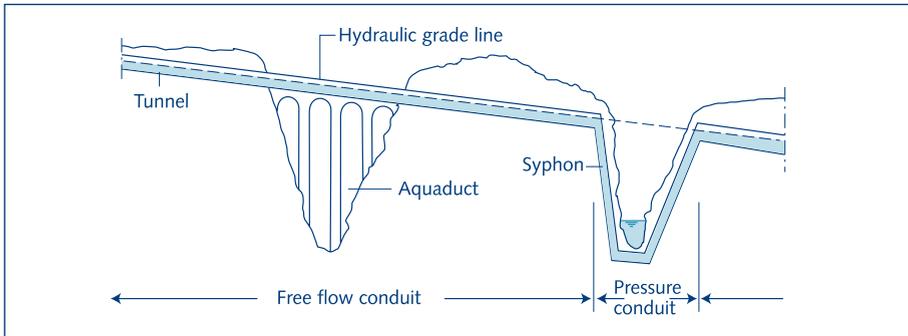


Fig. 20.3. Combined free-flow/closed conduit

20.2 Types of water conduits

Canals

Canals are laid in areas where the required slope of the conduit more or less coincides with the slope of the terrain. Generally they have a trapezoidal cross-section but the rectangular form will be more economical when the canal traverses solid rock. Flow conditions are assumed to be uniform if a canal has the same cross-section dimensions, slope and surface lining throughout its length.

Aqueducts and tunnels

Aqueducts and tunnels are constructed in hilly areas. They should be of such a size that they are approximately three-quarters full at the design flow rate. Tunnels for free-flow water transmission are frequently horseshoe shaped. They are constructed to shorten the overall length of a water transmission route, and to circumvent the need for any conduits traversing uneven terrain. To reduce head losses and infiltration seepage, tunnels are usually lined. However, when constructed in stable rock they require no lining.

Free-flow pipelines

Free-flow pipelines are used for transport of smaller quantities of water than tunnels. Compared with canals and aqueducts they offer better protection from pollution. Due to the free-flow conditions, simple materials may be used for construction. Glazed clay or concrete pipes should be adequate. Similar hydraulic conditions occur as for other free-flow conduits.

Pressurised pipelines

The routing of pressurised pipelines is much less limited by the topography of the area to be traversed, than is the case of canals, aqueducts or free-flow pipelines. A pressure pipeline may run up- and downhill and there is considerable freedom in selecting the pipeline alignment. Nevertheless, such pipelines often follow the topography quite closely, being buried at a similar depth for the length of the route. Also, a routing alongside roads or public ways will be preferred in order to facilitate inspection (for detection of any damage, leakage at pipe joints, faulty valves, etc.) and to provide ready access for maintenance and repair.

20.3 Design parameters

Design flow

The water demand in a distribution area will fluctuate considerably during a day. Usually a service reservoir is provided to accumulate and even out the variation in water demand. The service reservoir is supplied from the transmission main, and is located at a suitable position to be able to supply the distribution system (Fig. 20.4). Again, its site

needs to be chosen by the local people, based on technical advice and their own socio-cultural criteria. The transmission main is normally designed for the carrying capacity needed to supply water demand on the maximum consumption day at a constant rate. All hourly variations in the water demand during the day of maximum consumption are then assumed to be evened out by the service reservoir.

The number of hours the transmission main operates each day is another important factor. For a water supply with diesel engine or electric motor-driven pumps, the daily pumping often is limited to 16 hours or less. In such a case, the design flow rate for the transmission main as well as the volume of the service reservoir need to be adjusted accordingly.

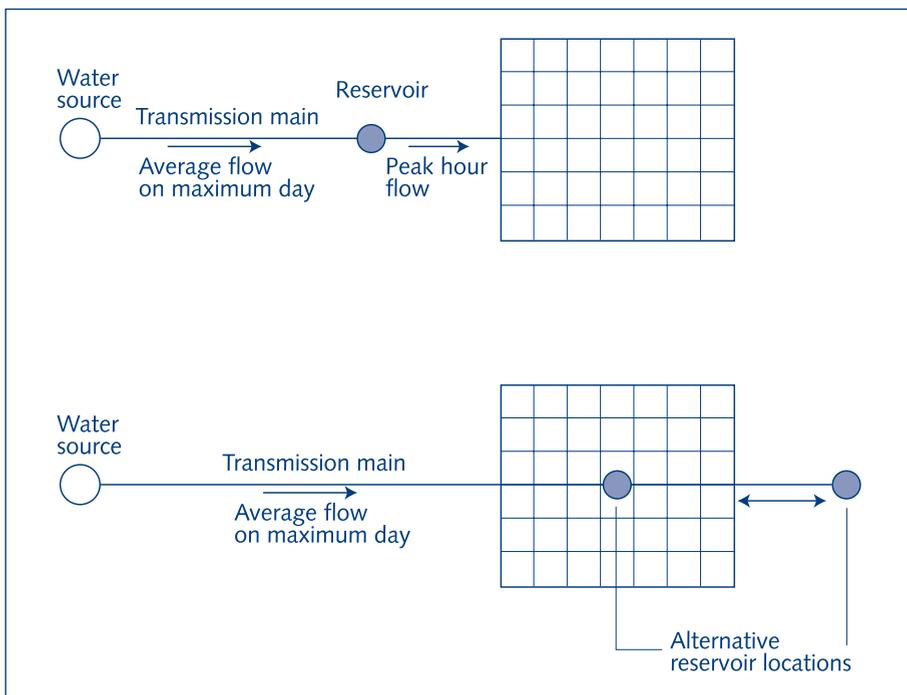


Fig. 20.4. Transmission main and service reservoir (schematic)

Design pressure

Pressure as a design parameter is only relevant for pressurised pipelines. Consumer connections on transmission lines are rare, so the water pressure can be kept low, provided that the hydraulic grade line is positioned above the pipe over its entire length and for all flow rates. A minimum of a few metres water column is also required to prevent intrusion of pollution through damaged parts of the pipe or faulty joints. In fact, nowhere should the operating pressure in the pipeline be less than 4-5 mwc (metres water column).

High pressures in transmission pipes occur as a result of long distances or specific topography. During supply by gravity the maximum pressure does not occur under operating conditions. It is the static pressure when the pipeline is shut (Fig. 20.5). In order to limit the maximum pressure in a pipeline and thus the cost of the pipes, the route can be divided into sections separated by a break-pressure tank. The function of such a tank is to limit the static pressure by providing an open water surface at certain places along the pipeline. The flow from the upstream section can be throttled when necessary.

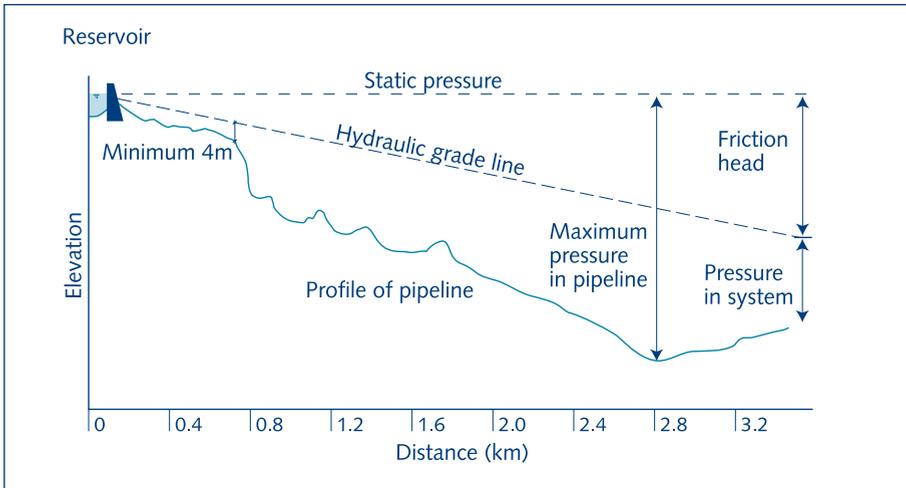


Fig. 20.5. Pressure distribution in gravity transmission mains

If water is to be transported to higher elevations, the maximum pressures will occur in the vicinity of pumping stations (Fig. 20.6). High pressures in the transmission pipe can be avoided in this case by application of multistage pumping along the pipe route.

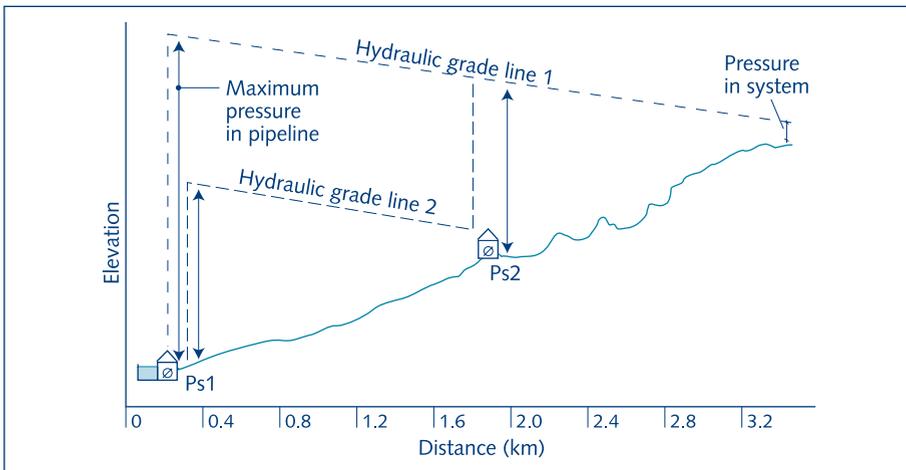


Fig. 20.6. Pressure distribution in pumped transmission mains

Critical pressures may also develop as a result of pressure surge or water hammer in the pipeline. This phenomenon is caused by the instant or too rapid closure of valves, or by sudden pump starts or stops, e.g. due to electricity failure. A longitudinal water wave created in such a way causes over- and under-pressures well above the normal working pressure. This is potentially a very dangerous situation that may result in damage to the pipeline over long distances. Proper prevention includes construction of surge tanks, air vessels or water towers as well as selection of suitable pipe materials that can withstand the highest pressures. Regarding valves, specified minimum shut-off times should be strictly respected. This makes it important how communities choose, train and supervise valve operators and that operators understand, and practise, the proper regulation of the valves.

Design velocity and hydraulic gradient

A velocity range is established for design purposes for two reasons. On the one hand, a certain minimum velocity will be required to prevent water stagnation causing sedimentation and bacteriological growth in the conduits. On the other hand, the maximum velocity will have to be respected in order to control head losses in the system as well as to reduce the effects of water hammer.

The velocity of flows in canals, aqueducts and tunnels usually ranges between 0.4 and 1.0 m/s for unlined conduits, and up to 2 m/s for lined conduits. Flows in pressurised transmission mains have the velocity range between 1 and 2 m/s.

In the case of pressurised pipes, design values may also be set for the hydraulic gradient. This is done primarily to limit the head losses, i.e. to minimise the energy consumption for pumping the water. Common values of the hydraulic gradients for transmission pipes are around 0.005, which means 5 mwc of head loss per km of the pipe length.

20.4 Hydraulic design

Flow Q (m^3/s) through a cross-section A (m^2) is determined as $Q = vA$, where v (m/s) is the mean velocity of the cross-section. Assumptions of 'steady' and 'uniform' flow apply in basic hydraulic calculations for the design of water transmission systems. The flow is steady if the mean velocity of one cross-section remains constant within a certain period of time. If the mean velocity between the two cross-sections is constant at a certain moment, the flow is uniform.

Free-flow conduits

The Strickler formula is widely used for conduits with free-flow conditions. The formula reads:

$$v = K_s R^{2/3} S^{1/2}$$

where:

v = mean water velocity in the cross-section (m/s)

K_s = Strickler coefficient ($m^{1/3}/s$)

R = hydraulic radius (m)

S = hydraulic gradient (m/km)

The Strickler coefficient represents roughness of the conduit. For design purposes, table 20.1 provides indicative values of this coefficient for various types of linings in clean, straight conduits. In practice, these values may differ from one channel section to another and are often subjected to seasonal variations.

Table 20.1 Indicative values of the Strickler coefficient for various types of linings

Type of Lining	Strickler coefficient K_s ($m^{1/3}/s$)
Planed timber, joints flush	80
Sawn lumber, joints uneven	70
Concrete, trowel finished	80
Masonry	
- Neat cement plaster	70
- Brickwork, good finish	65
- Brickwork, rough	60
Excavated	
- Earth	45
- Gravel	40
- Rock cut, smooth	30
- Rock cut, jagged	25

In a wide range of literature the Strickler coefficient is listed as the Manning coefficient, n in $m^{-1/3}s$, where $n = 1/K_s$. Consequently, the formula is called the Manning formula:

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

The hydraulic radius, $R = A/P$, where A (m^2) is the cross-section area and P (m), the wetted perimeter. The formulas for a few typical cross-sections are listed in figure 20.7. Finally, the hydraulic gradient, S , can be substituted by the slope of the conduit where the assumption of uniform flow conditions is valid.

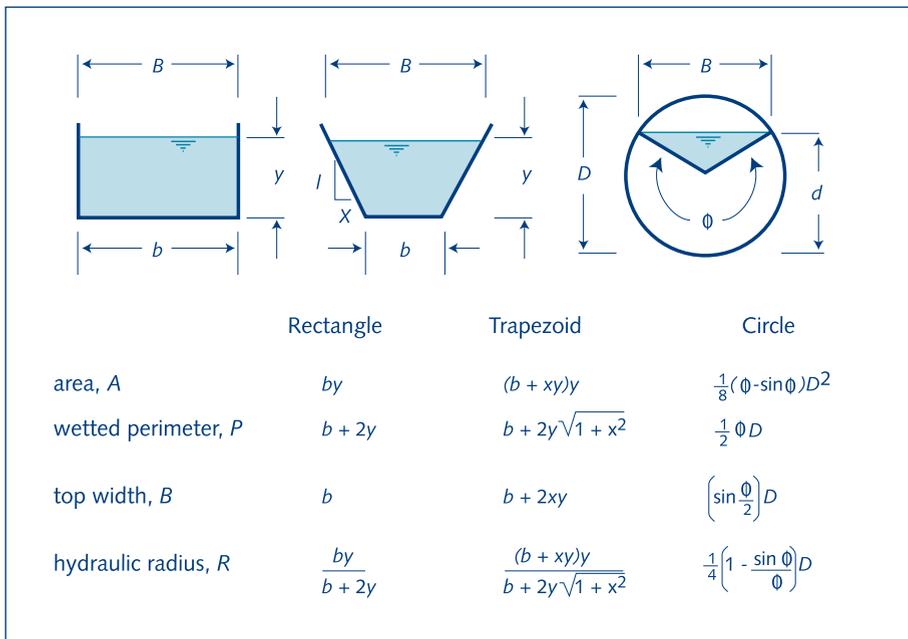


Fig. 20.7. Geometric properties of typical cross-sections

Closed Conduits

The Strickler and Manning formulas are also applicable for closed conduits by introducing the real hydraulic gradient of the flow and the wetted perimeter as the full perimeter of the conduit. Nevertheless, a problem may occur in the selection of the roughness factors for a wide range of pipe materials and flow conditions.

More appropriate formulas for computing the head loss of water flowing through a pressurised pipeline are those of Darcy-Weisbach and Hazen-Williams.

The Darcy-Weisbach formula states:

$$\Delta H = \lambda \frac{L v^2}{D 2g} = \frac{8\lambda L}{\pi^2 g D^5} Q^2 = \frac{\lambda L}{12.1 D^5} Q^2$$

where:

ΔH = head loss (mwc)

L = pipe length (m)

D = pipe diameter (m)

λ = friction factor (-)

v = the mean velocity in the pipe (m/s)

g = gravity (9.81 m/s²)

Q = flow rate (m³/s)

Introducing the hydraulic gradient, $S = \Delta H/L$, the formula can be rewritten as:

$$v = \sqrt{\frac{2gDS}{\lambda}}$$

The factor λ is the friction coefficient that can be calculated by the Colebrook-White formula:

$$\frac{1}{\sqrt{\lambda}} = -2 \log \left[\frac{2.51}{\text{Re} \sqrt{\lambda}} + \frac{k}{3.7D} \right]$$

where:

- Re = the Reynolds number (-)
- k = absolute roughness of the inner pipe wall (mm)
- D = pipe diameter (mm)

The Reynolds number indicates the flow regime:

$$\text{Re} = \frac{vD}{\nu}$$

where:

- v = the mean velocity in the pipe (m/s)
- D = pipe diameter (m)
- ν = kinematic viscosity (m^2/s)

Finally, the kinematic viscosity is dependent on the water temperature. For T in °C:

$$\nu = \frac{497 \cdot 10^{-6}}{(T + 42.5)^{1.5}}$$

The Colebrook-White formula is developed for a turbulent flow regime, i.e. Re-values above ± 4000 . The common values in practice are much higher, typically in the order of 10^4 and 10^5 . If by chance the flow is laminar ($\text{Re} < 2000$), the friction factor λ will be calculated as: $\lambda = 64/\text{Re}$.

Calculation by the Colebrook-White formula is not straightforward, as the λ -factor appears on both sides of the equation. The alternative formula of Barr can be used instead:

$$\frac{1}{\sqrt{\lambda}} = -2 \log \left[\frac{5.1289}{\text{Re}^{0.89}} + \frac{k}{3.7D} \right]$$

To by-pass somewhat cumbersome computations, the work can also be facilitated by use of the Moody diagram, hydraulic tables or pipe charts (see Chadwick & Morfett, 1996 or Bhawe, 1991). These tables/charts, which are produced for certain water temperatures

and k-values, show the velocities (flows) for a range of pipe diameters and hydraulic gradients. Any of the three parameters can be determined by fixing the other two.

The common range of k-values is listed in table 20.2 for various pipe materials. For practical calculation these values can be increased depending on the number of years the pipe was in service and the influence of head losses caused by bends, joints, valves, etc.

Table 20.2 Absolute roughness (Bhave, 1991)

Pipe material	k (mm)
Asbestos cement	0.015 - 0.03
Bitumen/Cement lined	0.03
Wrought iron	0.03 - 0.15
Galvanised/Coated cast iron	0.06 - 0.3
Uncoated cast iron	0.15 - 0.6
Ductile iron	0.03 - 0.06
Uncoated steel	0.015 - 0.06
Coated steel	0.03 - 0.15
Concrete	0.06 - 1.5
Plastic, PVC, PE	0.02 - 0.05
Glass fibre	0.06
Brass, cooper, lead	0.003

The Hazen-Williams formula is simpler, although less accurate than the Darcy-Weisbach equation. It states for SI-units:

$$v = 0.355C_{hw}D^{0.63}S^{0.54}$$

The values of the Hazen-Williams factor, C_{hw} , are listed in table 20.3.

This formula is applicable for a common range of flows and diameters. Its accuracy becomes reduced at lower values of C_{hw} (much below 100), and/or velocities that are appreciably lower or higher than 1 m/s. Also, the Hazen-Williams formula is not dimensionally uniform and if other units are used than SI, it has to be readjusted. Nevertheless, due to its simplicity this formula is still widely used in the USA and in many, predominantly Anglophone, developing countries.

Table 20.3 The Hazen-Williams factors (Bhave, 1991)

Pipe material	C_{hw}	C_{hw}	C_{hw}	C_{hw}	C_{hw}
	D=75 mm	D=150 mm	D=300 mm	D=600 mm	D=1200 mm
Uncoated cast iron	121	125	130	132	134
Coated cast iron	129	133	138	140	141
Uncoated steel	142	145	147	150	150
Coated steel	137	142	145	148	148
Wrought iron	137	143			
Galvanised iron	129	133			
Uncoated asbestos cement	142	145	147	150	
Coated asbestos cement	147	149	150	152	
Concrete, minimum values	69	79	84	90	95
Concrete, maximum values	129	133	138	140	141
Prestressed concrete	147	149	147	150	150
PVC, brass, cooper, lead	142	145	150	152	153
Wavy PVC	147	149	147	150	150
Bitumen/cement lined			150	152	153

Application of the discussed head loss formulas is illustrated in the examples.

Example 1

Determine the capacity of the rectangular concrete canal if the water depth is 0.2 m. The width of the canal is 1.0 m and the slope of the bottom is $S = 1^0/_{00}$.

Solution

From table 20.2, K_s for concrete = $80 \text{ m}^{1/3}/\text{s}$. Further:

$$R = \frac{A}{P} = \frac{by}{b+2y} = \frac{1 \cdot 0.2}{1+2 \cdot 0.2} = 0.1429 \text{ m}$$

$$v = K_s R^{2/3} S^{1/2} = 80 \cdot 0.1429^{2/3} \cdot 0.001^{1/3} = 2.19 \text{ m/s}$$

$$Q = vb = 2.19 \cdot 1 \cdot 0.2 = 0.437 \text{ m}^3/\text{s} = 437 \text{ l/s}$$

Example 2

Find out the head loss in the concrete transmission pipe, $L = 300$ m and $D = 150$ mm, flowing full. The flow rate is $80 \text{ m}^3/\text{hour}$ and the water temperature is 10°C . Compare the results of the Darcy-Weisbach, Hazen-Williams and Strickler formulas.

Solution

For water temperature of 10°C , the kinematic viscosity:

$$\nu = \frac{497 * 10^{-6}}{(T + 42.5)^{1.5}} = \frac{497 * 10^{-6}}{(10 + 42.5)^{1.5}} = 1.31 * 10^{-6} \text{ m}^2/\text{s}$$

$$\text{The pipe velocity: } v = \frac{4Q}{D^2\pi} = \frac{4 * 80 / 3600}{0.15^2 * 3.14} = 1.26 \text{ m/s}$$

$$\text{and the Reynolds number: } v = \frac{4Q}{D^2\pi} = \frac{4 * 80 / 3600}{0.15^2 * 3.14} = 1.26 \text{ m/s}$$

From table 20.3, the k -value for concrete pipes ranges between 0.06 and 1.5 mm.

For $k = 0.8$ mm, the λ factor from the Barr equation:

$$\frac{1}{\sqrt{\lambda}} = -2 \log \left[\frac{5.1289}{\text{Re}^{0.89}} + \frac{k}{3.7D} \right] = -2 \log \left[\frac{5.1289}{(1.44 * 10^5)^{0.89}} + \frac{0.8}{3.7 * 150} \right] = 5.60887 \quad ; \quad \lambda = 0.032$$

$$\text{Finally: } \Delta H = \lambda \frac{L}{D} \frac{v^2}{2g} = 0.032 * \frac{300}{0.15} * \frac{1.26^2}{2 * 9.81} = 5.12 \text{ mwc}$$

According to table 20.4, the Hazen-Williams factor for ordinary concrete pipe

$D = 150$ mm ranges between 79 and 133. For C_{hw} assumed at 105:

$$v = 0.355 C_{hw} D^{0.63} S^{0.54} \quad ; \quad S = \left(\frac{v}{0.355 C_{hw} D^{0.63}} \right)^{1/0.54} = \left(\frac{1.26}{0.355 * 105 * 0.15^{0.63}} \right)^{1.852} = 0.01722$$

Consequently: $\Delta H = SL = 0.01722 * 300 = 5.17 \text{ mwc}$

Finally, for $K_s = 85 \text{ m}^{1/3}/\text{s}$:

$$R = \frac{A}{P} = \frac{D^2\pi}{D\pi} = \frac{D}{4} = \frac{0.15}{4} = 0.0375 \text{ m}$$

$$v = K_s R^{2/3} S^{1/2} \quad ; \quad S = \left(\frac{v}{K_s R^{2/3}} \right)^2 = \left(\frac{1.26}{85 * 0.0375^{2/3}} \right)^2 = 0.01743$$

$\Delta H = SL = 0.01743 * 300 = 5.23 \text{ mwc}$

All three formulas show similar results in this case. This can differ more substantially for different choice in roughness parameters. E.g. in case of $C_{hw} = 120$, the same calculation by the Hazen-Williams formula would yield $\Delta H = 4.03 \text{ mwc}$ while for $K_s = 80 \text{ m}^{1/3}/\text{s}$, the Strickler formula gives $\Delta H = 5.90 \text{ mwc}$.

In practice, the accuracy of any head loss formula is of less concern than a proper choice of the roughness factor (k , Chw or Ks) for a given surface. Errors in results originate far more frequently from insufficient knowledge about the condition of the conduit, than from a wrong choice of formula.

Example 3

What will be the flow in a 100 mm-diameter pipe to transport water from a small dam to a tank at 600 m distance? The difference between the water surfaces in the two points is 3.60 m. The absolute roughness of the pipe wall is $k = 0.25$ mm and the water temperature equals 10°C .

Solution

The difference between the water levels indicates the available head loss. Hence, $\Delta H = 3.60$ mwc and $S = 3.60/600 = 0.006$. From the previous example, $\nu = 1.31 \cdot 10^{-6}$ m²/s for the water temperature of 10°C . The calculation has to be iterative due to the fact that the velocity (flow) is not known and it influences the Reynolds number, i.e. the flow regime. A common assumption is $v = 1$ m/s. Further:

$$Re = \frac{1.0 \cdot 0.1}{1.31 \cdot 10^{-6}} = 7.65 \cdot 10^4$$

$$\frac{1}{\sqrt{\lambda}} = -2 \log \left[\frac{5.1289}{(7.65 \cdot 10^4)^{0.89}} + \frac{0.25}{3.7 \cdot 100} \right] = 6.0851 \quad ; \quad \lambda = 0.027$$

$$v = \sqrt{\frac{2gDS}{\lambda}} = \sqrt{\frac{2 \cdot 9.81 \cdot 0.1 \cdot 0.006}{0.027}} = 0.66 \text{ m/s}$$

The calculated velocity is different from the assumed one of 1 m/s. The procedure has to be repeated starting with this new value. For $v = 0.66$ m/s, $Re = 5.06 \cdot 10^4$, and $\lambda = 0.028$, which yields $v = 0.65$ m/s. The difference of 0.01 m/s is considered as acceptable and hence:

$$Q = 0.65 \cdot \frac{0.1^2 \pi}{4} = 0.0051 \text{ m}^3/\text{s} = 5.11/\text{s}$$

20.5 Water transmission by pumping

Transmission by pumping is applied in cases when the water has to be transported over large distances and/or to higher elevations. The pumping head is the total head, and comprises the static head plus the friction head loss for the design flow rate. The pump to be selected must be able to provide this head (Fig. 20.9).

The head loss corresponding to the design flow rate can be computed for several pipe diameters using the principles presented in paragraph 20.4. Each combination of the pumping head and corresponding pipe diameter should be capable of supplying the

required flow rate over the required distance, and up to the service reservoir. Smaller pipe diameters will require a higher pumping head to overcome the increase in head losses, and the other way round. As a result, one pipe diameter will represent the least-cost choice taking into account the initial costs (capital investment), maintenance costs and the energy costs for pumping. The total cost, capitalised, is the basis for selecting the most economical pipe diameter.

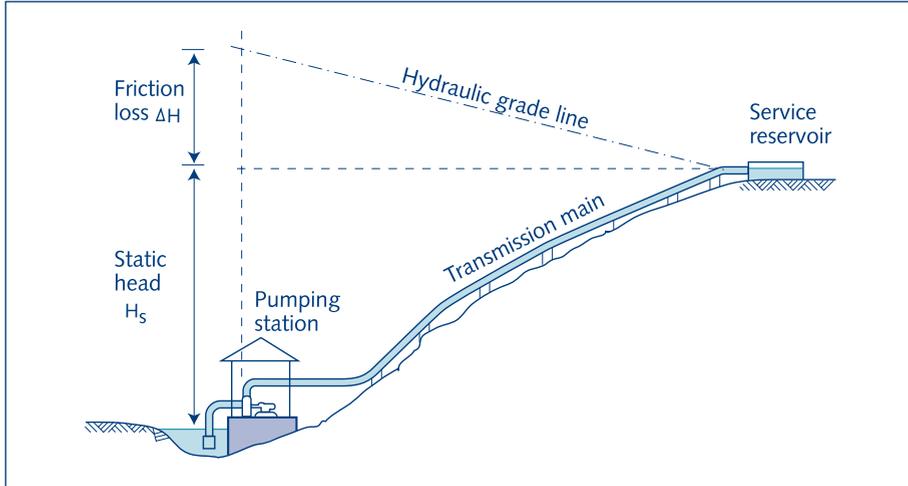


Fig. 20.8. Pumped supply

For this analysis, the calculated costs for different pipe sizes are plotted in a graph of which figure 20.9 shows an example.

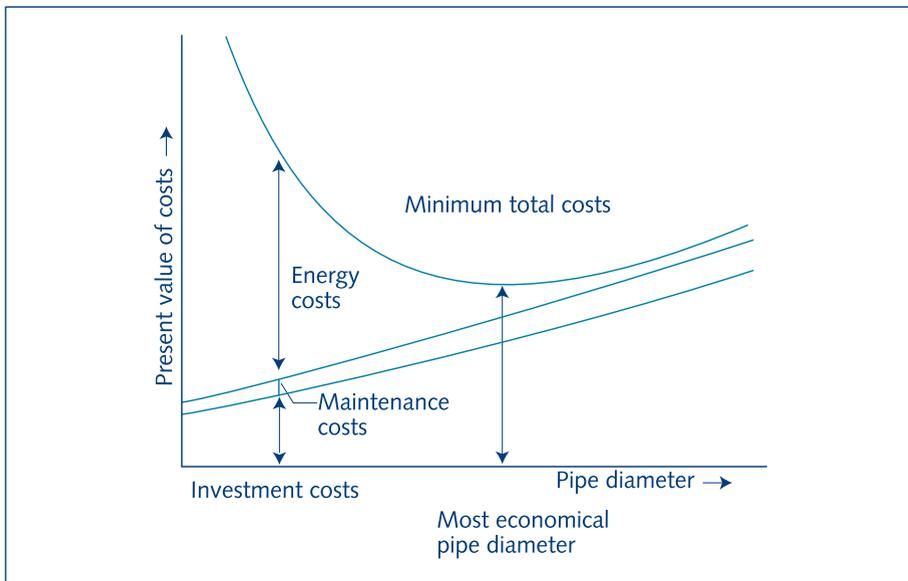


Fig. 20.9. Determination of most economical pipe diameter

The most economical pipe diameter will tend to be large when energy costs are high, unit costs of pipe low, and capital interest rates low. Nevertheless, it should not be forgotten that a larger pipe means lower velocity, i.e. potential water quality problems. As a preliminary estimate, the range of possible most economic diameters can be selected based on velocities around 1 m/s.

Selection of pumps

Various types of pumps have been mentioned in chapter 9: centrifugal, axial-flow, mixed-flow and reciprocating pumps. The choice of pump will generally depend on its duty in terms of pumping head and capacity.

Pumps with rotating parts have either a horizontal or a vertical axis. The choice between these is generally based on the pump-motor drive arrangement and the site conditions. At a site subject to flooding, the motor and any other electrical equipment must be placed above the flood level. Local knowledge is invaluable to identify such risks.

In water transmission for community water supply purposes it is not unusual that a substantial head is required. This implies that the pumps selected frequently are of the centrifugal (radial-flow) type.

Many drinking water pumps are designed to run (almost) continuously throughout a day. In such cases an increase in pump efficiency of a few percent may represent a considerable saving in the running costs over a long period of time. However for rural water supplies, an even more important requirement is that any pump installed should be reliable.

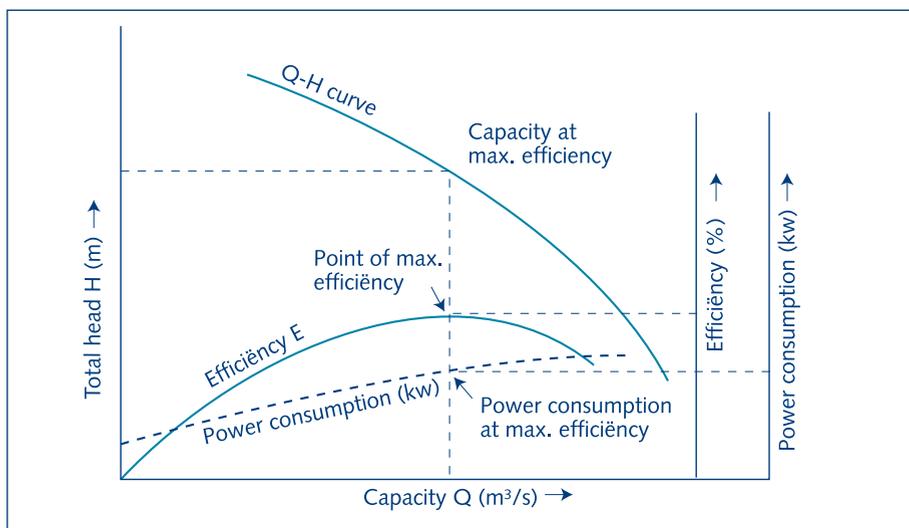


Fig. 20.10. Typical pump characteristics

The head/capacity characteristic of a pump and its efficiency are indicated in catalogues that are supplied by the manufacturers of the pump. Figure 20.10 shows an example.

In practice it is not an easy task to have a pump permanently run at its maximum efficiency because the operating point of the pump is determined by both the pumping head and the capacity, and thus can vary considerably. This applies to some degree for water transmission systems and even more in the case of distribution networks. Efficiencies of small-capacity pumps operating under the conditions of rural areas in developing countries are frequently quite low. A tentative estimate would be as low as 30% for a 0.4-Kilowatt pump, up to 60% for a 4-Kilowatt pump.

Power requirements

The power required for driving a pumping unit can be computed by the following formula:

$$N = \frac{\rho g Q (H_s + SL)}{\eta}$$

where:

N = power required for pumping (Watts)

Q = maximum pumping capacity (m³/s)

ρ = specific weight of water (kg/m³)

η = pumping efficiency (-)

H_s = static head (m)

S = hydraulic gradient (m/km)

L = pipe length (m)

Introducing the specific weight in kg/dm³ gives the result for power in Kilowatts (kW). Assuming ρ = 1 kg/dm³, g = 10 m/s² and η for small-capacity pumps estimated at 0.5 (50 %), the above formula can be further simplified to:

$$N = 20Q(H_s + SL)$$

This formula gives N in Watts for Q in l/s, either in Kilowatts for Q in m³/s. The head is in both cases specified in metres of water column (mwc).

Example

For a water supply, pumping is required at rate of 150,000 litres per 12 hours. The static head is 26 m, and the length of the pipeline is 450 m. Determine the power requirement for pumping station, if a PVC pipe, $D = 80$ mm is used.

Solution

$$Q = \frac{150000}{12 * 3600} = 3.471/s \quad ; \quad v = \frac{0.00347}{\frac{0.08^2 \pi}{4}} = 0.69 \text{ m/s}$$

From table 20.4, C_{hw} for PVC of $D = 80$ mm can be assumed at 147. Further:

$$v = 0.355C_{hw}D^{0.63}S^{0.54} \quad ; \quad S = \left(\frac{v}{0.355C_{hw}D^{0.63}} \right)^{1/0.54} = \left(\frac{0.69}{0.355 * 147 * 0.08^{0.63}} \right)^{1.852} = 0.0063$$

$$N = 20Q(H_s + SL) = 20 * 3.47 * (26 + 0.0063 * 450) = 2003 \text{ W} \approx 2 \text{ kW}$$

Pump Installations

Pumping stations may be of the wet-pit type (with submersible pumps or pumps driven by motors placed above the pump in the sump), or of the dry-pit type (pump installed in a pump room). The wet-pit type has the pumps immersed in the water, and the dry-pit type has the pump in a dry room separated from the water by a wall.

For ease of installation horizontal pumps are sometimes situated above ground level. In that case the pump must be of the self-priming type, which is generally not such a reliable arrangement for rural water supply installations. With too high values of the suction head, a risk of cavitation may occur.

Examples of various types of pump installations are shown in figures 20.11 and 20.12.

20.6 Pipe materials

Pipelines frequently represent a considerable investment and selection of the right type of pipe is important. Pipes are available in various materials, sizes and pressure classes. The most common materials are cast iron (CI), ductile iron (DI), steel, asbestos cement (AC), polyvinyl chloride (PVC) and polyethylene (PE). Galvanised steel (GS) is sometimes selected because of its resilience for situations where subsidence of the pipes is expected. Apart from these, indigenous materials such as bamboo may have limited application.

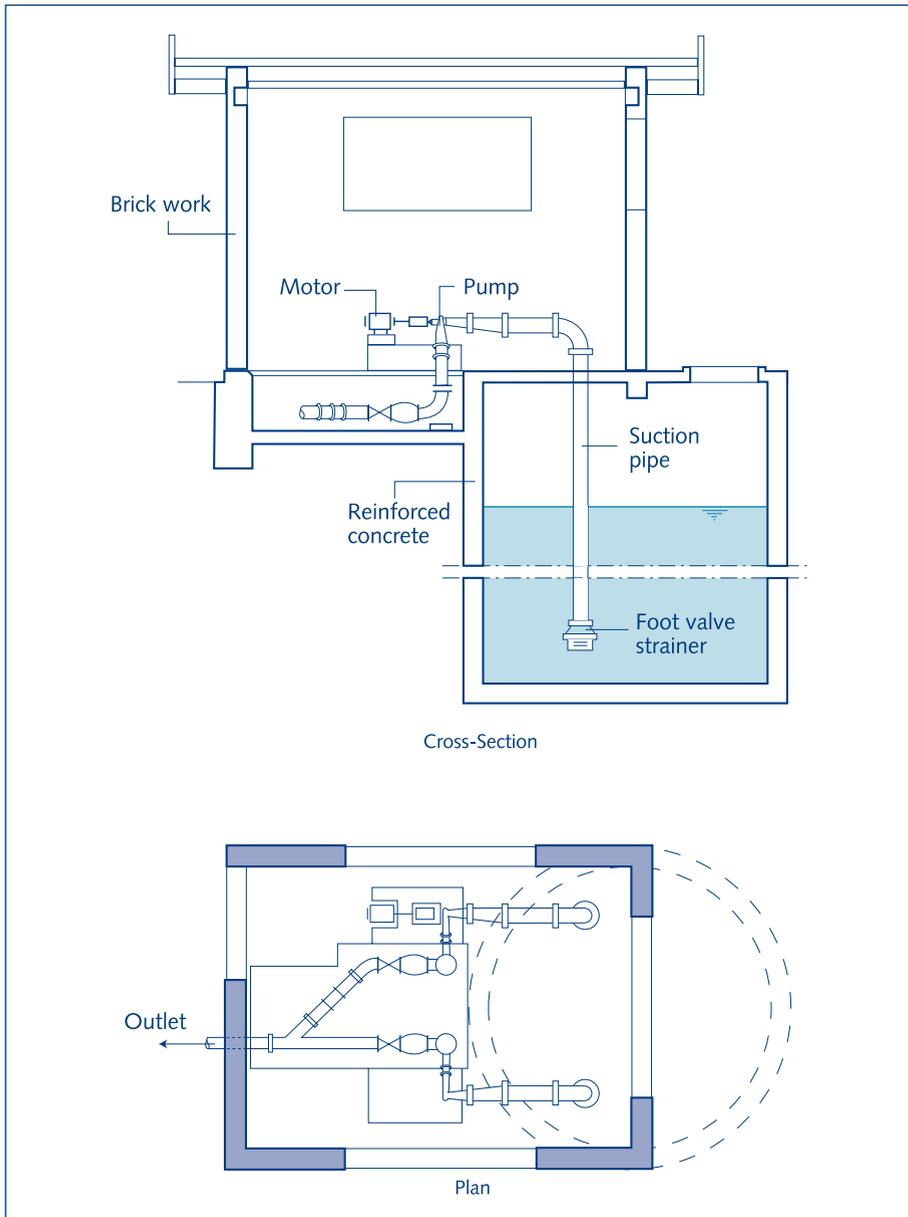


Fig. 20.11. Pumping station with horizontal pumps (self priming)

Factors influencing the choice of pipe material are:

- the cost and local availability of different types of pipe;
- the design pressure in the distribution system;
- the corrosiveness of the water and of the soil in which the pipes are to be laid;
- conditions such as traffic overload, proximity to sewer lines, and crowded residential areas.

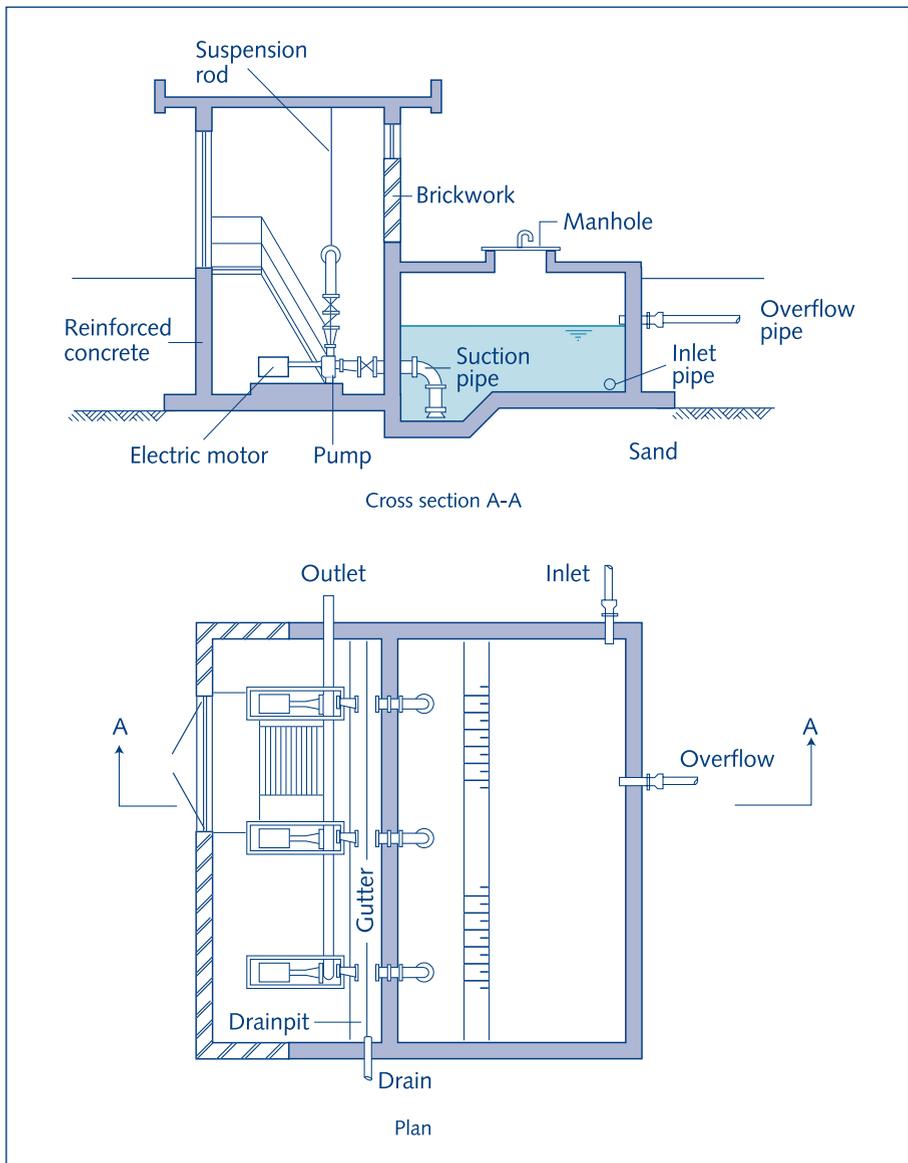


Fig. 20.12. Pumping installation (dry pit)

Although specific conditions will vary from one country to another, the following general observations apply in most cases.

Ductile iron and steel are the strongest pipe materials, making them the best choice when very high operating pressures are to be expected. These pipes are almost impossible to tap without special tools and equipment, which is an advantage in cases where illegal connections pose a real threat for the water company. However, the costs of fittings, valves, etc. increase rapidly for higher pipe pressure classes and it is therefore often advisable to reduce the maximum internal pipe pressure through the provision of

a pressure reducing valve or break-pressure tank. A break-pressure tank is generally more reliable than a pressure reducing valve.

In spite of higher investment costs, ductile iron pipes are a better alternative than cast iron pipes because they have a longer service life, are lighter and more flexible and require hardly any maintenance. The pipe is practically corrosion resistant due to coatings applied inside and out.

Compared with metal pipes, asbestos cement pipes are light and easy to handle. Except for soils containing sulphate, these pipes show good corrosion resistance. They are widely used in sizes up to 300 mm, mainly for secondary pipes and for low-pressure mains. Asbestos cement may be less suitable for transmission mains because non-authorized tapping of such mains is possible. Moreover, this pipe material may be subject to scale bursts when tapped without sufficient skill.

The carcinogenic effect of AC materials has been analysed carefully in recent years. Although not dangerous when used to supply drinking water, asbestos fibres can be very harmful when inhaled. AC pipes are slowly being phased-out due to possible hazards during manufacturing and maintenance works. Alternative materials are in this case PVC, PE or DI.

Polyvinyl chloride pipes have the advantage of easy jointing and their corrosion resistance is good. They can be manufactured in several quality classes to meet the selected design pressure. PVC, however, suffers a certain loss in strength when exposed to sunlight for long periods of time and care should be taken to cover the pipes when these are stocked in the open. This is one of the aspects that informed villagers can easily check. Secondly, in case of calamity, PVC pipe breaks along considerable lengths, causing large water losses. Therefore, the pipe should not be laid directly on rocky soil and heavy surface loads are to be avoided. Burying the pipe deeper in the ground can diminish the effect of extreme ambient temperatures. Non-authorized tapping of rigid plastic (PVC) mains is also difficult to prevent.

High-density polyethylene (PE) is a very suitable pipe material for small-diameter mains because it can be supplied in coil. The potential of laying this pipe in longer lengths reduces the number of necessary joints. Particularly in cases where rigid pipe materials would necessitate a considerable number of special parts such as elbows and bends, the flexible PE makes for an ideal pipe material. Polyethylene does not deteriorate when exposed to direct sunlight. Conventional jointing of the PE-pipes may cause leakage and welding is considered to be a better alternative. Furthermore, formation of bio-film in the pipe may be enhanced in some cases.

To summarise, for pipelines of small-diameter (less than 200 mm) PVC and PE may generally be the best alternative unless high working pressures are expected (above 60 mwc). These pipes can also be used for medium- to large-sized pipelines (diameters up to 500-600 mm) where lower pressures can be maintained. Cast iron, ductile iron and steel are generally only used for large-diameter mains and also in cases where very high pressures necessitate their use in small- or mid-range diameter pipes. Due to heavy weight and lower flexibility, CI pipes are becoming less advantageous than DI, despite lower prices. Asbestos cement can be considered only if no other viable alternative exists. Stringent measures that have to be introduced while handling these pipes involve the prevention of the production and inhalation of fibre dust (use of special saws, cutting under wet conditions, protection masks for the workers, etc).

Table 20.4 lists the comparative characteristics of pipe materials for pipelines.

Valves

Apart from the sluice ("gate") valves and non-return valves fitted to the pump outlets in case of a pumped supply, various types of valves and appurtenances are used in the transmission main proper. As the pipeline will normally follow the terrain, provision must be made for the release of trapped air at high points. Air release valves should be provided at all these points on the pipeline and may also be required at intermediate positions along long lengths of even gradient. To avoid under-pressure, air admission valves may also have to be used. These serve to draw air into the pipeline when the internal pressure falls below a certain level. At the lowest points of the pipeline, drain or discharge valves must be installed to facilitate emptying or scouring the pipeline.

In long pipelines sluice valves should be installed to enable sections of the pipeline to be isolated for inspection or repair purposes. Especially when parallel pipes are used it is advantageous to connect them at intervals. In the event of leakage or pipe burst only one section of such an interconnected main needs to be taken out of operation, whereas the other sections and the entire other main can still be used. In this way the capacity of the parallel pipe as such is hardly reduced. It should be mentioned that this advantage is obtained at a cost because each connection between the twin mains requires at least five valves.

Sluice valves perform their function either fully opened or completely closed. For pipe diameters of 350 mm and less, a single valve may be used. For larger diameters a small-diameter bypass with a second valve will be needed because otherwise the closing of the large-diameter valve might prove very difficult. In those cases where the flow of water has to be throttled by means of a valve, butterfly valves should be used. This type of valve may equally be used instead of the sluice valves mentioned above, but the cost is usually somewhat higher.

Table 20.4 Comparison of pipe materials (Smith et al., 2000)

Characteristic	CI	Lined DI	Steel	GS	AC	PVC	PE
Material category	Metal	Metal	Metal	Metal	Concrete	Plastic	Plastic
Int. corrosion resistance	Poor	Good	Poor	Fair	Good	Good	Good
Ext. corrosion resistance	Fair	Moderate	Poor	Fair	Good	Very good	Very good
Cost	Moderate	Moderate	Moderate	Moderate	Low	Low	Low
Weight	High	High	High	Moderate	Moderate	Low	Low
Life expectancy	High	High	High	High	Moderate	Moderate	Moderate
Primary use	T/D*	T/D	T/D	T/D	D	D	S/D
Tapping characteristics	Fair	Good	Good	Good	Fair	Poor	NA
Internal roughness	Moderate to high	Low	Moderate to high	Moderate to high	Low to moderate	Low	Low
Effect on water quality	High	Low	Moderate	Moderate	Low	Moderate	Low
Equipment needs	Moderate	High	Moderate	Moderate	Moderate	Low	Low
Ease of installation	Low to moderate	Low to moderate	Low to moderate	Low to moderate	Moderate	Moderate to high	High
Joint watertightness	Fair	Very good	Very good	Fair	Good	Good	Poor
Pressure range (mwc)	NA	100-250	Varies	NA	70-140	Max.160	Max.140
Diameter range (mm)	NA	80-1600	100-3000	NA	100-1100	100-900	100-1600

* T: transmission, D: distribution, S: service connections

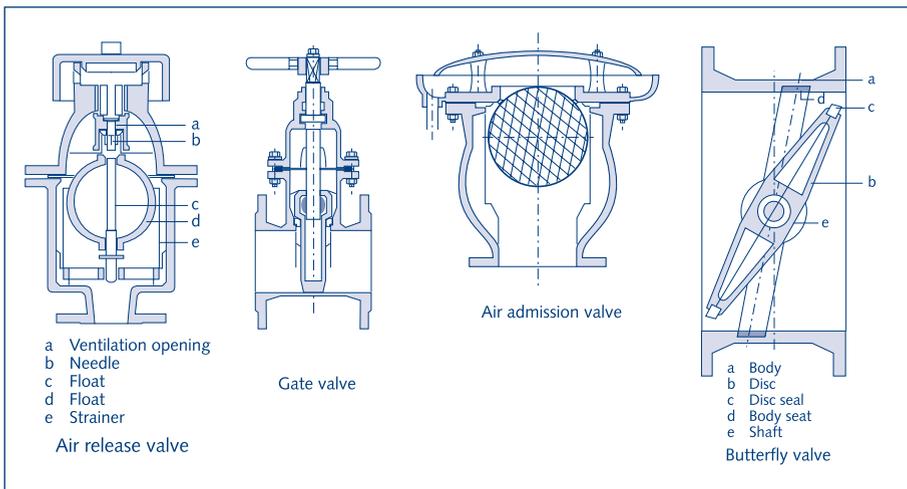


Fig. 20.13. Various types of valves

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21 Water distribution

Nemanja Trifunovic

21 Water distribution

21.1 Introduction

Water distribution systems convey water drawn from the water source or treatment facility, to the point where it is delivered to the users. Unlike the transmission systems discussed in chapter 20, these systems deal with water demand that varies considerably in the course of a day. Water consumption is highest during the hours that water is used for personal hygiene and cleaning, and when food preparation and clothes washing are done. Water use is lowest during the night.

This variation in flow can be dealt with by operating pumps in parallel and/or building balancing storage in the system. For small community water supplies the distribution system with water storage (e.g. a service reservoir) is the preferable option, given that supplies of electricity or diesel to power pumps will usually be unreliable. Although it can be kept simple, construction of such a system may represent a substantial capital investment and the design must be done properly.

Generally, the distribution system of a small community water supply is designed to cater for the domestic and other household water requirements. Stock watering and garden plot irrigation water may also be provided. Service reservoirs accumulate and store water during the night so that it can be supplied during the daytime hours of high water demand.

It is necessary to maintain sufficient pressure in the distribution system to protect it against contamination by the ingress of polluted seepage water. For small community supplies a minimum pressure of 5-10 mwc (metres of water column) should be adequate in most instances.

21.2 Types of distribution systems

There are basically two main layouts of a distribution network:

- Branched configuration (figure 21.1a)
- Looped (or "grid") configuration (figure 21.1b)

Branched networks are predominantly used for small-capacity community supplies delivering the water mostly through public standpipes and having few house connections, if any. Although adequate, having in mind simplicity and acceptable investment costs, branched networks have some disadvantages:

- Low reliability, which affects all users located downstream of any breakdown in the system

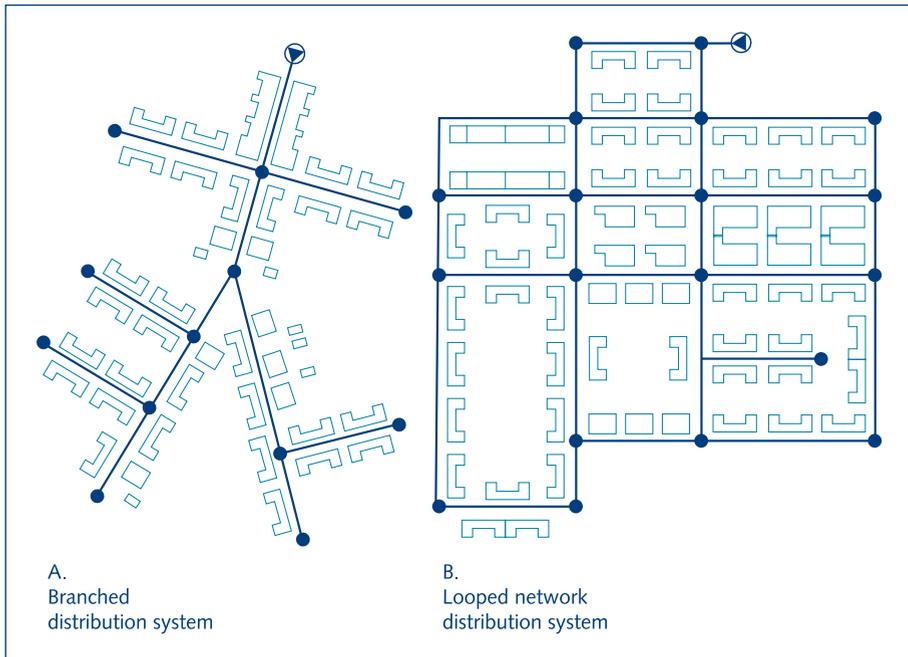


Fig. 21.1. Types of distribution systems

- Danger of contamination caused by the possibility that a large part of network will be without water during irregular situations
- Accumulation of sediments, due to stagnation of the water at the system ends ("dead" ends) occasionally resulting in taste and odour problems
- Fluctuating water demand producing rather large pressure variations

Branched systems are easy to design. The direction of the water flow and the flow rates can readily be determined for all pipes. This is different in looped distribution networks, where consumers can be supplied from more than one direction. Looped networks greatly improve the hydraulics of the distribution system. This is of major importance in the event that one of the mains is out of operation for cleaning or repair.

A looped network usually has a skeleton of secondary mains that can also be in a form of branch, one loop ('ring'), or a number of loops. From there, the water is conveyed towards the distribution pipes and further to the consumers. The secondary mains are connected to one or more loops or *rings*. The network in large (urban) distribution systems will be much more complex (figure 21.2); essentially a combination of loops and branches with lots of interconnected pipes that requires many valves and special parts. To save on equipment costs, over-crossing pipes that are not interconnected may be used but at the cost of reduced reliability (figure 21.3).

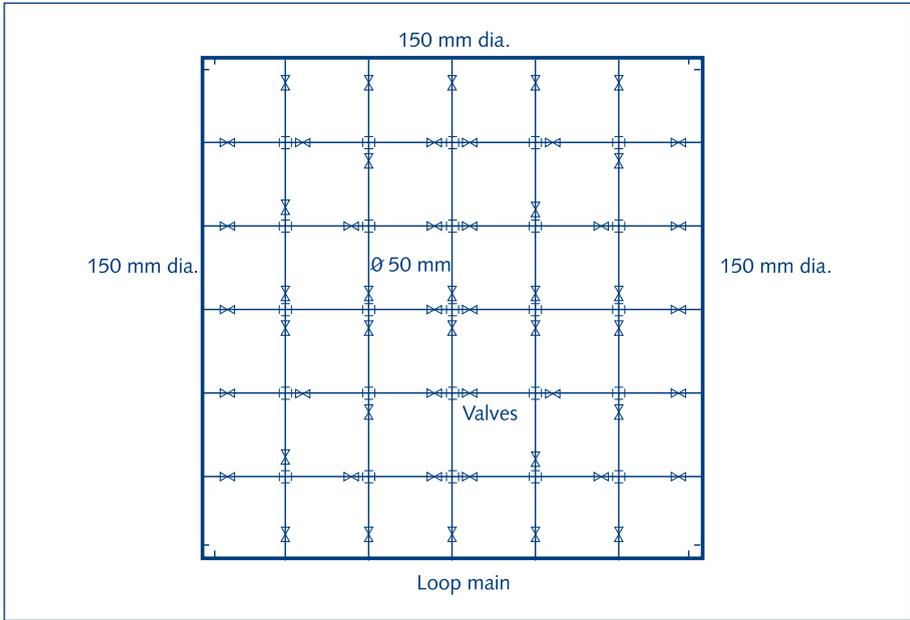


Fig. 21.2. Fully interconnected pipes

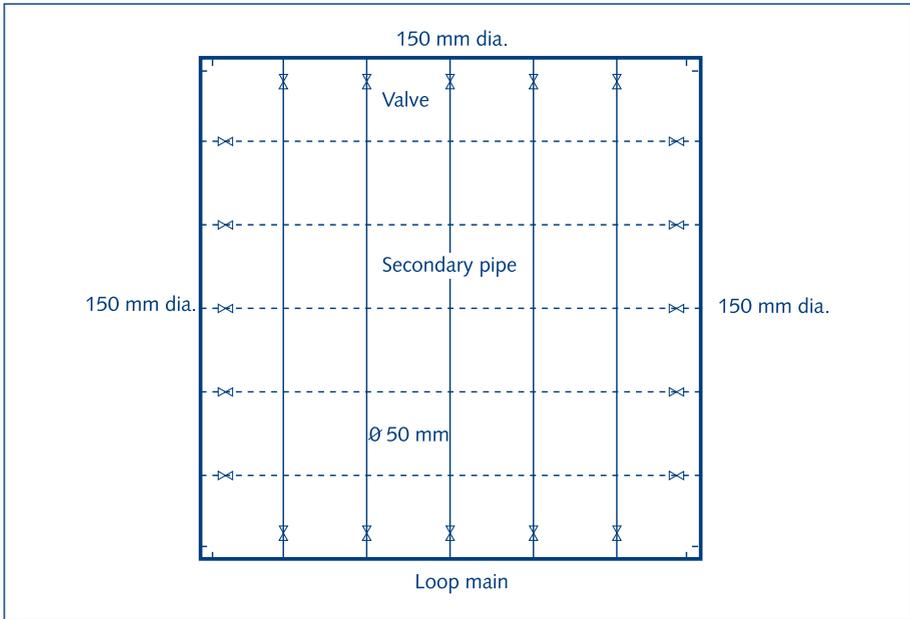


Fig. 21.3. Over-crossing single pipes

Points at which the water is delivered to the users are called service connections. The number and type of service connections has considerable influence on the choice of a network layout.

The following types of service connections may be distinguished:

- House connection
- Yard connection
- Group connection
- Public standpipe

A house connection is a water service pipe connected with in-house plumbing to one or more taps, e.g. in the kitchen and bathroom. Usually 3/8 inch (9 mm) and 1/2 inch (12 mm) taps are used. A typical layout is shown in figure 21.4.

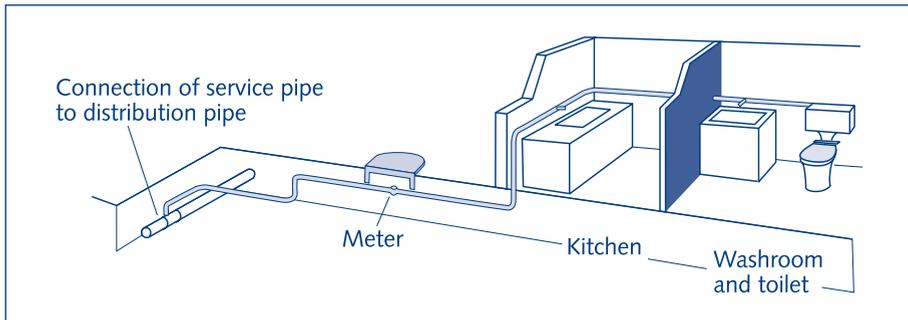


Fig. 21.4. House connection

The service pipe is connected to the distribution main in the street by means of a T-piece (on small-diameter pipes), a special insert piece (*ferrule*) or a saddle (on larger-size secondary pipes). A special insert piece is mostly used for cast iron and ductile iron pipes.

A yard connection is quite similar to a house connection, the only difference being that the tap is placed in the yard outside the house. No in-house piping and fixtures are provided (figure 21.5).

Plastic (polyvinyl chloride or polyethylene), cast iron and galvanised steel pipes are used for both house connections and yard connections.

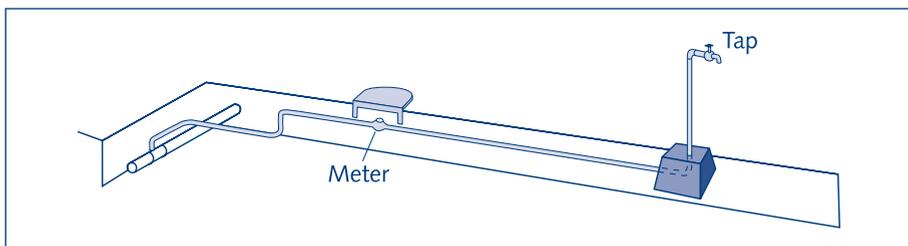


Fig. 21.5. Yard connection

Group connections are outside taps that are shared by a clearly defined group of households, often neighbours. They share the tap and pay the bill together. Each family may pay the same (flat) share or contributions may be weighed according to the estimated volume of water each family consumes. Consumption estimates may be based on indicators such as family size and composition and types of use by the different households. Sometimes the taps can be locked and a local committee holds the key and manages the use and financing.

Public standpipes can have one or more taps. Single-tap and double-tap standpipes are the most common types in rural areas. They are made of brickwork, masonry or concrete, or use wooden poles and similar materials. The design should be done in close consultation with the users (especially women) in order to arrive at an ergonomically optimal solution. Standpipes may have platforms at different levels, making it easy for adults and children to use them with containers of different sizes. Examples are shown in figures 21.6 and 21.7. Cattle watering and/or washing and/or bathing facilities may be constructed nearby. The design and often also the construction is best done in consultation and with participation of the user households, i.e. both men and women. Public taps drawing from a small reservoir (*cistern*) represent an alternative method of water distribution (figure 21.8).

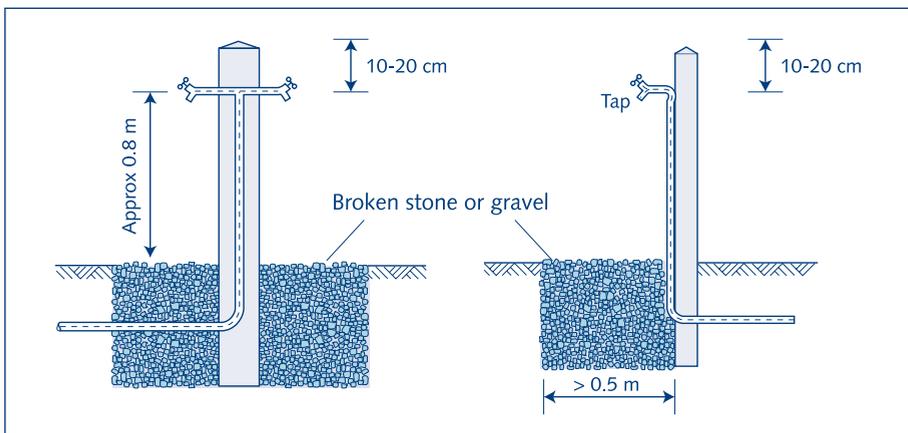


Fig. 21.6. Cross-section of simple standpipes

Each standpipe should be situated at a suitable point within the community area in order to limit the distance the water users have to go to collect their water. The walking distance for the most distant user of a standpipe should, whenever possible, be limited to 200 m; in sparsely populated rural areas 500 m may be acceptable. The required discharge capacity of a standpipe normally is about 14-18 litres/minute at each outlet. A single-tap standpipe should preferably be used by not more than 40-70 people; a multiple-tap standpipe may provide a reasonable service for up to 250-300 persons; in no case should the number of users dependent on one standpipe exceed 500.

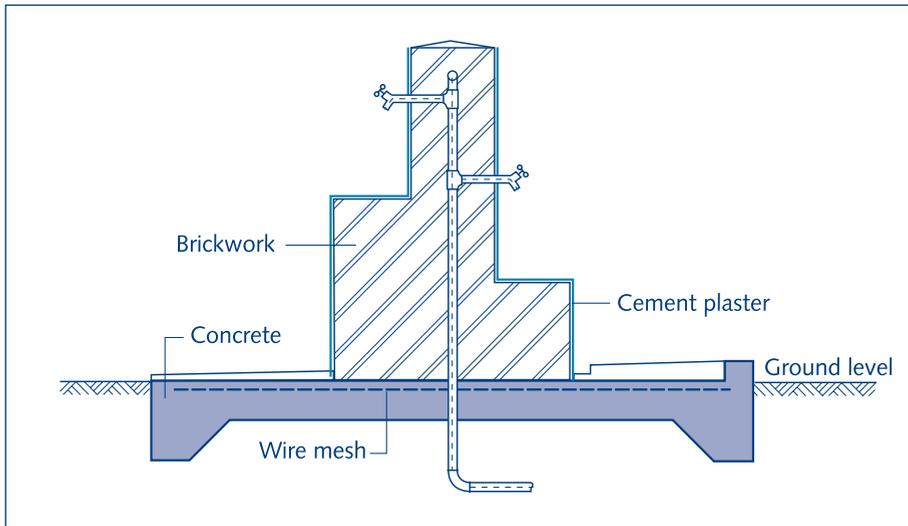


Fig. 21.7. Cross-section of multiple-tap standpipe

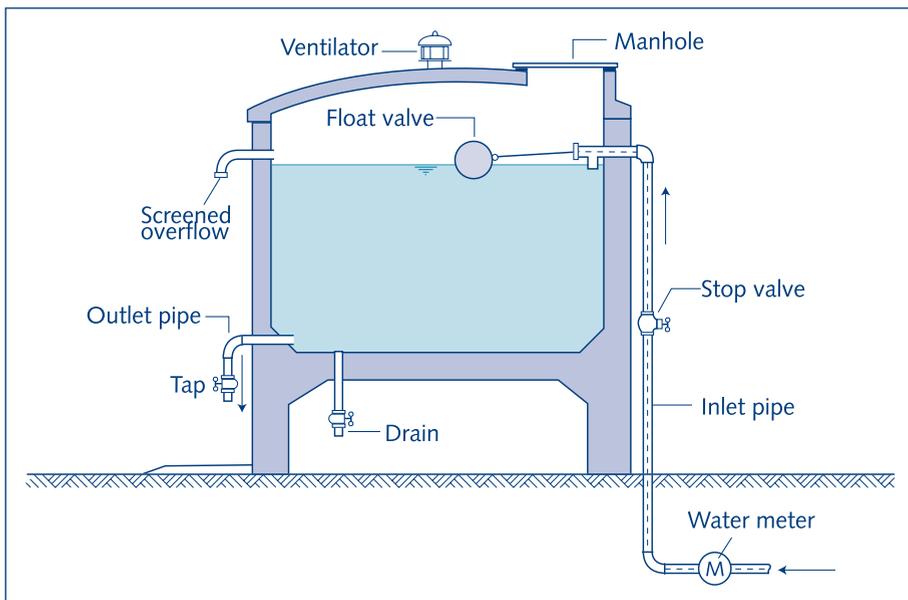


Fig. 21.8. Communal taps supplied from a small reservoir (Cistern)

Public standpipes can operate at a low pressure. Distribution systems that serve only standpipes may therefore use low pressure piping, whereas the pipes for distribution systems with house connections generally have to be of a higher pressure class.

Water collected at a public standpipe will have to be carried home in a container (bucket, jerry can, vessel, pot, etc.). This means that the water that was safe at the moment of drawing may no longer be so at the moment it is used in the house. Water consumption from standpipes generally is not higher than 20-30 litres per person per

day. This consumption increases when other facilities (e.g. for washing/bathing) are added to reduce the amount of water women and children have to carry home. Water use for other purposes than drinking and cooking is likely to be curtailed when the water has to be fetched from a standpipe. Yard and house connections will usually encourage a more generous water use for personal hygiene and cleaning purposes.

Wastage of water from standpipes can be a serious problem, especially when users fail to turn off the taps. Furthermore, poor drainage of spilled water may cause stagnant pools of dirty water with the associated health hazards. It is also not uncommon for the taps to be damaged by the users and pilferage sometimes occurs. These problems occur particularly when designs do not meet the user requirements, i.e. there has not been adequate consultation with the users (women and men) and/or there are no clear management arrangements. One way to cope with these problems is through payment for water consumed, which is a fair and effective way of water demand management. Often, those selling water are women, as they are chosen for their reliability and trustworthiness, their need to be present for work within their own neighbourhood and their suitability as hygiene promoters with other women and children.

In spite of their shortcomings, group connections and public standpipes are really the only practical options for water distribution at minimum cost to a large number of people who cannot afford the much higher costs of house or yard connections. In fact, housing is frequently not suitably constructed to allow the installation of internal plumbing. It would often be impossible for a small community to obtain the substantial capital for a water distribution system with house connections. Also, the costs of adequate disposal of the considerable amounts of wastewater generated by a house-connected water supply service would place an additional heavy financial burden on the community. Consequently, public standpipes have to be provided and the principal concern should be to lessen their inherent shortcomings as much as possible.

To achieve sustainability of this type of service, the management at the public standpipe level needs special attention in terms of organisation, O&M and cost recovery. Chapter 2 gives more specific information about this.

Staged development of distribution systems

It is possible to develop a water distribution system in stages, upgrading it in steps when a community's standard of living improves and funds become available. This is an important point for community consultation, as the initial cost to each household can be limited while they may foresee a future improvement in service level. When designing the distribution system an allowance should be made for its later upgrading. The design engineer has to take into account the higher per capita water demand associated with better household water supply facilities.

The cost of a water distribution system depends mainly on the total length of pipes installed, and less on the diameters of these pipes. Hence, it can be advantageous to design the major components directly for the ultimate capacity. This is even so when initially only part of the distribution system is installed for supplying water at a few standpipes. Thus, for a start, fairly wide-spaced standpipes are provided that probably can be supplied from one or a few mains. An elevated reservoir (or tank) will be very useful to obtain a reliable feeding of water to the distribution system, particularly if the water is taken from the water source by pumping.

In the next stage additional standpipes will be installed in order to reduce the spacing, and thus the distance the water has to be carried by the users. This may mean laying more distribution mains serving the most densely populated clusters in the community. When this basic level of water service has spread throughout the community, the installation of yard taps and house connections may follow. This will probably be concurrent with the provision of yet more standpipes to improve the service to those users dependent on this type of supply. The distribution of public taps is a very sensitive issue and should always be done in a public and accountable manner to avoid certain groups or individuals dominating the choice. Choosing the possible locations in a social map and agreeing on these locations in a public meeting with a quorum of male and female heads of households helps to curb the influence of the “elite”.

21.3 Design considerations

Water demand variation

The daily water demand in a community area will vary during the year due to seasonal climate patterns, the work situation (e.g. harvest time) and other factors, such as cultural or religious occasions. The typical figures for domestic water usage and other water requirements as given in chapter 4 are the averages. The maximum daily demand is usually estimated by adding 10-30% to the average daily demand. Thus, the peak factor for the daily water demand (k_1) is 1.1-1.3.

The hourly variation in domestic water demand during the day is much greater. Generally, two peak periods can be observed: one in the morning and one late in the afternoon (Fig. 21.9). The peak hour demand can be expressed as the average hourly demand multiplied by the hourly peak factor (k_2). For a particular distribution area this factor depends on the size and character of the community served. The hourly peak factor tends to be high for small villages. It is usually lower for larger communities and small towns. Where roof tanks or other water storage vessels are common, the hourly peak factor will be further reduced. Usually, the factor k_2 is chosen in the 1.5-2.5 range. Where a pipe connection is designed to supply a small group of consumers, a higher value should be adopted because of the effects of instantaneous demand.

A water distribution system is typically designed to cater for the maximum hourly demand. This peak demand may then be computed as $k_1 \times k_2 \times$ average hourly demand.

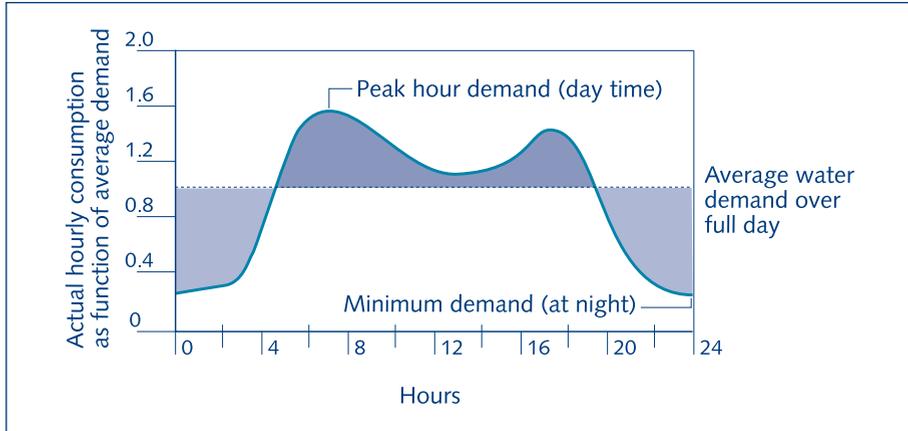


Fig. 21.9. Variation of domestic water demand during the day

Example

For a particular distribution area the average daily water demand is estimated (using the design figures given below) at 500,000 litres per day.

$$\begin{aligned}
 Q_{\text{average day}} &= 500,000 \text{ l/d} \\
 Q_{\text{peak day}} &= 1.2 \times 500,000 = 600,000 \text{ l/d} \\
 Q_{\text{average hour on peak day}} &= 600,000 / 24 = 25,000 \text{ l/h} \\
 Q_{\text{peak hour}} &= 1.8 \times 25,000 = 45,000 \text{ l/h} = 12.5 \text{ l/s}
 \end{aligned}$$

The above calculation reflects the situation of an existing system where the average daily demand is obtained from the field, usually by measuring at supply points. Hence, the registered demand and therefore the peak factors will include leakage.

While developing a new system or extension, the starting point can also be some specific consumption that does not necessarily include leakage. In that case the leakage percentage has to be added in the following way:

$$Q_{\text{peak hour}} = \frac{Q_{\text{average day}}}{f} \left(k_1 k_2 + \frac{l}{100 - l} \right)$$

Factor, f , in the equation is a unit conversion factor while l represents the leakage percentage of the total quantity supplied to the system.

It is common to assume that 'demand = consumption + leakage'. If in the above example $Q_{\text{average day}}$ would represent pure consumption, the peak hour demand with leakage of, say, 20% becomes:

$$Q_{\text{peak hour}} = \frac{500,000}{24 \times 3600} \left(1.2 \times 1.8 + \frac{20}{100 - 20} \right) = 14.0 \text{ l/s}$$

Balancing storage

Without storage of water in the distribution area the source of supply and the water treatment plant would have to be able to follow all fluctuations in the water demand of the community served. This is generally not economical, and sometimes not even technically feasible. The design capacities of the various components of a water supply system are usually chosen as indicated in figure 21.11.

In summary:

System component	Design capacity
Water source	Peak day water demand
Raw water main	Peak day water demand
Treatment plant	Peak day water demand
Transmission main	Peak day water demand
Distribution system	Peak hour water demand

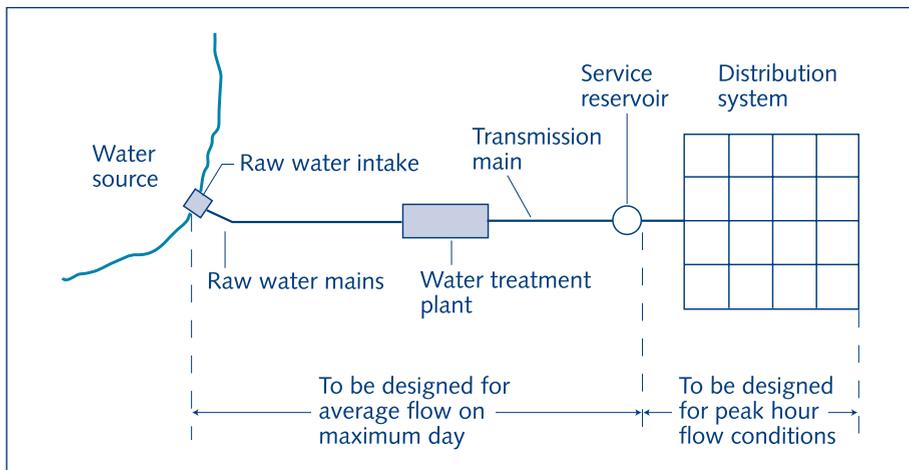


Fig. 21.10. Design capacities for water supply system components

The service reservoir is provided to balance the (constant) supply rate from the water source/treatment plant with the fluctuating water demand in the distribution area. The storage volume should be large enough to accommodate the cumulative differences between water supply and demand.

The required storage volume can be determined as follows. The estimated hourly water demand (example given in figure 21.9) is expressed as a percentage of the total demand over the peak day and plotted in a cumulative water demand curve (Fig. 21.11).

The constant supply rate is then drawn in the same diagram as a straight line¹.

1 In the example the supply operates at a constant rate. If the supply rate is not constant, a broken line will represent the cumulative quantity of water supplied.

The required volume of storage can now be read from the graph. For a constant-rate supply, 24 hours a day, the required storage is represented by A-A' plus B-B', which is about 28% of the total peak day demand. This figure is directly proportional to the two shaded areas in figure 21.9. The balancing volume available at the beginning of the day should equal the B-B' percentage of the peak day demand. In that way, the tank will be full at the moment the A-A' percentage has been added to it and empty i.e. at the reserve volume when the B-B' deficit has been reached. These two moments occur, for example, in figure 21.11 after four hours and 20 hours, respectively. Based on the shape of the tank the expected water level variation in the tank can also be determined. If the supply capacity is so high that the daily demand can be met with 12 hours pumping a day, the required storage is found to be C-C' plus D-D', in this case about 22% of the total peak day demand. However, if the same pumping takes place overnight or in intervals (in order to reduce the load on the electricity network, i.e. save by pumping at cheaper tariff), the required balancing volume will have to be much larger. For example for pumping between 6 p.m. and 6 a.m., C'-C'' + D'-D'' \approx 76%.

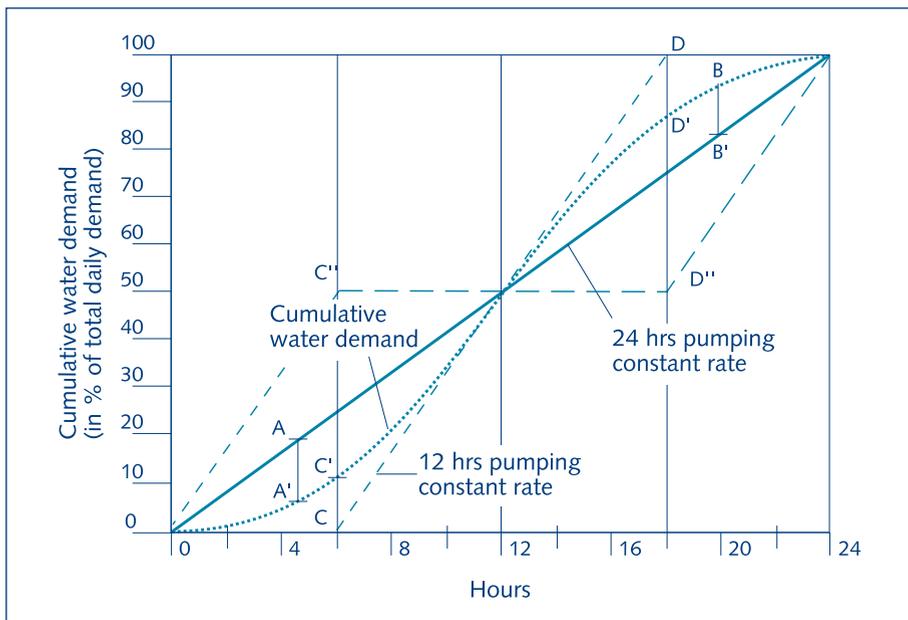


Fig. 21.11. Graphical determination of required storage volume (service reservoir)

A service reservoir with a storage volume of 25-40% of the peak day water demand should generally be adequate. Yet, a larger reservoir may be called for in situations where any interruption of the water supply would be particularly critical.

Any reservoir should be situated at a higher elevation than the distribution area, but as close to it as possible. If such a site is available only at some distance, the reservoir has to be placed there. Figure 21.12 shows two possible arrangements.

In flat areas where no suitable hill sites or other high points for ground reservoirs are available, elevated tanks have to be used. In theory such storage should have the same balancing volume as the ground reservoir. In practice, however, these are usually costly solutions. Alternatively, more water is pumped directly into the network, in combination with an elevated tank of much smaller size (Fig. 21.13). These tanks, commonly called water towers, are less than 1000 m³ and serve for minor balancing of demand variations. In essence, their prime purpose is to stabilise pumping station operation and pressures in the system. They also serve as a protection against water hammer or as a temporary buffer in cases of power failure before a diesel generator can be switched on (if one is available).

A set-up with a ground reservoir on the suction side of the pumping station can also be used (Fig. 21.14). It allows for stable production at the source and also creates a buffer in case this production is temporarily terminated. This, however, is too complex an arrangement for small community supplies.

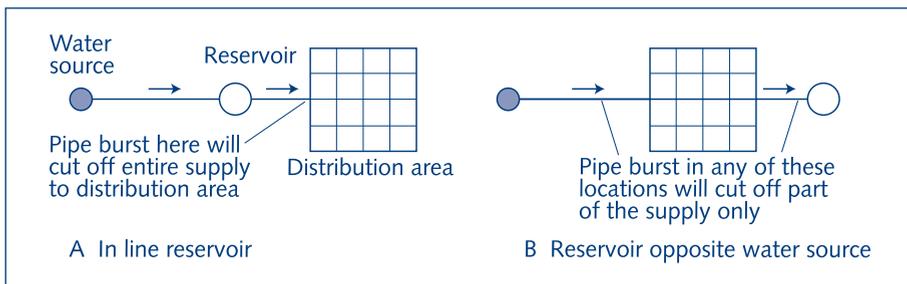


Fig. 21.12. Reservoir siting

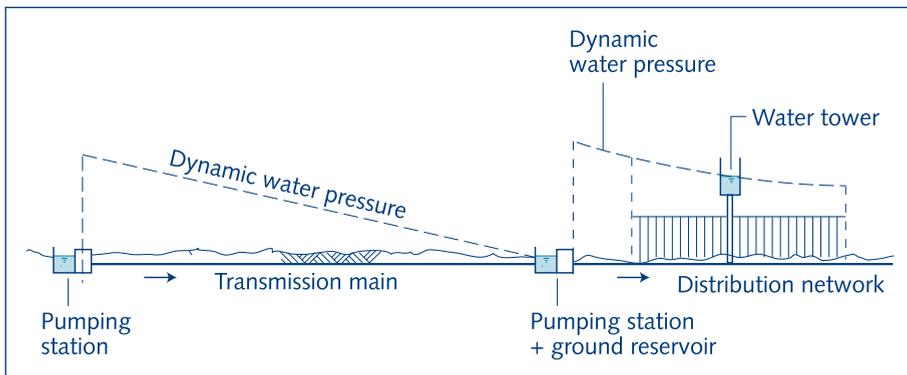


Fig. 21.13. Ground reservoir with pumping station combined with water tower

Sizeable ground reservoirs, of a few thousand cubic metres, are normally made of reinforced concrete. Smaller ones can be of mass concrete or brick masonry. Water towers are made of steel, reinforced concrete or brickwork on concrete columns. Steel tanks are mostly placed on a steel or wooden support framework.

Examples of small service reservoirs are shown in figures 21.14 and 21.15.

An elevated service reservoir (steel tank on brick masonry support) is shown in figure 21.16. Figure 21.17 features a reinforced brickwork tank supported by masonry walls.

21.4 Distribution system design

Design parameters

Design criteria for pressures, velocities and hydraulic gradients are similar to those for transmission pipes discussed in chapter 20, but set in a bit wider range because of demand variations.

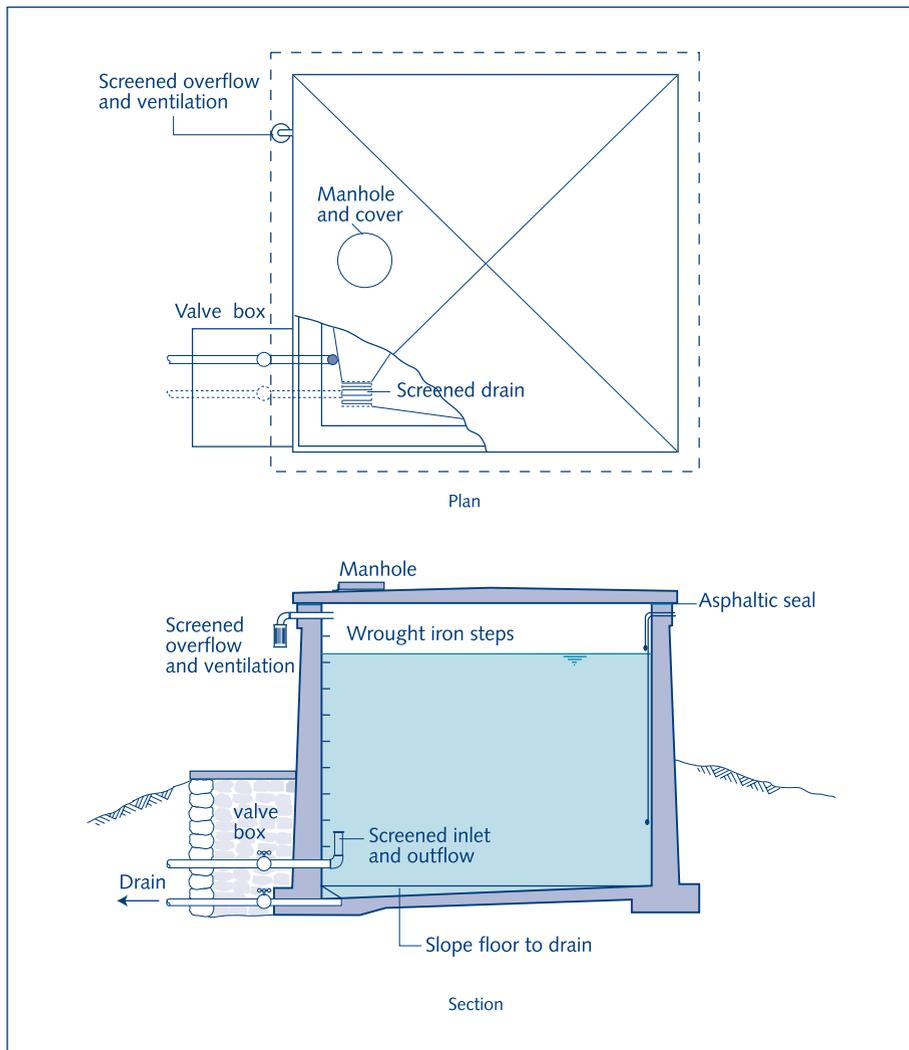


Fig. 21.14. Construction details of small reservoir

A typical range of velocities in distribution pipes is between 0.5 and 1.0 m/s, occasionally up to 2 m/s. The hydraulic gradients usually range between 1 and 5 m/km, occasionally up to 10 m/km. In case of smaller pipes, $D < 50$ mm, the hydraulic gradient can even be higher.

The pressure criterion is dependent on topographic conditions, availability of water at the source and overall condition of the pipes. The minimum pressures should not drop below 5-10 mwc. In larger distribution areas where water scarcity is not an issue, the minimum pressures can range between 20 and 30 mwc above street level, where there are house connections. This is sufficient for supply of 2-3 storey buildings.

Pressures higher than 60 mwc should be avoided in general, due to increased leakage and risks of bursts, especially in poorly maintained networks.

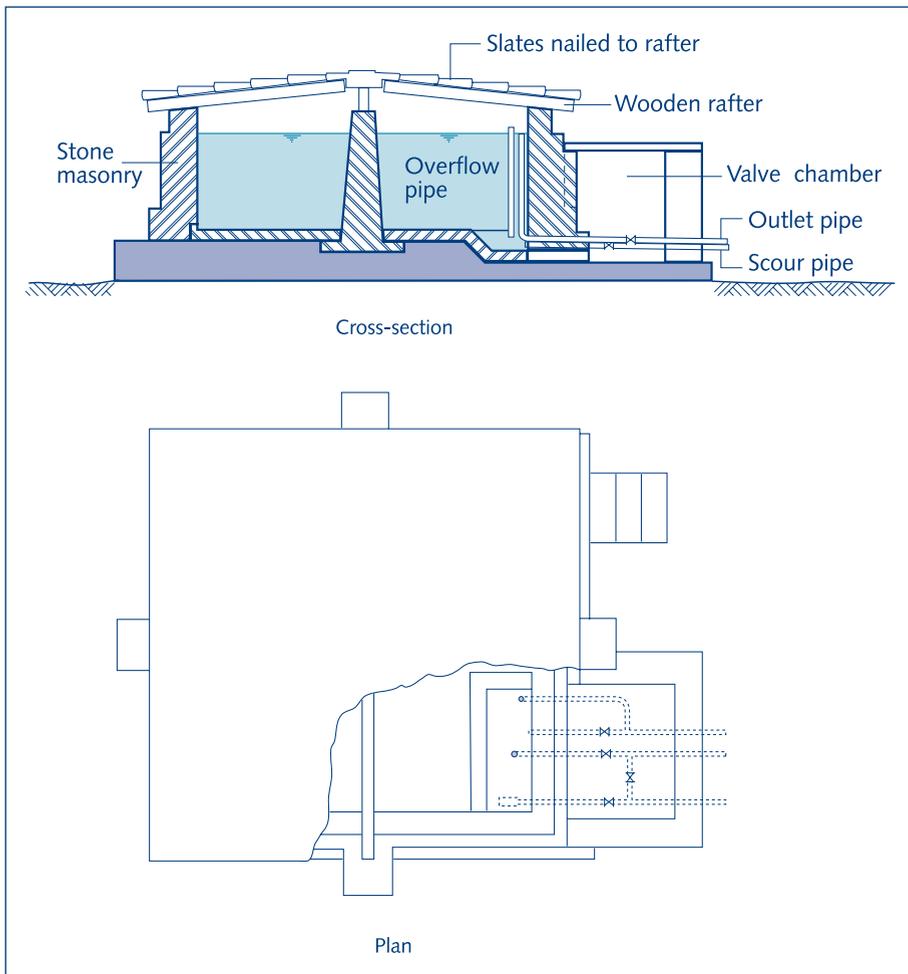


Fig. 21.15. Small service reservoir

Layout and nodal demands

After choosing a preliminary supply scheme (gravity/pumping) and type of network layout (branched/looped) with its main components, the distribution area is divided into a number of demand districts according to the topography, land use classification and density of population. Boundaries may be drawn along rivers, roads, high points or other features that distinguish each district. The secondary mains and distribution pipes can then be plotted on the plan.

Once all the sectors are fixed, the population for each district can be estimated or computed from data collected in the community involvement process. The water demand per district is then computed using per capita water usage figures for domestic water consumption and selected values for the other, non-domestic water requirements (including productive uses). This per capita usage figure may vary depending on the water service level that the community in the concerned district has decided on.

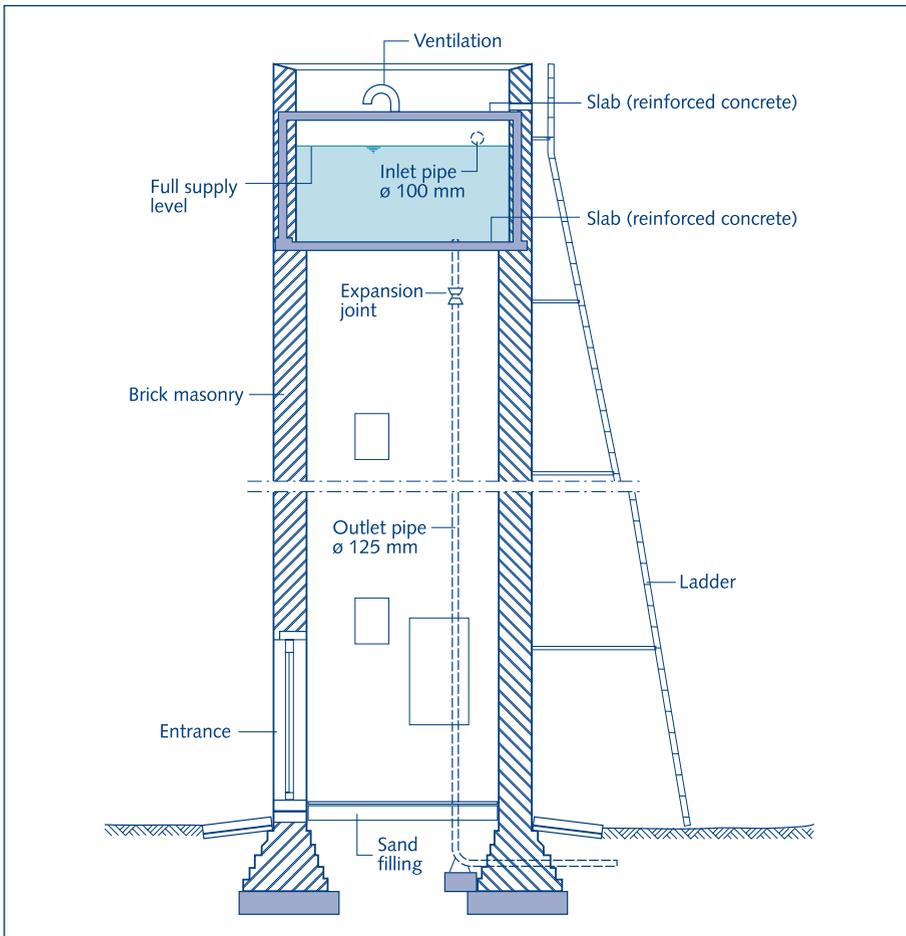


Fig. 21.16. Elevated service reservoir made of steel

The community process that can be followed, also to get realistic figures on water demand, is described in chapter 2. A certain leakage percentage is to be assumed too; some 10% of the water production is a fair assumption in the case of new systems.

Although in reality the water will be drawn off at many points along the length of the pipes, it is common engineering practice to assume that all demand is concentrated in the nodal points (i.e. pipe junctions) of the distribution network. The hydraulic calculation is much simplified by this assumption and the errors so introduced are acceptable.

For preliminary calculation of the nodal demands, a simple method may be employed using the water consumption rate per linear metre of distribution pipe. This rate is, of course, greatly influenced by the type of discharge points: public standpipes, yard taps,

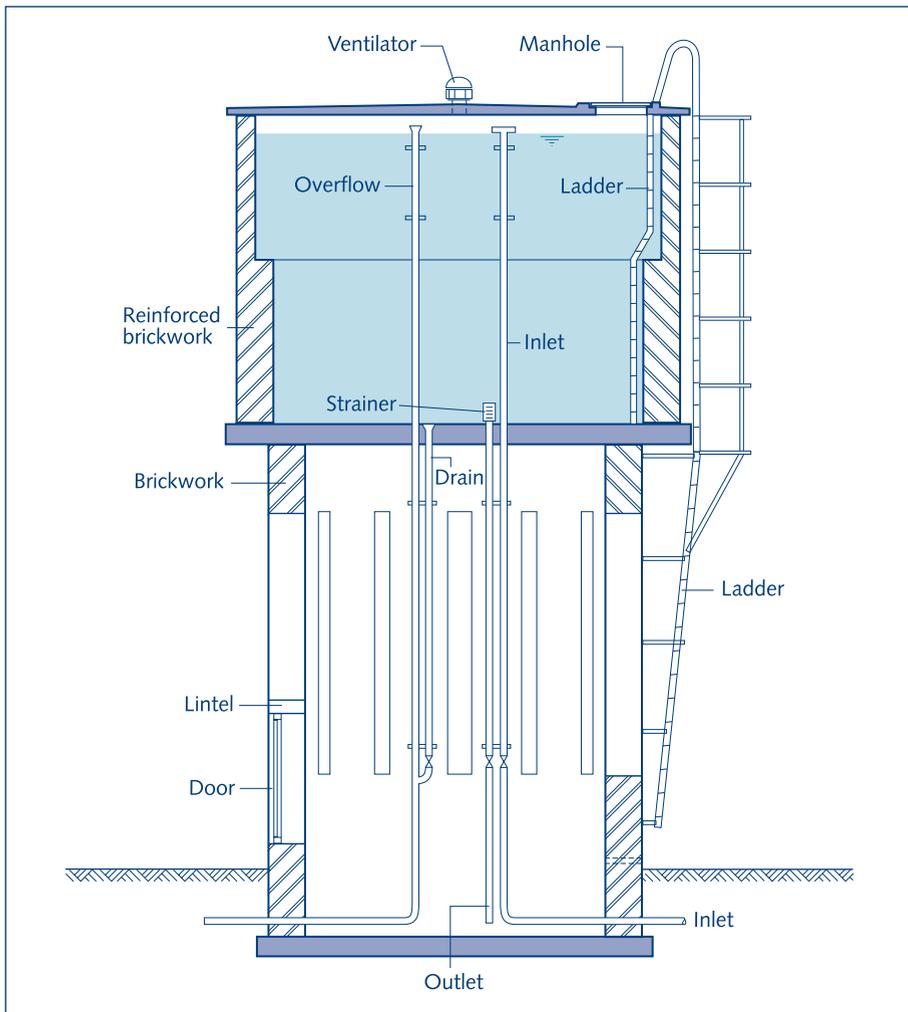


Fig. 21.17. Elevated service reservoir made of concrete

house connections, or combinations of these. Once the total demand alongside each pipe has been determined, it is going to be split between the two corresponding nodes. This approach, based on the assumption of even dispersion of the service connections, makes a good starting point for hydraulic calculations. How much it is really true is to be checked in the field. In addition, a separate demand node should be assigned to any major user in the system (school, hospital, etc.).

Having calculated demands at the nodal points, the required pipe diameters can be estimated. Based on the demand distribution, the main pipes should be laid towards the areas of higher demand, usually alongside the main traffic routes in the area.

Hydraulic calculation

If the network is of branched structure, the flows and their directions can easily be determined assuming the flow continuity in each node. For node n that is a junction of j pipes:

$$\sum_{i=1}^j Q_i - Q_n = 0$$

where Q_i is the pipe flow and Q_n the nodal demand. The equation is basically saying that the sum of all inflows and outflows in each node equals zero.

When the flow in each pipe is determined, the diameters will be adopted based on the hydraulic properties of the available manufactured sizes. The head loss calculation has to be done according to the principles explained in chapter 20. Lastly, the pressures in the system will be determined, either starting from the minimum required pressure in a specified (critical) node or from the known head at the supply point.

The following example illustrates this simplified design method (Fig. 21.18).

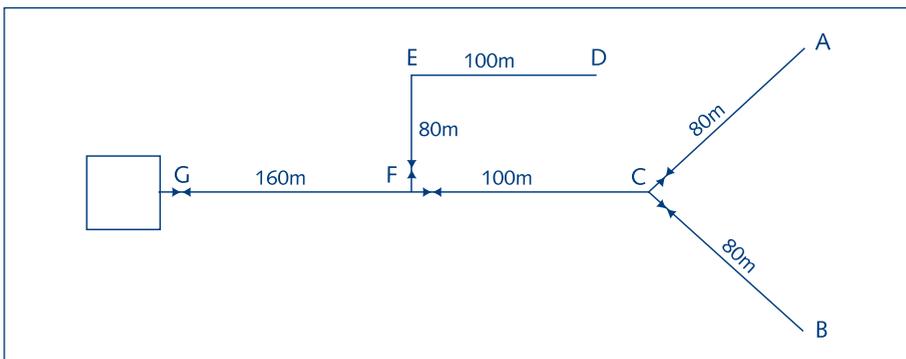


Fig. 21.18. Simple water distribution system (schematic)

Design data

Number of persons served:	1750
Total length of pipes:	600 m
Specific demand:	50 litres per person per day
Daily peak factor (k_1):	1.2
Hourly peak factor (k_2):	2.0

Calculation

Average flow carried by the distribution system: $Q_{\text{avg}} = 1750 \times 50 = 87500 \text{ l/d} = 1.0 \text{ l/s}$.

Peak flow carried by the system: $Q_{\text{max}} = 1.2 \times 2.0 \times 1.0 = 2.4 \text{ l/s}$.

Water use rate per length of distribution pipes: $q = 2.4/600 = 0.004 \text{ l/s/m}^1$.

Multiplying the length of each pipe by the unit flow rate gives the tentative demand along the pipe (table 21.1a).

Table 21.1a Tentative determination of pipe demands

Sections	Length L (m)	Piped demand $q \cdot L$ (l/s)
A-C	80	0.32
B-C	80	0.32
C-F	100	0.40
D-E	100	0.40
E-F	80	0.32
F-G	160	0.64
Total	600	2.40

Splitting the pipe demand between the corresponding nodes yields the nodal demands.

Table 21.1b Nodal demands

Node	A	B	C	D	E	F	G	Total
Q_n (l/s)	0.16	0.16	0.52	0.20	0.36	0.68	0.32	2.40

Consequently, the pipe flows will be determined from the continuity equation.

Table 21.1c Pipe flows

Pipe	A-C	B-C	C-F	D-E	E-F	F-G
Q (l/s)	0.16	0.16	0.84	0.20	0.56	2.08

In the next step the maximum carrying capacity can be computed for selected design velocity and range of available (manufactured) pipe diameters: $Q_{\max} = v \times D^2 \pi / 4$. In this example the maximum flow carried by the pipes for a design velocity of 0.75 m/s is shown in table 21.2. Comparing the pipe flows in table 21.1c with the maximum flows in table 21.2 leads to the selection of diameters. The hydraulic calculation proceeds with the adopted D-values. The results in table 21.3 are obtained by using the Darcy-Weisbach formula (see chapter 20), for $k = 0.1$ mm and $T = 20^\circ$. Finally, the pressures in the nodes will be calculated for known elevations (in metres above sea level), and assumed head at the supply point of 50 mwc (table 21.4).

Table 21.2 Pipe flow (for $v = 0.75$ m/s)

D (mm)	D (inch)	Q_{\max} (l/s)
30	1¼	0.53
40	1½	0.94
50	2	1.47
60	2½	2.12
80	3	3.77
100	4	5.89

Table 21.3 Head loss calculation ($k = 0.1$ mm, $T = 20^\circ$)

Sections	Pipe Flow Q (l/s)	Adopted D (mm)	Velocity v (m/s)	Hydr.grad. S (m/km)	Head loss SxL (mwc)
A-C	0.16	30	0.23	3.38	0.27
B-C	0.16	30	0.23	3.38	0.27
C-F	0.84	40	0.67	16.95	1.70
D-E	0.20	30	0.28	5.05	0.50
E-F	0.56	40	0.45	7.97	0.64
F-G	2.08	60	0.74	12.10	1.94

Calculation of looped systems is much more complicated, due to the fact that the flow rates and directions in the pipes are not initially known. Smaller looped networks can be disintegrated into branched type layouts by disconnecting selected peripheral pipes from some junctions (Fig. 21.19). This "imaginary disconnection" is done only for the sake of easier calculation and will result in smaller diameters for those pipes.

Table 21.4 Determination of pressures

Nodes	Elevation z (msl)	Head H (mwc)	Pressure p (mwc)
A	24.8	46.09	21.29
B	22.2	46.09	23.89
C	17.5	46.36	28.86
D	20.0	46.92	26.92
E	28.2	47.42	19.22
F	35.6	48.06	12.46
G (Source)	-	50.00	-

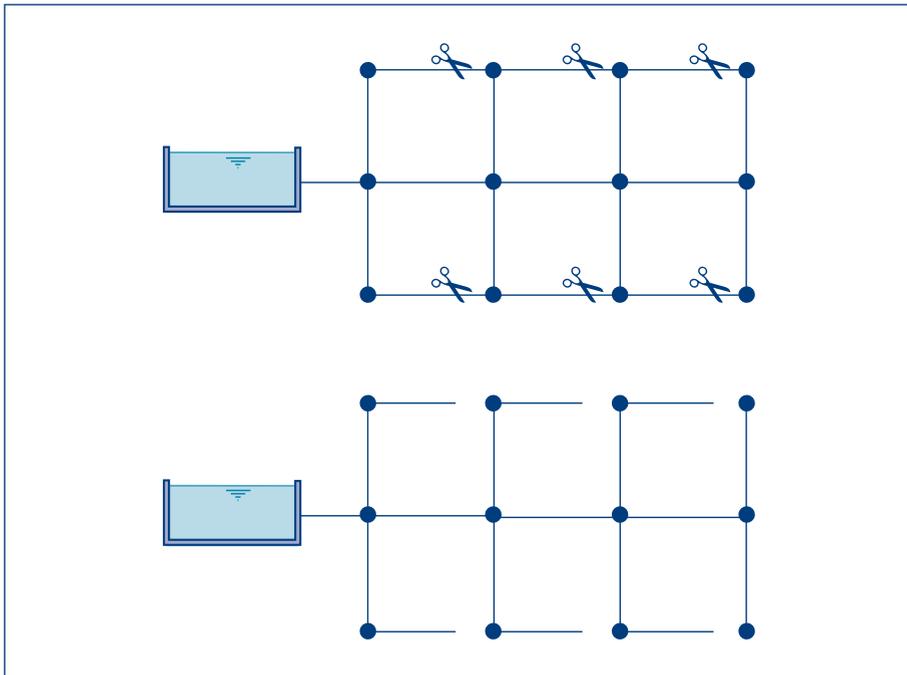


Fig. 21.19. Imaginary branching of looped layouts

Larger systems have to be considered as being looped. Apart from the continuity and head loss equation, an additional equation is required in this case:

$$\sum_{i=1}^j \Delta H_i = 0$$

The equation reflects the continuity of the hydraulic grade line in each loop of the network, i.e. the sum of the head losses being equal zero.

The methodology of the calculation of the looped networks was first developed by Hardy-Cross. In short, it consists of the following steps:

1. Estimated flows are initially assigned to each pipe, fitting the continuity equation in all nodes, as well as possible.
2. Based on these flows, the head loss is calculated in each pipe, as described in chapter 20.
3. The sum of the head losses along each loop is checked, respecting the orientation of flows (e.g. clockwise).
4. If the sum at any loop is out of requested accuracy $0 + \sum \Delta H$, the following flow correction is to be introduced for each pipe of that loop:

$$\delta Q_j = \frac{-\sum_{j=1}^n \Delta H_j}{2 * \sum_{j=1}^n \left| \frac{\Delta H_j}{Q_j} \right|}$$

5. The iteration procedure is then repeated with the new flows $Q + \delta Q$, until the accuracy is satisfied for all loops.
6. After the iteration of flows and head losses has been completed, the nodal pressures can be determined from the reference pressure.

Obviously the manual calculation is very time consuming even with only a few-loop system. Luckily the development of computer technology has allowed such iteration methods to be programmed, and the Hardy-Cross method was further developed as a basic algorithm for computer calculations. Since the 1990s, this has opened new possibilities for design of distribution networks through application of computer models.

Use of computer models

Widespread introduction of personal computers has enhanced hydraulic design of distribution networks. Commercial programmes available on the market, sometimes even free of charge, enable very precise and quick calculations, which makes them equally suitable for the design of simple rural systems or large urban networks of a few thousand pipes. Accessibility of such software and PCs to the engineers of developing countries has been significantly improved since the mid-nineties.

The computer programmes in use are all pretty similar in concept, with the following common features:

- PC-Windows™ based applications
- Allow extended period hydraulic simulations
- Possess integrated module for water quality simulations
- Handle virtually unlimited size of the network in any configuration
- Have excellent graphical interface for presentation of results

The main distinctions between these programmes are in specific formats of input data used, as well as in the way the calculation results are processed.

The modelling process consists of the following steps:

1. Input data collection
2. Network schematic
3. Model building
4. Model testing
5. Problem analysis

Input data collection

Powerful computational tools have enabled the focus to be shifted from the calculation to the collection of reliable input data. High quality information about demands, system dimensions and materials is crucial for accurate results. Operation and maintenance data are important too, in the case of rehabilitation of existing systems. This is often a limitation for the model. Well-conducted fieldwork data collection is therefore a very important initial step of the modelling procedure. The information to be investigated is listed below.

1. General

General layout of the network is taken from the maps available in the water distribution company. The specific data to look for are:

- 1.1 Topography - ground elevations in the area of the system; some specific natural barriers.
- 1.2 Type of the system - distribution scheme: gravity, pumping, combined; location and role of each system component.
- 1.3 Population - distribution and estimated growth.

2. Water demand

- 2.1 Demand categories present in the system: domestic, industry, tourism, etc.
- 2.2 Average consumption, patterns of variation: daily, weekly and seasonal.
- 2.3 Type of domestic water use: direct supply, roof tanks, etc.; average household size; habits with respect to the water use.
- 2.4 Demand forecasting.

3. Network layout

- 3.1 Nodes (discharge points) - concerns predominantly the supply points of at least a few hundred consumers or major industry. Relevant for each point are
 - location (X,Y) in the system
 - ground elevation (Z)
 - average consumption and dominant category(ies)

- 3.2 Pipes - concerns predominantly the pipes, $D > 50$ mm. Relevant for each pipe are
 - length
 - diameter (internal)
 - material and age
 - assessment of corrosion level (k or C value, if available)
- 3.3 Service reservoirs - type (ground, elevated), capacity, minimum and maximum water level, shape (e.g. through the "volume-depth" curve).
- 3.3a Individual roof tanks (where applicable) - type and height of the tank, capacity, inflow/outflow arrangement, average number of users per house connection, description of house installations (existence of direct supply in the ground floor).
- 3.4 Pumping stations - number and type of pumps; duty head and flow and preferably the pump characteristics for each unit; age and condition of pumps.
- 3.5 Others - description of appurtenances that may significantly influence the system operation (e.g. valves, measuring equipment, etc.).

4. System operation & monitoring

Important (and preferably simultaneous) measurements for calibration of the model are:

- pressure in a few points covering the entire network
- level variations in the service reservoirs and roof tanks (where applicable)
- pressures and flows in the pumping stations
- flows in a few main pipes in the network
- valve operation (where applicable)

All this information may not be easy to collect. However, some knowledge about the system should exist, even in descriptive form. For instance, in which period of the day is a certain reservoir empty (full), a certain pump on (off), a certain valve open (closed), a certain consumer with (without) water or with (without) sufficient pressure, etc. Where there is a possibility of continuous measurements, typical days should be compared: the same day of the week in various seasons, or various days of the week in the same season.

5. System maintenance

Type of maintenance, water metering, the unaccounted-for water level and sources (leakage, faulty water meters, illegal connections, etc.), water quality in distribution network.

6. Water company

Organisation, facilities, practice, plans for future extension of the system. This information can be used to opt for a certain degree of reliability i.e. reserve capacity in the system. It may also have implications on the way the phased development should be approached. Some components of the system can deliberately be oversized to cater for future development of particular areas or connection to a new water source.

Network schematic

Hydraulic calculation of looped networks is based on systems of equations with a complexity directly proportional to the size of the system. Thus, some schematisation (also called skeletonisation) is necessary up to the level where the model accuracy will not be substantially affected, enabling quicker calculations at the same time.

The answers should be reached with the minimum necessary number of pipes and nodes. Expanding the model by the system parts that have no significant impact on its operation only adds to the calculation time and the volume of printout.

Model building

Just as in reality, it is advisable to build the network model in stages, increasing the level of detail gradually. Starting immediately with the full-size network with all components included will almost certainly yield lots of troubles during the testing of the model.

In many cases the input file format has to be obeyed strictly; this is the only code the programme can understand while reading the data. Making errors during the model building is therefore common. Some programmes allow input in an interactive way so the chance of false network configuration will be diminished. In other cases the error check and warning messages will be activated after running the calculation.

Model testing

Once the first simulation run is completed the immediate concern is whether the result matches the reality. In this phase several runs have to be executed that should confirm that

- the model gives a logical response to any altering of the input data (model validation)
- the model's behaviour corresponds to the reality (model calibration)

The reasons for any inaccuracy may be that

- some input data were badly estimated because of lack of the field data
- the network is wrongly interpreted by the model due to possible typing errors, omitted data or inaccurate field measurements
- format of the input file is incorrect but the error is 'unknown' to the error library of the programme

It should never be forgotten that the computer models cannot match the real situation totally; the results should be judged based on the quality of the input data and the measurements used for model calibration.

Problem analysis

With correct execution of all other steps, the real analysis of the problem is the final and the shortest step. After a few simulations a well-managed model will show the full picture of the selected alternative.

One of the most popular computer programmes in the market at present is EPANET made by the US Environmental Protection Agency. The programme possesses all of the above listed features and can be downloaded from the Internet, free of charge. The site can be accessed through any search engine by typing keyword 'epanet'.

Graphical layout of the simulation results in EPANET is shown in figure 21.20.

21.5 Operation and maintenance of distribution systems

Unaccounted-for water

The amount of water that can be billed will always be smaller than the amount supplied. Moreover, the water actually passing through the taps is also smaller than the amount supplied, be it charged or not. The difference in the first case refers to the unaccounted-for water (UFW) while the second one represents leakage. Thus, the leakage is a component of UFW. Other important sources can be faulty water meters, illegal connections, poor education of consumers, etc.

UFW is an important element of water demand and a great concern of many water companies. In some systems UFW is the most significant "consumer", reaching up to 50% of the total water supply. There can be various ways of fighting this problem but due to high costs of such programmes the real consideration tends to start only when the UFW levels exceed 20-30%. Limited capacity of the source can also be an important factor in such cases. Water conservation is increasing in importance as more and more regions begin to experience serious water shortages, and reducing UFW is a good way to start.

Leakage is usually the most significant contributor to high UFW levels. The factors influencing leakage are

- soil characteristics, soil movement, traffic loading
- defects in pipes, poor quality of joints
- poor quality of workmanship, damage due to excavation for other purposes
- pipe age and corrosion level
- high pressures in the system
- extreme temperatures

There are a variety of managerial and technical ways to address the UFW problem. Managerial measures include the following:

- Regular checks by caretakers or by alerts from consumers for pipeline damage, leakage and illegal connections
- Regular checks by the water point committee, caretakers and technicians on the quality and leakage of connections (also the presence of illegal connections), meters (if present) and taps

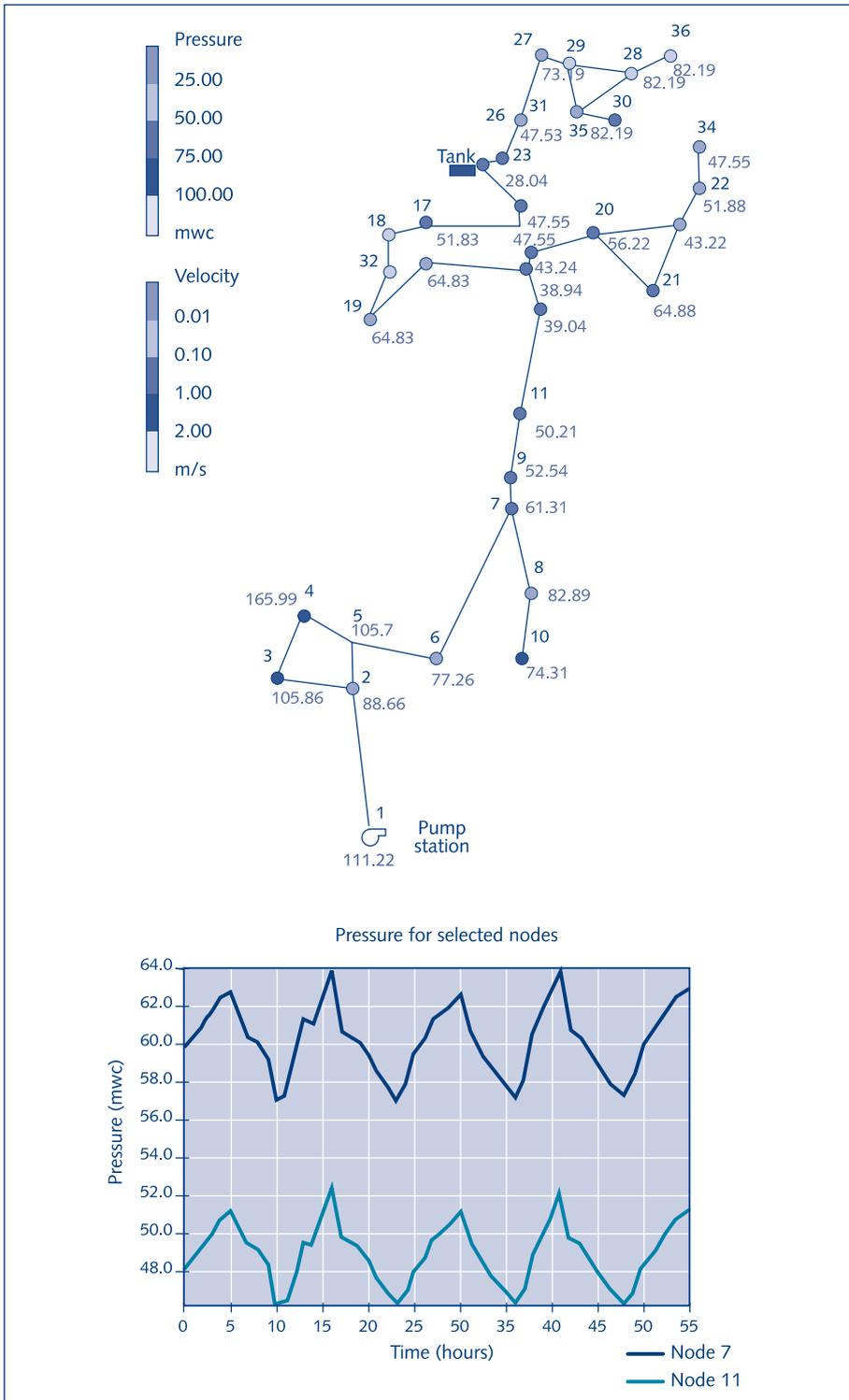


Fig. 21.20. Layout of the EPANET calculation results

- Arrangements at the water-vending points, public taps and group connections to minimise the waste of water by good caretaking

Global estimates of leakage levels come from an annual balance of the delivery and metered consumption for the whole network. Bursts of main pipes can be detected by the flow measurements at supply points (Fig.21.21).

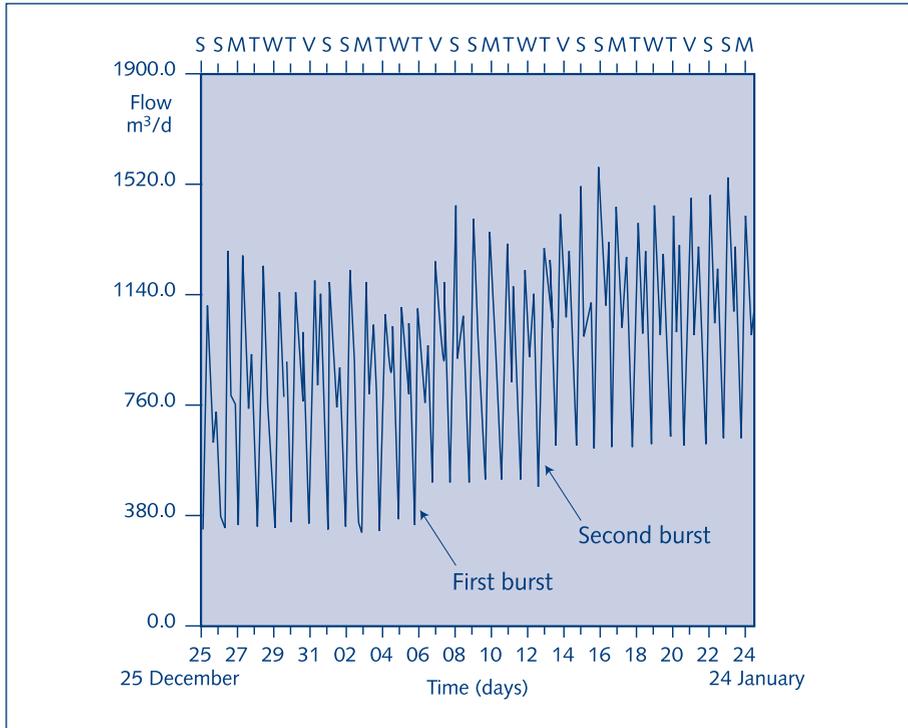


Fig. 21.21. Leak detection from the flow measurements

For more detailed analysis of the leakage, suspected parts of the system have to be inspected during several hours or days, depending on the size of controlled area. Those temporary measurements are usually carried out overnight, when real consumption and overall noise level are minimal. The area is isolated from the rest of the system by closing the border valves and its inflow and outflow are measured. Some knowledge about the actual night consumption should exist in advance; flows detected on top of that are part of the UFW, mostly leakage.

Measurements are repeated at weekly intervals for a period of a few months. Any pipe burst between two measurements would be reflected in sudden increase in registered demand.

Average leakage level can also be estimated by monitoring pressures in the system. A sudden drop of pressure could also indicate a major pipe failure. Under normal conditions night pressures should be kept as low as possible in order to reduce the leakage levels.

Finding a precise leak location can be a difficult problem. In case of severe breaks the water may appear on the surface and the exact position of the leak can be determined by drilling test holes alongside the pipe route. If the leak is not visible on the surface, leak detection equipment has to be used. The most common devices are an acoustic (sound) detector and a leak noise correlator.

Acoustic detectors rely on sounding directly on the pipe or fitting, or indirectly on the ground surface. The noise generated from the leak is transmitted by the receiver attached to a stick, to the amplifier connected to a stethoscope (Fig. 21.22). This method is not always reliable; leaks at lower pressures and specifically those in plastic pipes may produce undetectable noise. However, with skilled personnel working under silent (night) conditions, some 70-80% of the leaks should be discovered.

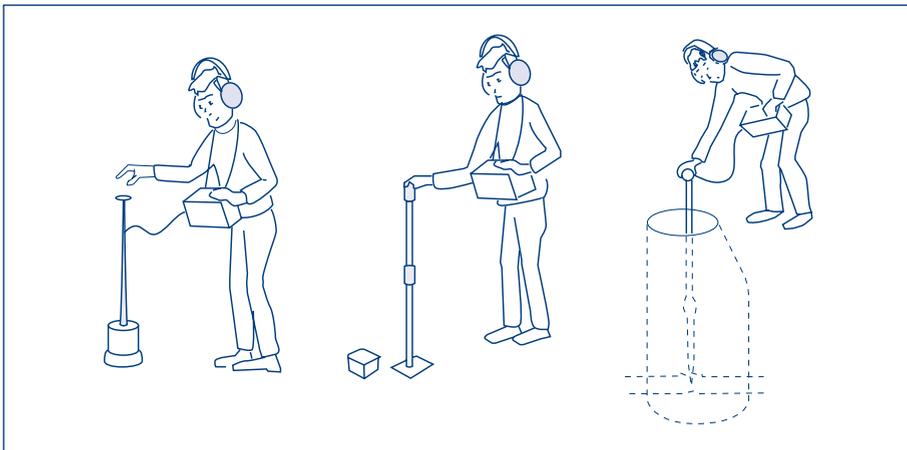


Fig. 21.22. Leak location by using sound detector

Leak noise correlators detect the exact burst location by registering the noise spreading through the water. By placing microphones at the ends of the controlled pipe section (up to a few hundred metres), the difference in time required for the leak noise to reach the microphones can be measured. The leak position can then be calculated from the known length of the section. This method is very effective in detecting leaks under background noise levels. However, it may be less accurate when being used in sections with plastic pipes or with more than one leak.

Faulty water meters are the second main source of unaccounted-for water. Typical water meters register flows with average accuracy of about 2%, when they are new. However, this error becomes higher for small flows, below 50 l/h. When not properly maintained the water meter may register flows with errors between 20 and 40% after a couple of years in service. This lack of accuracy can cause serious revenue losses.

Complicated methods of monitoring and leak detection would normally not be employed in small community water supply schemes. They require expensive equipment and trained personnel. Even so, installing at least a few measuring devices at the right points in the network may be of great help in collecting information about the system operation. The minimum is to have flow and/or pressure meters in the pumping stations. Water levels in the reservoirs should also be observed at regular intervals during the day. Ideally, a few pressure gauges should be installed within the network.

Corrosion

Corrosion of metal pipes is one of the major causes of poor operation of water distribution systems. It appears as a result of reaction between the water and metal. This internal corrosion causes three problems:

- The pipe mass is lost through oxidation to soluble iron, resulting in increased rate of pipe bursts
- The second by-product of oxidation is iron-bearing scale that accumulates on the pipe wall in the form of tubercles, causing a reduction of pipe capacity (head loss increase)
- Both soluble and particulate iron affects the water quality creating colour problems ('brown' or 'red' water)

External corrosion is a result of aggressive soils and may also contribute to the pipe burst rate to a large extent.

To reduce corrosion levels, metal pipes need to have internal and external coatings. Ductile iron and steel pipes are normally delivered with internal cement lining and external coatings made of plastic, epoxy, or bitumen. Steel pipes in pumping stations will usually be protected by painting. Handling of pipes during transport and laying has to avoid damage to the coatings. Once in service the cement lining can be dissolved due to leaching of calcium at low pH values. High turbulence or sudden change of the flow direction at high velocities may also damage the coating.

Water quality adjustment is the easiest and most practical way to make water non-corrosive. However, it is not always effective because of possible differences in water quality at the sources. Two basic methods are pH correction and oxygen reduction. Chemicals commonly used for pH adjustment are lime, caustic soda or sodium

(bi)carbonate. Oxygen removal is rather expensive but some control measures can be introduced through optimisation of aeration processes and sizing of well and distribution pumps that will avoid air entry.

Other options, such as adding inhibitors or cathodic protection of the pipes are too complex and expensive for small distribution schemes.

Bad design of the pipes and structures may cause severe corrosion even in materials that are highly resistant. Some of the important design considerations include

- selection of appropriate flow velocity
- selection of appropriate metal thickness
- reduction of mechanical stresses
- avoiding sharp bends and elbows
- avoiding grounding of electrical circuits to the system
- providing easy access to the structure for periodic inspection, maintenance and replacement of damaged parts

Pipe cleaning and disinfection

Corrosion deposits in pipes or sediments caused by improper treatment have to be removed to prevent water quality deterioration. Three techniques commonly used are flushing, swabbing and air scouring. These techniques may greatly help to improve the water quality but additional maintenance equipment has to be employed. Moreover, the network layout needs to include a number of hydrants or washouts to connect this equipment.

Flushing is the simplest method of cleaning but with some disadvantages:

- Large amounts of water used (particularly in large diameters)
- It is less effective in low pressure areas
- It may disturb flow patterns upstream of the cleaned section
- In areas with active corrosion, it offers only a temporary improvement

The efficiency of flushing can be increased by injection of compressed air into a continuous but smaller flow of water. Pushed by the air, the water will form into discrete slugs forced along the pipe at high velocities. The length that can be effectively cleaned by air scouring depends on:

- available static pressure (higher pressure - longer length)
- pipe friction (higher friction - shorter length)
- compressor size (larger compressor - longer length)
- pipe gradient (longer lengths when cleaning uphill)

Swabbing is a technique where a cylindrical swab is inserted into the pipe and driven along by the water pressure pushing the deposits ahead. The swab is porous and allows about 10% of the water flow to pass, which aids the transport of the deposits. Swabs are usually made of polyurethane of different hardness and construction for various degrees of reduction in pipe cross-section. Recommended travelling speed of the swab is 0.3-1.2 m/s.

The problem of animal populations appearing in water distribution systems is predominantly aesthetic and it is therefore a matter of maintaining it at such a level that the consumer is unaware of their presence. Pipe disinfection can be done either by cleaning or by chemical treatment. Swimming animals can be removed relatively easily by flushing. Chemical treatment is carried out where the flushing is insufficient. The chemicals commonly used are chlorine, pyrethrins and permethrin. Pyrethrins and permethrin are toxic to fish, so they should be used and disposed of very carefully.

When using chlorine, higher concentrations are required than the normal dosages in water leaving the treatment plant. The concentrations applied during pre-chlorination may be effective in reducing animal appearance in the treatment works. An infestation in the distribution system can be controlled in most cases by maintaining 0.5-1.0 mg/l of residual chlorine for a week or two.

Long-term measures include removal of organic matter (restricting nutrients for the animals), which can be achieved by the following methods:

- Improvement of the treatment process regarding suspended solids removal and animal penetration
- Periodic cleaning of pipes and service reservoirs
- Maintenance of a chlorine residual throughout the distribution system
- Proper protection of openings on service reservoirs
- Elimination of dead ends and stagnant waters where ever possible.

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22 Technologies for fluoride removal

Leela Lyengar

22 Technologies for fluoride removal

22.1. Introduction

Fluoride is a normal constituent of natural water samples. Its concentration, though, varies significantly depending on the water source. Although both geological and man-made sources contribute to the occurrence of fluoride in water, the major contribution comes from geological resources. Except in isolated cases, surface waters seldom have fluoride levels exceeding 0.3 mg/l. Examples are streams flowing over granite rich in fluoride minerals and rivers that receive untreated fluoride-rich industrial wastewater. There are several fluoride bearing minerals in the earth's crust. They occur in sedimentary (limestone and sandstone) and igneous (granite) rocks. Weathering of these minerals along with volcanic and fumarolic processes lead to higher fluoride levels in groundwater. Dissolution of these barely soluble minerals depends on the water composition and the time of contact between the source minerals and the water.

Over the years groundwater has generally been considered to be a protected and safe source of water, fit for drinking without treatment, as the main focus has been on the bacteriological quality of potable water. Little consideration used to be given to the risks of chemical pollution, particularly to the presence of elevated levels of fluoride, arsenic and nitrate in groundwater. This chapter deals with only fluoride. Consumption of water having excess fluoride over a prolonged period leads to a chronic ailment known as fluorosis. Incidence of high-fluoride groundwater has been reported from 23 nations around the globe. It has led to endemic fluorosis, which has become a major geo-environmental health issue in many developing countries. According to a recent estimate, 62 million people are affected by various degrees of fluorosis in India alone (Susheela, 2001).

22.2. Health impacts of excess fluoride in potable waters

Low dental caries incidence rates demonstrate that fluoride concentrations of up to 1.0 mg/l in potable water are beneficial to the oral health of children and, to a lesser extent, adults. In several developed countries fluoridation of water supplies is practised if the natural concentration is below the desired level. Recently, however, fluoridation of drinking water has been questioned and many countries have expressed concerns over this practice due to the adverse health effects of fluoride.

Dental fluorosis, also called "mottled enamel", occurs when the fluoride level in drinking water is marginally above 1.0 mg/l. A relationship between fluoride concentration in potable water and mottled enamel was first established in 1931. Typical manifestations of dental fluorosis are loss of shining and development of horizontal yellow streaks on

teeth. Since this is caused by high fluoride in or adjacent to developing enamel, dental fluorosis develops in children born and brought up in endemic areas of fluorosis. Once formed, the changes in the enamel are permanent. When the above manifestations are seen in an adult, they clearly indicate that the person has been exposed to high fluoride levels during her or his childhood.

Skeletal fluorosis affects both adults and children and is generally manifested after consumption of water with fluoride levels exceeding 3 mg/l. Typical symptoms of skeletal fluorosis are pain in the joints and backbone. In severe cases this can result in crippling the patient. Recent studies have shown that excess intake of fluoride can also have certain non-skeletal health impacts such as gastro-intestinal problems, allergies, anaemia and urinary tract problems. Nutritional deficiencies can enhance the undesirable effects of fluoride.

22.3. Guidelines and standards

Taking health effects into consideration, the World Health Organization (1996) has set a guideline value of 1.5 mg/l as the maximum permissible level of fluoride in drinking waters. However, it is important to consider climatic conditions, volume of water intake, diet and other factors in setting national standards for fluoride. As the fluoride intake determines health effects, standards are bound to be different for countries with temperate climates and for tropical countries, where significantly more water is consumed. Although water is generally the major route of fluoride intake, exposure from diet and air may become important in some situations. However, in many cases, the required data on different routes of exposure may be lacking. Data obtained by monitoring fluoride levels in local water supplies and the incidence of fluorosis in the local population can be used to arrive at the appropriate standards.

22.4. Fluoride control options

Search for alternative sources

If fluoride concentration in a community's water supply is significantly and consistently beyond the permissible level, it is essential to consider remedial measures to combat fluorosis. The first choice should be to search for water with a lower fluoride level.

Options are:

(a) Provision of a new and alternate source of water with acceptable fluoride levels

It may be possible to get a safe water source in the vicinity by drilling a new well and/or drawing the water from different depths, as leaching of fluoride into groundwater is a localised phenomenon. Periodic monitoring is needed though, as mixing of water from different aquifers with different fluoride concentrations can occur.

(b) Transporting water from a distant source

This may lead to lasting benefits, but initial cost will be high. Such an approach has been implemented in endemic fluorosis areas in few countries.

(c) Blending high fluoride with low fluoride water

Mixing high and low fluoride waters so as to bring the concentration within permissible levels can be an appropriate long-term solution provided the low fluoride source is available within reasonable distance and is of acceptable quality with respect to other characteristics. This has been successfully implemented in the USA. Recently it has been tried in some parts of India.

(d) Dual water sources

If there are sources with both high and low fluoride levels available to the same community, the source having low fluoride levels can be strictly limited to drinking and cooking. The water source with high fluoride can then be used for other purposes. However, implementation of such dual supply systems requires extensive community awareness programmes. The use of different sources for different purposes may also be hampered by socio-economic factors such as greater distances and burdens of water collection and having to share water sources with users from other neighbourhoods. Education alone is often insufficient to change practices, especially since the impact comes only after prolonged use of high fluoride water sources. Cost of a piped water supply system will be almost doubled when water with low and high fluoride content is supplied through parallel systems. Low-fluoride water could also be sold in containers via commercial outlets.

(e) Rainwater harvesting

There are two ways in which rainwater harvesting (see chapter 7) can be used as a solution for the fluoride problem. Individual household-roof rainwater harvesting and container storage can provide potable water for families. Or harvested surface water run-off can be used to recharge high-fluoride groundwater sources.

Presenting these options with their implications, advantages and disadvantages to community leaders and male and female heads of households will help these groups to make informed choices on the most appropriate solutions in their particular situation.

Defluoridation of water

When none of the above options is feasible or if the only solution would take a long time for planning and implementation, defluoridation of drinking water has to be practised. Two options are then available: (i) the central treatment of water at the source and (ii) the treatment of water at the point of use that is, at the household level. In developed countries treatment at the source is the method adopted. Defluoridation is carried out on

a large scale under the supervision of skilled personnel, usually at a treatment works alongside other treatment processes. Cost is not then a limiting factor. The same approach may not be feasible in less developed countries, especially in rural areas, where settlements are scattered. Treatment may only be possible at a decentralised level, i.e. at the community, village or household level. Treatment at the point of use has several advantages over treatment at community level. Costs are lower, as defluoridation can be restricted to the demand for cooking and drinking – usually less than 25% of the total water demand. Chemical treatment of the entire water demand would lead to production of large volumes of sludge, which requires a safe disposal.

Limitations of point of use treatment are that reliability of the treatment units has to be assured, and that all users should be motivated to use only the treated water for drinking and cooking when untreated water is also available in the house.

Defluoridation methods can be broadly divided into three categories according to the main removal mechanism:

- Chemical additive methods
- Contact precipitation
- Adsorption/ion exchange methods

Chemical additive methods

These methods involve the addition of soluble chemicals to the water. Fluoride is removed either by precipitation, co-precipitation, or adsorption onto the formed precipitate. Chemicals include lime used alone or with magnesium or aluminium salts along with coagulant aids. Treatment with lime and magnesium makes the water unsuitable for drinking because of the high pH after treatment. The use of alum and a small amount of lime has been extensively studied for defluoridation of drinking water. The method is popularly known as the *Nalgonda technique* (RENDWM, 1993), named after the town in India where it was first used at water works level. It involves adding lime (5% of alum), bleaching powder (optional) and alum ($\text{Al}_2(\text{SO}_4)_3 \cdot 18\text{H}_2\text{O}$) in sequence to the water, followed by coagulation, sedimentation and filtration. A much larger dose of alum is required for fluoride removal (150 mg/mg F⁻), compared with the doses used in routine water treatment.

As hydrolysis of alum to aluminium hydroxide releases H⁺ ions, lime is added to maintain the neutral pH in the treated water. Excess lime is used to hasten sludge settling. The dosage of alum and lime to be added to raw waters with different initial fluoride concentrations and alkalinity levels is given in table 22.1.

Table 22.1 Approximate volume of 10% alum solution (ml) to be added in 40 litres of test water to obtain the acceptable limit (1.0 mg F/l) of fluoride at various alkalinity and fluoride levels. The lime to be added is 5% of the alum amount (mg/l)

Test water fluorides (mg/l)	Test water alkalinity as mg CaCO ₃ /l							
	125	200	300	400	500	600	800	1000
2	60	90	110	125	140	160	190	210
3	90	120	140	160	205	210	235	310
4		60	165	190	225	240	275	375
5			205	240	275	290	355	405
6			245	285	315	375	425	485
8					395	450	520	570
10							605	675

(Adapted from RGNDWM, 1993)

The Nalgonda technique has been successfully used at both individual and community levels in India and other developing countries like China and Tanzania. Domestic defluoridation units are designed for the treatment of 40 litres of water (Fig. 22.1) whereas the fill-and-draw defluoridation plant (Fig. 22.2) can be used for small communities. Alum treatment is seldom used for defluoridation of drinking water in developed countries.

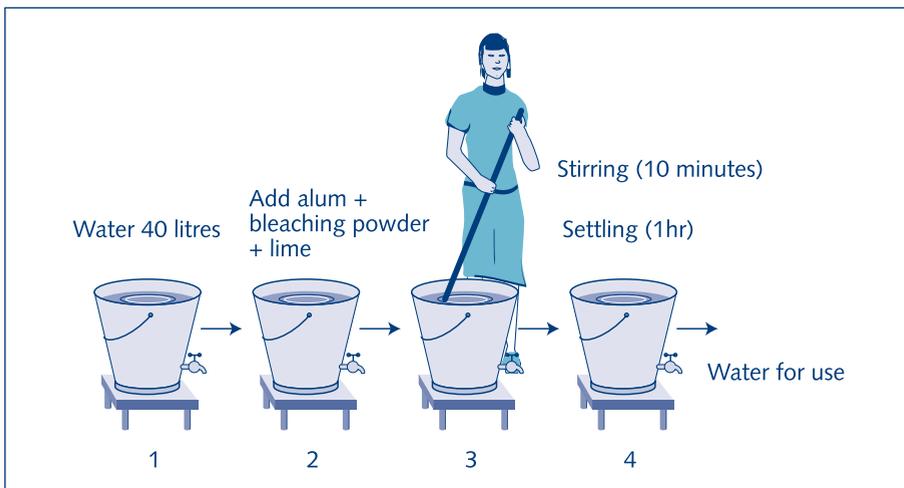


Figure 22.1. Home-based defluoridation using Nalgonda technology (Adapted from RDNDWM, 1993)

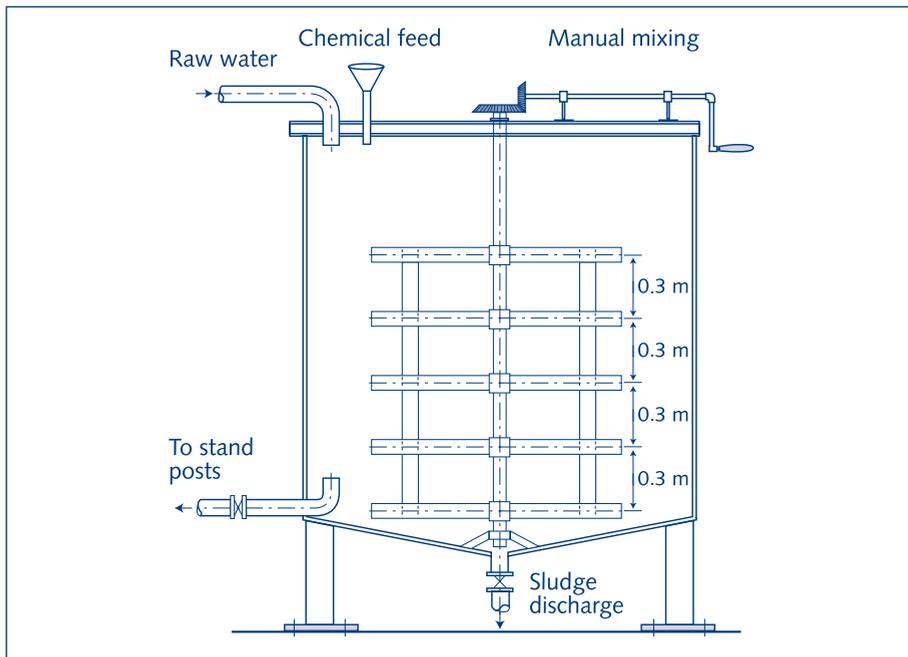


Figure 22.2. Fill-and-draw defluoridation plant for small community
(Adapted from RDNDWM, 1993)

Contact Precipitation

Contact precipitation is a recently reported technique in which fluoride is removed from water through the addition of calcium and phosphate compounds. The presence of a saturated bone charcoal medium acts as a catalyst for the precipitation of fluoride either as CaF_2 , and/or fluorapatite (Fig. 22.3). Tests at community level in Tanzania have shown promising results of high efficiency. Reliability, good water quality and low cost are reported advantages of this method (Chilton, et al., 1999).

Adsorption/ion-exchange method

In the adsorption method, raw water is passed through a bed containing defluoridating material. The material retains fluoride either by physical, chemical or ion exchange mechanisms. The adsorbent gets saturated after a period of operation and requires regeneration.

A wide range of materials has been tried for fluoride uptake. Bauxite, magnetite, kaolinite, serpentine, various types of clays and red mud are some of the naturally occurring materials studied. The general mechanism of fluoride uptake by these materials is the exchange of metal lattice hydroxyl or other anionic groups with fluoride. Fluoride uptake capacity can be increased by certain pre-treatments like acid washing, calcinations, etc. None of the mentioned materials generally exhibits high fluoride uptake capacities.

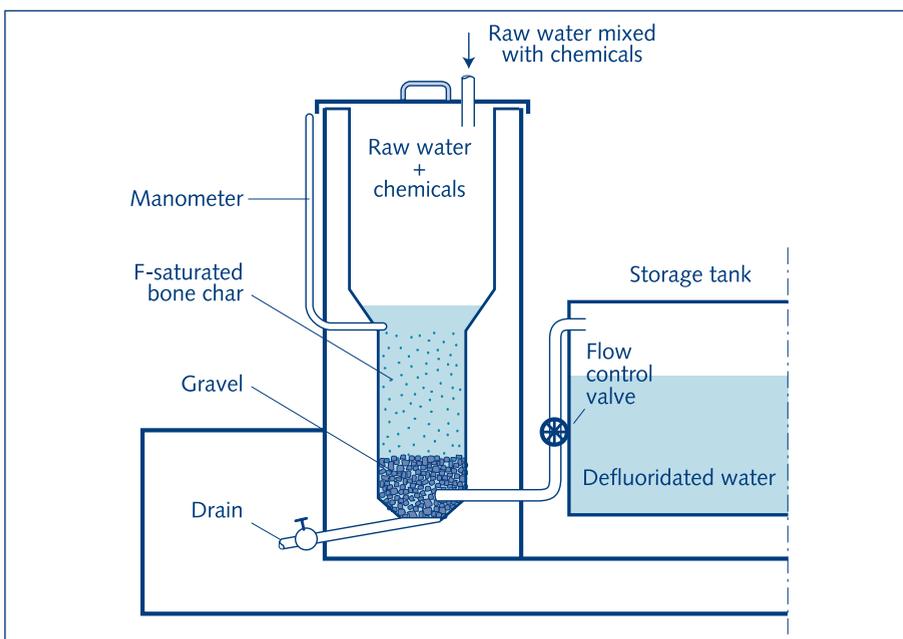


Fig. 22.3. Contact precipitation for fluoride removal (design used in Tanzania)
Source: Bailey K, et al., 1999

Processed materials like activated alumina, activated carbon, bone char, defluoron-2 (sulphonated coal) and synthetic materials like ion exchange resins have been extensively evaluated for defluoridation of drinking water. Among these materials, bone char, activated alumina and calcined clays have been successfully used in the field; (Cummins, 1985, Susanna Rajchagool and Chaiyan Rajchagool, 1997; and Priyanta and Padamasiri, 1996). Application of these materials is described below.

Bone char as a defluoridating material. Bone char consists of ground animal bones that have been charred to remove all organic matter. Major components of bone charcoal are calcium phosphate, calcium carbonate and activated carbon. The fluoride removal mechanism involves the replacement of carbonate of bone char by fluoride ion. The method of preparation of bone charcoal is crucial for its fluoride uptake capacity and the treated water quality. Poor quality bone char can impart bad taste and odour to water. Exhausted bone char is regenerated using caustic soda. Since acid dissolves bone char, extreme care has to be taken for neutralising caustic soda. Presence of arsenic in water interferes with fluoride removal.

In the USA in the past, a few defluoridation plants were using bone char. Now they have been largely replaced by activated alumina. Bone char is considered as an appropriate defluoridating material in some developing countries. The ICOH¹ domestic

1 ICOH = Inter-Country Centre for Oral Health at the University of Chiang Mai, Thailand

defluoridator was developed in Thailand and uses crushed charcoal and bone char (Fig. 22.4). Its defluoridation efficiency depends on the fluoride concentration in raw water as well as the fluoride uptake capacity and the amount of bone char used in the filter. Field trials in Thailand, Sri Lanka and some African countries have shown very encouraging results (Priyanta and Padamasiri, 1996; Mjengera et al., 1997; and Susanna Rajchagool and Chaiyan Rajchagool, 1997). Reports from Sri Lanka have shown that with 300 gm charcoal (mainly to remove colour and odour) and 1 kg bone char an ICOH filter can defluoridate on an average 450 litres of water containing 5 mg/l F⁻ at a flow rate of 4 litres per hour. Regeneration of spent bone char is not recommended for these household units. Instead it should be replaced with fresh material commercially available in local shops.

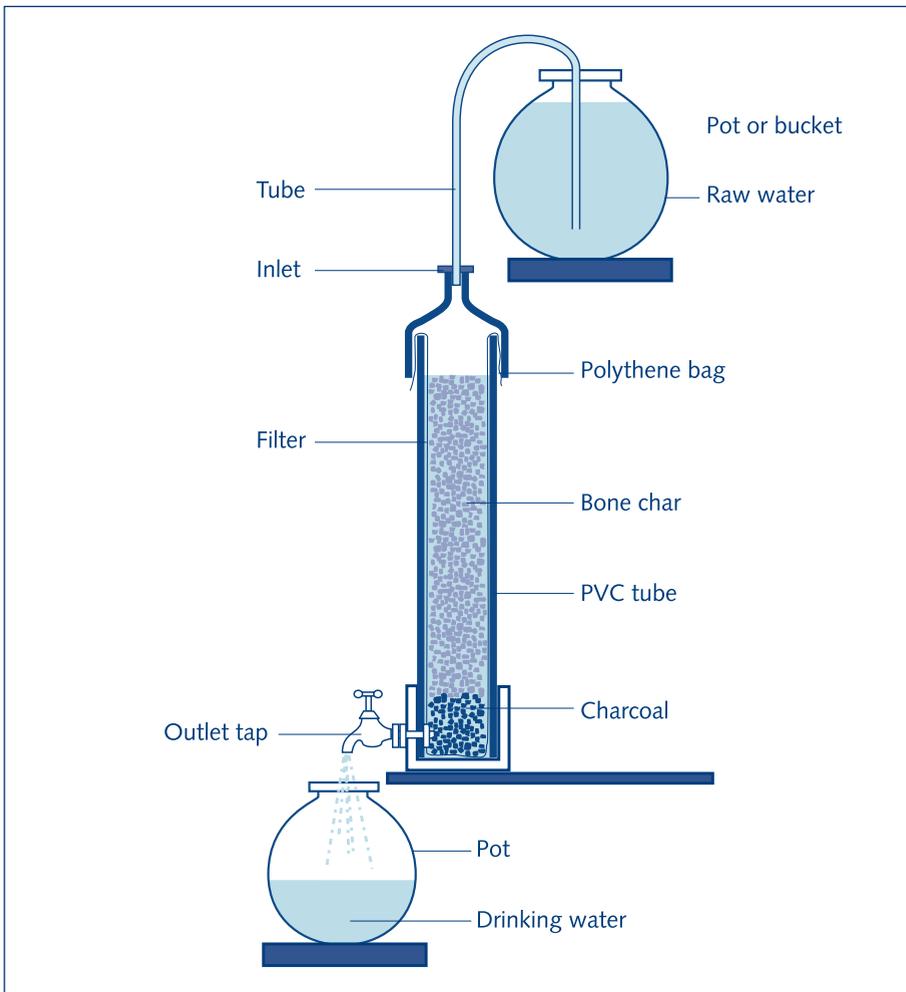


Figure 22.4. Bone char domestic defluoridator developed by ICOH-Thailand

Activated alumina as a defluoridating material. Activated alumina or calcined alumina, is aluminium oxide, Al_2O_3 . It is prepared by low temperature dehydration (300-600°C) of aluminium hydroxides. Activated alumina has been used for defluoridation of drinking water since 1934, just after excess fluoride in water was identified as the cause of fluorosis.

The fluoride uptake capacity of activated alumina depends on the specific grade of activated alumina, the particle size and the water chemistry (pH, alkalinity and fluoride concentrations). In large community plants the pH of the raw water is brought down to 5.5 before defluoridation, as this pH has been found to be optimum and it eliminates bicarbonate interference. The mechanism of fluoride removal is most probably the ligand exchange reaction at the surface of activated alumina. Exhausted activated alumina has to be regenerated using caustic soda. To restore the fluoride removal capacity, basic alumina is acidified by bringing it into contact with an excess of dilute acid (Clifford, 1990).

Activated alumina has been the method of choice for defluoridation of drinking water in developed countries. Generally it is implemented on a large scale in point of source community plants. A few point of use defluoridation units have been developed which can be directly attached to the tap. During recent years this technology is gaining wide attention even in developing countries. Domestic defluoridation units (Fig. 22.5) have been developed in India using indigenously manufactured activated alumina, which is commercially available in bulk quantities. Choosing the proper grade of activated alumina is important for its effective reuse in multiple defluoridation cycles. Around 500-1500 litres of safe water could be produced with 3 kg of activated alumina when the raw water fluoride is 11 and 4 mg/l respectively at natural water pH of 7.8-8.2. The frequency of regeneration is once in 1.5-3 months. The cost of activated alumina is around US\$ 2 per kg and the total cost of the domestic filter depends upon material used for filter assembly. Regeneration of exhausted activated alumina costs around US\$ 0.5 (Venkobachar et al., 1997).

Major requirements are the creation of demand for treatment and the setting up of good supply and financing systems and arrangements for regeneration. Sale of the ingredients and the provision of education and monitoring through visits to user households has in some places become a source of income for trained women promoters. The units are being evaluated in several villages in India. Daily operational care for using these units is normally negligible. However, the exhausted activated alumina has to be regenerated once every few months. This is carried out at the village level.

Calcined clay. Freshly fired brick pieces are used in Sri Lanka for the removal of fluoride in domestic defluoridation units (Fig. 22.6). The brick bed in the unit is layered on the top with charred coconut shells and pebbles. Water is passed through the unit in an upflow mode. The performance of domestic units has been evaluated in rural areas of

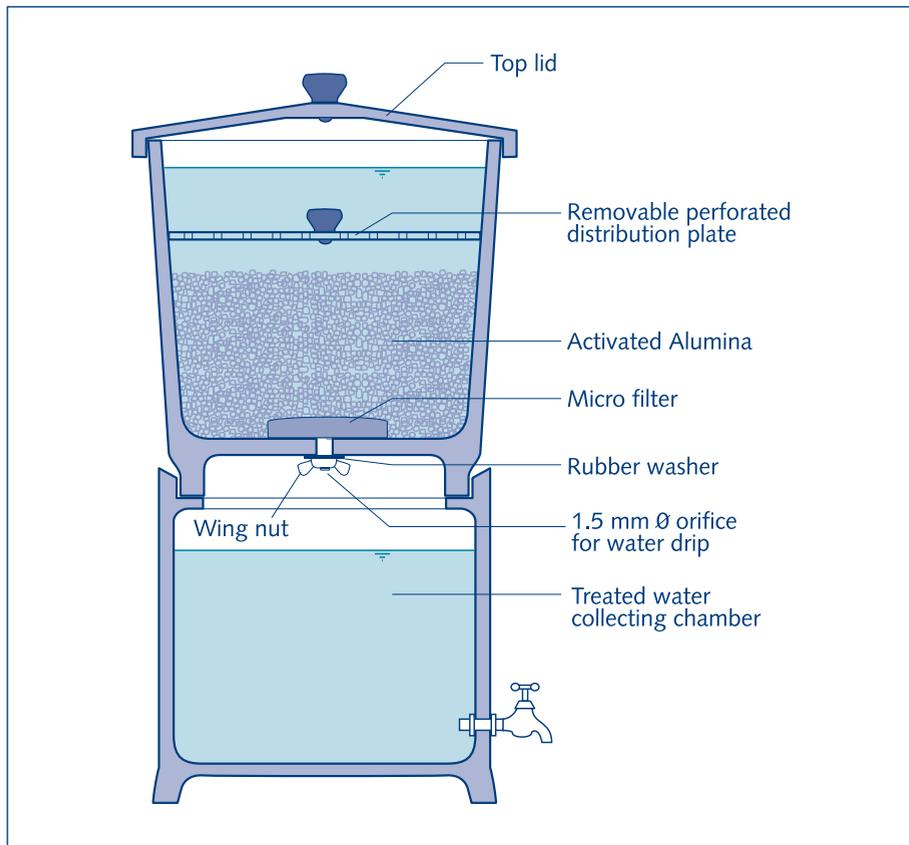


Fig. 22.5. Activated alumina-based domestic defluoridation filter

Sri Lanka (Priyanta & Padamsiri 1997). It is reported that efficiency depends on the quality of the freshly burnt bricks. The unit could be used for 25-40 days, when withdrawal of defluoridated water per day was around 8 litres and raw water fluoride concentration was 5 mg/l. As PVC pipes are costly, a defluoridator made out of cement and bricks has also been recommended. Apart from the methods discussed above, specific synthetic ion exchangers and separation technologies such as reverse osmosis and electro dialysis have also been developed for fluoride removal from potable water.

To select a suitable defluoridation method for application in developing countries, some of the following criteria need to be considered:

- Fluoride removal capacity
- Simple design
- Easy availability of required materials and chemicals
- Acceptability of the method by users with respect to taste and cost

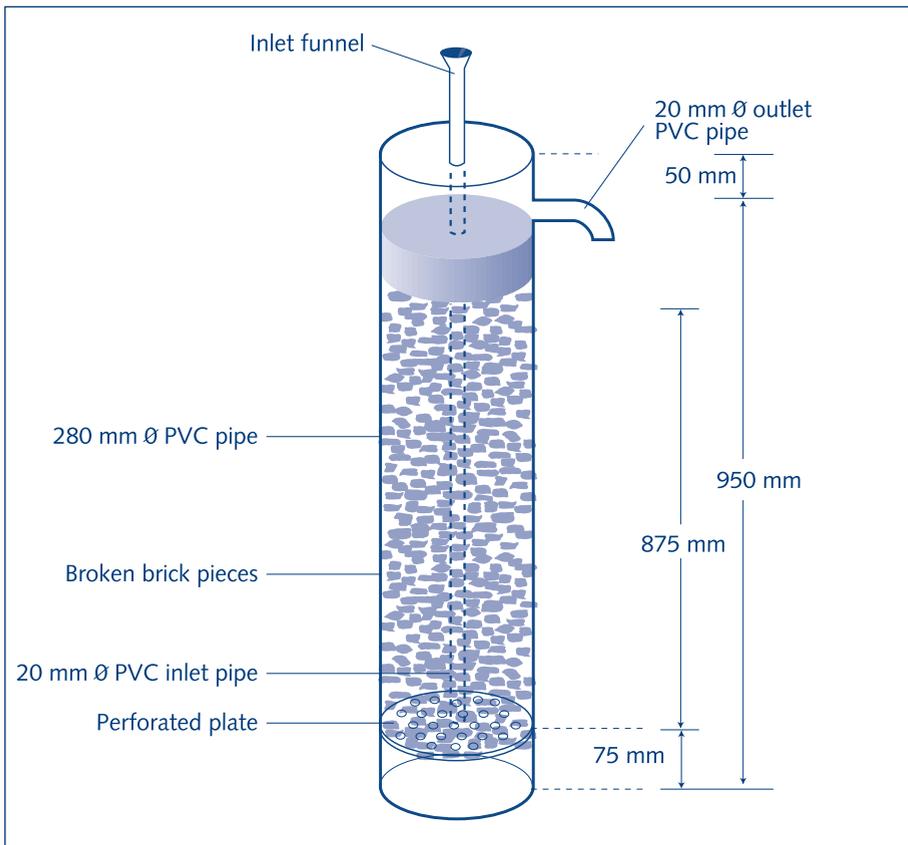


Fig. 22.6. Domestic defluoridation unit using brick pieces
Source: Priyanta and Padamasiri, 1996

Both precipitation and adsorption methods have advantages and limitations. In the Nalgonda technique easily available chemicals are used and the method is economically attractive. Limitations of the method are varying alum doses depending on fluoride levels in water, daily addition of chemicals and stirring for 10-15 min, which many users may find difficult. In adsorption-based methods like activated alumina and bone char, daily operation is negligible. Activated alumina is costly. Hence exhausted alumina has to be regenerated using caustic soda and acid and repeatedly reused, at least for a few cycles. Improperly prepared bone char imparts taste and odour to the treated water. Since bone char from point of use units is not generally regenerated, a ready supply of properly prepared material needs to be available. Furthermore, bone char may not be culturally acceptable to certain communities as defluoridating material. Some of the merits and demerits of defluoridation methods are given in table 22.2.

Table 22.2 Merits and demerits of some defluoridation methods

Method	Capacity	Working pH	Merits	Demerits	Estimated relative cost
Nalgonda	150 mg alum dose / mg F removal; dose varies with alkalinity	Ambient	Low technology; adaptable at point of use & point of source level	<ul style="list-style-type: none"> • Large quantities of sludge • High chemical dose • Dose depending on F- level • Daily addition of chemicals and stirring in point of source units 	Low-medium
Bone char	Variable design value 1000-2000 mg/kg	Ambient	Locally available media	<ul style="list-style-type: none"> • May impart taste and odour and result in organic leaching if not prepared properly • Requires regeneration periodically • Affected by high alkalinity • May not be acceptable in some countries 	Low-medium
Activated alumina	Variable 1000-1500 mg/kg	5-6 in large defluoridation plants; ambient in point of use units	Effective; much experience	<ul style="list-style-type: none"> • Periodic regeneration • Skilled personnel for plant operation • Properly trained staff for regeneration of point of use units • Suitable grades may not be indigenously available in less developed countries 	Medium-high
Contact precipitation	No information	Ambient	No much experience	<ul style="list-style-type: none"> • Algal growth can occur in phosphate solution. • Bone char used as a catalyst may not be acceptable in many countries 	High-very high
Brick	No information	Ambient	Low-cost technology	<ul style="list-style-type: none"> • May not be universally applicable 	High-very high
Reverse osmosis/ electro dialysis	High	Ambient	Can remove other ions	<ul style="list-style-type: none"> • Skilled operation • Interference by turbidity • High cost 	Very high

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Web sites

Fluorosis: <http://www.education.vsnl.com/fluorosis>

Voice: <http://www.voice.buz.org/fluoridation>

No Fluoride: <http://www.nofluoride.com>

Inter country Centre for Oral Health: <http://www.icoh.org>

UNICEF: <http://www.unicef.org>

23 Technologies for arsenic removal from groundwater

Tanveer Ahsan

23 Technologies for arsenic removal from groundwater

23.1 Occurrence of arsenic in groundwater

Arsenic is the twentieth most abundant element in the earth's crust. It is normally found dissolved in surface and ground water in the order of magnitude of micrograms per litre ($\mu\text{g/L}$). It is a highly toxic element, but when dissolved in water it is colourless, odourless and tasteless and cannot be easily detected. Arsenic concentration above the drinking water supply standard has been found in groundwater from large parts of Argentina, Chile, Taiwan, Inner Mongolia, Mexico and the western USA. However, the worst situations are in Bangladesh and in the State of West Bengal in India. In Bangladesh, out of approximately 8 million tubewells, about 3 million are contaminated with arsenic above the acceptable limit, and 20 million people are potentially at risk. WHO has described this crisis as the largest mass poisoning in the world. In West Bengal another 5 million people are at risk.

In 1993 WHO lowered its guideline value for arsenic in drinking water from 50 to 10 $\mu\text{g/L}$. Several countries have also lowered their standard to match. However, many developing countries, including Bangladesh and India, still have 50 $\mu\text{g/L}$ as their standard.

Until recently arsenic was not an element for routine measurement in groundwater supply or quality assessment programmes, particularly in developing countries. However, the recent discovery of widespread arsenic contamination in Bangladesh and India has made it necessary to investigate this parameter in other hydro-geological environments. Mining-related arsenic contamination is often locally confined and it does not tend to disperse far beyond the mining areas because of its sorption onto mineral surfaces, particularly ferric hydroxide. The wide contamination of aquifers outside mining areas is due to natural geo-chemical processes over thousands of years. The exact mechanism of arsenic release and its mobility into the groundwater is not yet known. There are two hypotheses for the developments in Bangladesh and West Bengal:

- Arsenic occurs in sedimentary pyrite and arsenopyrite and is released by oxidation as the water table is lowered in response to groundwater pumping.
- Arsenic occurs adsorbed onto iron and manganese oxyhydroxides, which are carried in the suspended load of rivers, and is released by natural reductive processes accompanying burial of the sediments.

Arsenic is harmful only when it is consumed internally, i.e. it is used for drinking and cooking. No adverse effect had been reported for external usage like washing and bathing. Boiling does not remove arsenic from water.

23.2 Arsenic-related health problems

After fluoride, arsenic is the naturally occurring chemical constituent of groundwater that causes the most health-related problems. Arsenic-related diseases occur when arsenic-contaminated water is consumed over a long period of time (years). The severity of illness depends on the contamination level, the period of consumption, and the nutrition level of the person concerned. It also differs from person to person. A clear explanation for this variation is not yet known.

The symptoms of arsenic-related diseases are both visible and invisible (sub-clinical). Both are equally harmful. The main visible symptoms in the first stage of arsenic related disease are the darkening of the skin and the appearance of black spots (melanosis), and the hardening of palms and soles of feet (keratosis). In the second stage, raindrop-like white spots appear on the skin (leuko-melanosis), palms of hands and soles of feet grow hard, growths and cracks appear and they become very painful (hyper-keratosis). Other symptoms are the swelling of the feet and complications in kidney and liver functions. The third stage is damage of internal organs like lungs, kidney, liver, bladder, etc, and in extreme cases gangrene and cancer. As internal damage can occur without showing any visible symptoms, it is difficult to identify all arsenic affected patients.



Fig. 23.1. Visible symptoms of hard growths and cracks in the sole of arsenic affected patients.

23.3 Cure for arsenic-related diseases

Arsenic toxicity has no known effective medicine for treatment. **The only cure for arsenic-related disease is consuming arsenic-free water.** During the early stages of the disease it can be reversed by drinking arsenic-free water, which flushes the toxicity out of the body. But at a later stage the disease becomes irreversible and when the vital organs are damaged the patient often suffers a painful death. As for medical care, sometimes multi-vitamins are recommended to assist the body's immune system and ointments are applied in cracked hands and soles to prevent secondary infection.

23.4 Arsenic testing

Arsenic contamination in water can be tested in laboratories. Field test kits are available commercially and can detect arsenic contamination down to 50 µg/l with an acceptable accuracy. New generation field test kits claim to detect down to 10 µg/l. However, they have not been widely tested as yet. Handpump tubewells in villages can be tested with the field test kits. In Bangladesh, when the arsenic concentration is found to be above the allowable limit, the handpump is painted red, indicating that it is not suitable for drinking and cooking. Otherwise it is painted green, showing that it can be used for all purposes.

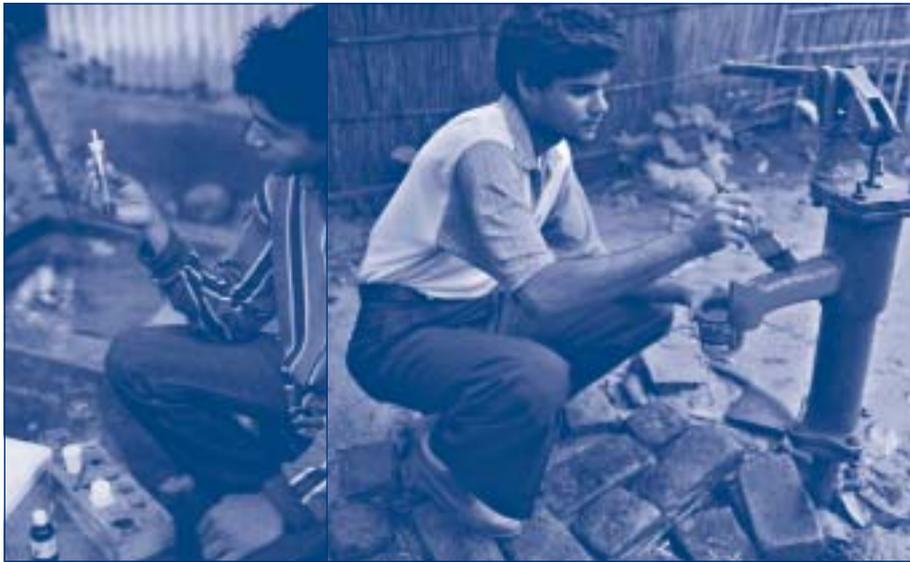


Fig. 23.2. Testing of tubewell water by a field test kit (left) and a contaminated handpump being painted red indicating that it should not to be used for drinking and cooking (right).

If the handpump used by a group of people has been painted red, there must be another water source nearby that gives water with an acceptable arsenic content suitable for drinking and food preparation. That can be a green-painted handpump, a rainwater tank, or any other safe source. That also means that more people will go to the “safe” water sources to get their drinking and cooking water there. Often, it will be necessary to make sharing arrangements with the families that are already using these sources. Some form of agreement has to be reached between new and old user families. This may relate to the maximum number of new users and the exclusive purpose of use of the safe water (for drinking and cooking), because the yield of the source may be limited. But also issues such as the times of collection and the realistic contribution for the O&M cost because of the extra wear and tear, need to be agreed. Here, a facilitator with skills in participatory planning tools such as social maps and time diagrams helps to work out locally feasible arrangements.

23.5 Physical-chemical processes for arsenic removal

The technologies for removal of arsenic are based on one or a combination of several physical-chemical processes. The five common types of treatment are summarised below:

Oxidation: Arsenic in groundwater may occur as arsenite, As(III), and arsenate, As(V), in different proportions. Most arsenic treatment technologies are very effective in removing the pentavalent form of arsenic (arsenate), but the removal efficiency of the trivalent form (arsenite) is very low. Therefore, many treatment technologies include oxidation as a pre-treatment step to convert As(III) to As(V).

Atmospheric oxygen, hypochlorite and permanganate are most commonly used for oxidation process of arsenic in developing countries.



Other chemicals that are used include gaseous chlorine, ozone and other oxidising agents. Natural oxidation of arsenic by air is very slow and can take several weeks, but the above chemicals can oxidise rapidly.

Adsorption and co-precipitation: Conventional water treatment through coagulation is also effective for arsenic removal. For this method a coagulant (e.g. alum or ferric chloride) is added and rapidly mixed for about one minute. Aluminium or ferric hydroxide micro-flocs are formed (coagulation). The water is then gently stirred for a few minutes (flocculation) and the majority of the micro-flocs agglomerate into larger settleable flocs. During the coagulation-flocculation process many micro-particles and negatively charged ions are attached onto the flocs. Arsenic also attaches to the flocs (adsorption). Subsequently sedimentation and filtration (co-precipitation) are used to separate the flocs, together with the adsorbed arsenic.

Commonly used coagulants are alum, $\text{Al}_2(\text{SO}_4)_3 \cdot 18\text{H}_2\text{O}$; ferric chloride, FeCl_3 ; and ferric sulphate, $\text{Fe}_2(\text{SO}_4)_3 \cdot 7\text{H}_2\text{O}$. Ferric salts are comparatively more effective on a weight basis and operate over a wider pH range (6.0 – 8.5). Alum is effective over a narrow range of pH (7.2 – 7.5). In both cases, As(III) is not effectively removed, but As(V) is. Pre-oxidation is highly recommended.

Removal of naturally occurring iron: In many areas, including Bangladesh and West Bengal, arsenic in groundwater is often found together with high levels of iron and manganese. The conventional iron (and manganese) removal method of aeration,

flocculation (optional), sedimentation (optional) and filtration can also significantly remove arsenic. This process is basically similar to the adsorption and co-precipitation process except that no coagulant is added. During aeration and flocculation, iron hydroxide flocs are created. Arsenic is attached onto these flocs, and separated by sedimentation and filtration. Some remaining arsenic in the solution can also be removed by adsorbing onto the iron coating formed over the filter sand grains.

The efficiency of this process is low if the water contains a low amount of iron or if the proportion of As(III) is low. In these cases, the removal efficiency can be increased by addition of coagulants (e.g. alum or ferric chloride) and oxidants (e.g. chlorine) respectively.

Sorption on filter media: When arsenic-contaminated water is passed through a sorption media bed, its media can remove arsenic. The commonly used media are activated alumina, activated carbon, iron and manganese coated sand, activated carbon, kaolinite clay, and hydrated ferric oxide. The efficiency and the total amount of water treated depend on the media and the water composition, as different contaminants and components of water compete for the available sites on the media. In most of the cases arsenic removal is very effective if oxidation is carried out before sorption.

Ion exchange: Synthetic ion exchange resins are a special kind of sorption media. They are used in water treatment to remove undesirable ions by replacing ions attached to the resins. With the continuing use of the resins their removal capacity is exhausted and they need regeneration.

The arsenic exchange equation can be represented as follows where **R** represents ion exchange resins and Cl⁻ is an ion attached to the resin.



The regeneration equation using common salt as regeneration agent is



Ion exchange is very effective when the form of arsenic in water is predominantly As(V). Otherwise a pre-oxidation step is necessary. Various anion exchange resins that can remove arsenic (arsenate) concentration below 1 µg/L are commercially available. Conventional sulphate-selective and nitrate-selective resins are suitable for arsenic removal. The arsenic removal capacity depends on the sulphate and nitrate contents of the raw water as they are exchanged prior to arsenic.

23.6 Technologies for arsenic removal

For large central treatment plants several technologies are available for removing arsenic. These technologies are based on one or more of the processes already described. Many of the technologies can also be scaled down to a smaller version for small towns and rural areas. The large-scale contamination of groundwater with arsenic was detected only in recent years. Several small version technologies have been installed on a pilot basis and others are emerging. So far experience with these technologies is low. Smaller version technologies can be classified according to their production capacity and number of users. Some promising technologies are discussed below. A summary of the processes and technologies is presented in table 23.1.

23.7 Community-scale treatment plants

Community-scale treatment plants are designed for small urban settlements or villages. The plants are operated in a continuous-flow mode and treated water is distributed through small-diameter pipelines.

Conventional plants: the treatment methods used in a conventional water treatment plant, i.e. application of coagulants (e.g. alum, ferric chloride), coagulation-flocculation, sedimentation and finally filtration, are also effective in arsenic removal. A small capacity conventional plant may be used for community level treatment. Normally groundwater is first aerated and other processes then follow. If groundwater contains a higher proportion of As(III), oxidation by air alone is not sufficient to oxidize As(III) to As(V) for effective arsenic removal. In that case after aeration an additional chemical oxidation step (e.g. chlorine) is recommended.

Iron removal plants: When groundwater contains an excessive amount of iron (and/or manganese), plants designed for iron removal treatment can at the same time remove arsenic. Depending on the proportion of As(III) in the water, a chemical oxidation step may be required. Many iron removal plants do not use flocculation and sedimentation; the flocs formed as a result of the presence of natural coagulants or added coagulants are removed in the direct filtration step. Figure 23.3 shows a direct filtration process of iron and arsenic removal.

Other methods that can be used in small communities are arsenic-specific filter units (like activated alumina filters) and ion exchange resin beds. These units may need a coarse sand pre-filter unit to remove excessive iron so that iron hydroxide flocs formed do not clog the subsequent filter units. Again a chemical oxidation step may be necessary depending on the proportion of As(III).

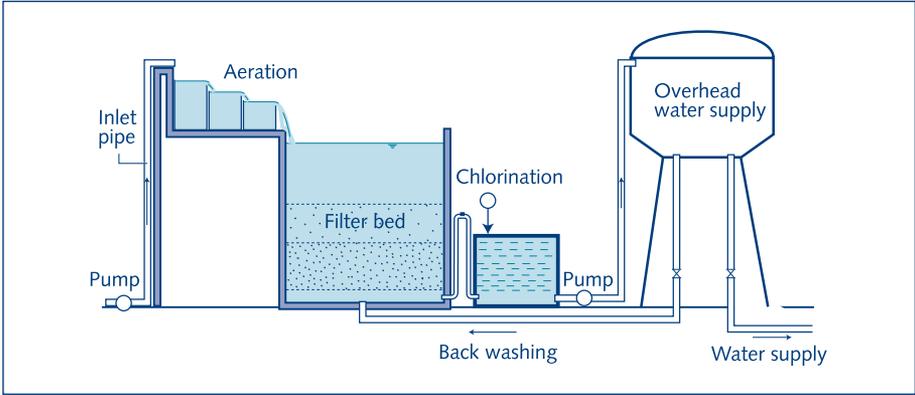


Fig. 23.3. Schematic diagram of iron and arsenic removal plant in Bangladesh

23.8 Arsenic removal units attached to tubewells

Handpump tubewells are widely used for rural water supply in many developing countries. However, many of the tubewells in Bangladesh and West Bengal, which were believed to be safe for drinking, are now found to be arsenic contaminated. Attempts have been made to develop treatment units attached to these tubewells. The units normally operate in intermittent flow mode and the treated water is carried manually in containers (such as pitchers, buckets and cans). Tubewell-attached units can be used by a number of families in the neighbourhood. Three types of units are described below.

Compact conventional treatment unit: In villages of West Bengal, India, compact conventional treatment units for arsenic removal are attached to contaminated handpump tubewells. Figure 23.4 shows a typical unit. In this case the treatment processes include chemical oxidation by sodium hypochloride, addition of alum as coagulant, mixing, coagulation-flocculation, sedimentation and upflow filtration.

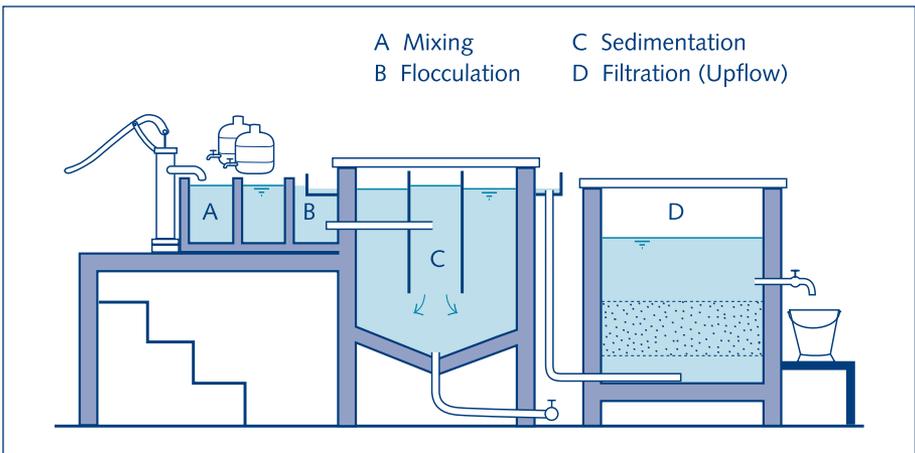


Fig. 23.4. A compact conventional treatment unit attached to a tubewell

Iron removal unit: This is a small version of an iron removal plant and is attached to a handpump tubewell. It is used where groundwater contains an excessive amount of iron. Figure 23.5 shows a typical design of a unit with a cube with side dimensions of about 1 m. The treatment method consists of aeration, sedimentation, flocculation and sedimentation in a roughing filter and final filtration. The water from the handpump tubewell is passed through a slotted horizontal PVC pipe and falls into the cubical structure. The structure consists of three chambers. Water from the slotted pipe falls into the first chamber and aeration occurs. Here partial sedimentation also takes place. Water from the first chamber then enters the bottom of the second chamber, which is an upflow roughing filter with coarse aggregates (20-30 mm). Here, flocculation-sedimentation occurs and iron hydroxide micro-flocs grow in size with most of them settling onto the coarse aggregates. The partially filtered water then overflows into the third chamber, consisting of a bed of coarse sand or small aggregates, and a final downflow filtration takes place. The filtered water is delivered through an underlying compartment and pipes. During the iron removal process arsenic is removed by adsorption and co-precipitation.

The iron removal units require regular washing to maintain them in a proper working condition. Where the iron concentration is over 10 mg/l, partial cleaning is required every seven to ten days. Partial cleaning is done by scraping the top layer of the smaller grain filter media in the third chamber, opening all washouts and pouring two to three buckets of water over each chamber. The scraped filter media is cleaned and replaced in position. Complete washing is required once a month. This is a laborious process that takes at least one person-day. It involves taking out all the filter materials, thoroughly washing and replacing them.

Sorption media filters: A filter unit consisting of sorption media can also be attached to a handpump tubewell. If the water contains a high amount of iron, a pre-filter unit of coarse sand is often required. The pre-filter unit must be washed periodically. Units with activated alumina on the sorption media are installed in many places in Bangladesh and West Bengal on a pilot basis. These units have a coarse sand pre-filter unit. The arsenic removal efficiency of activated alumina is high (> 95 %) and both arsenite and arsenate can be removed. Activated alumina can be used for months before breakthrough and then it needs to be replaced or regenerated.

Other sorption media like filters with ion exchange resins can also be attached to tubewells. If needed the water can be oxidised by adding potassium permanganate in a pre-treatment step.

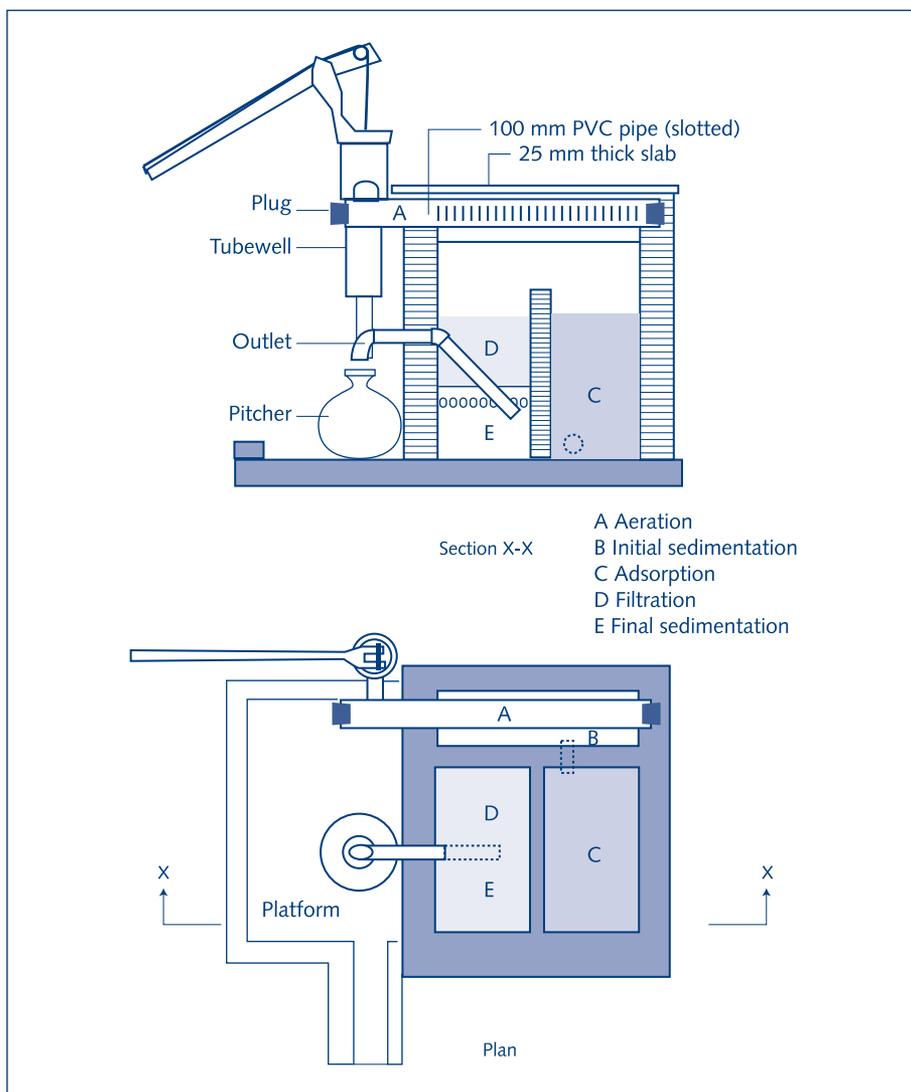


Fig. 23.5. A tubewell attached iron and arsenic removal unit

23.9 Household treatment units

Household units are basically meant to provide safe drinking and cooking water for a family. About 5 litres of water per capita per day is required. Several household treatment units are currently being proposed and others are under development. Normally, water from an arsenic affected source tubewell is collected and manually poured into the units. These units operate in batch flow mode. So far there is not much experience with the long-term effectiveness of household treatment units. Some promising ones are described below.

Two-bucket treatment unit: This method is based on the co-precipitation process. The unit consists of two buckets, each with a capacity of about 20 litres. Normally they are placed one above the other. Arsenic-contaminated water is poured in the top bucket. Chemicals are then added and vigorously stirred with a stick for about one to two minutes. Thereafter the water is gently stirred for flocculation for another two to three minutes. The mixed water is allowed to settle for about two hours. The chemicals added are a mixture of coagulant (e.g. alum, ferric chloride or sulphate) and oxidant (potassium permanganate, calcium hypochloride) in crushed powder form. They are normally supplied in single dose packages.

After settling, the top two thirds of the supernatant water from the first bucket is poured gently (or flows by plastic pipe) into the second bucket. The second bucket is half filled with filter sand and has a perforated underdrain pipe connected to an external tap. When the tap is opened the settled water passes through the filter sand, which removes the remaining micro-flocs. A schematic diagram of the two-bucket system is shown in figure 23.6.

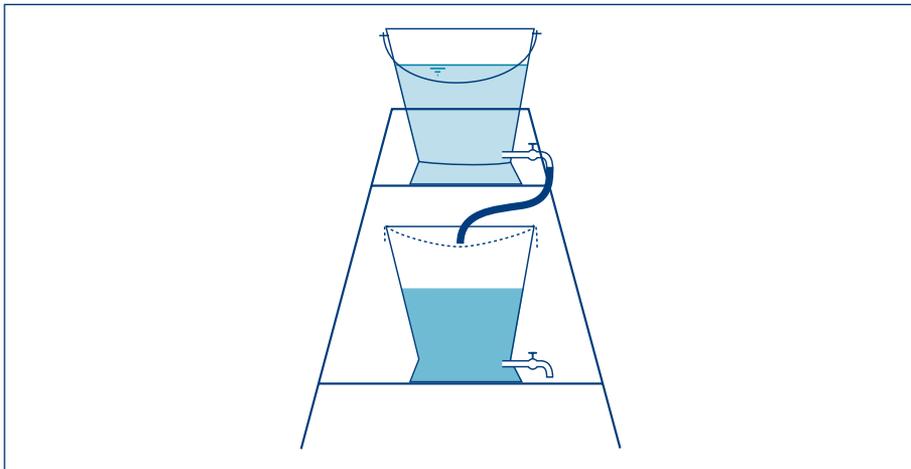


Fig. 23.6. Schematic diagram of a typical two-bucket household unit

The filter sand in the second bucket needs to be manually cleaned once or twice a week. Usually the women heads of households operate the filter. They need training in how to operate the system and to prevent bacteriological contamination of the water. Having women trainers facilitates communication and recognises women's water management roles.

Normally the two-bucket units can lower the arsenic concentration of effluent water to below the developing country standard of 50 µg/L. The performance varies with operation and maintenance skills and with the water quality parameters, especially pH. The use of ferric salts has been found to be most effective in arsenic removal.

Three-pitcher method: The three-pitcher filter consists of three 20-liter clay pitchers stacked in a frame. A schematic diagram of a typical three-pitcher filter is shown in figure 23.7. The top first pitcher contains 2 kg of coarse sand covered with 3 kg of iron filings. The second pitcher contains 2 kg of coarse sand with 1 kg of charcoal above it. The third pitcher collects the filtered water.

The three-pitcher filter has been found to be very effective in arsenic removal for the first four to six weeks. After that, the removal efficiency starts to decline. The flow rate of effluent is slow, about 1-2 litres per hour. Normally the three-pitcher filter needs to be replaced after three to four months as the efficiency declines and the iron filings in the first filter become clogged and hardened and cannot be removed. Although this method is effective in arsenic removal, the bacteriological contamination in effluent water is sometimes high as the open filter media harbour growth of micro-organisms.

Several household filter units have been developed that use different kinds of filter media. Some of the promising ones are iron coated sand filters, iron coated granular activated carbon filters, activated alumina and resin filters (see references). Research is also under way on a technology using solar energy – SORAS¹.

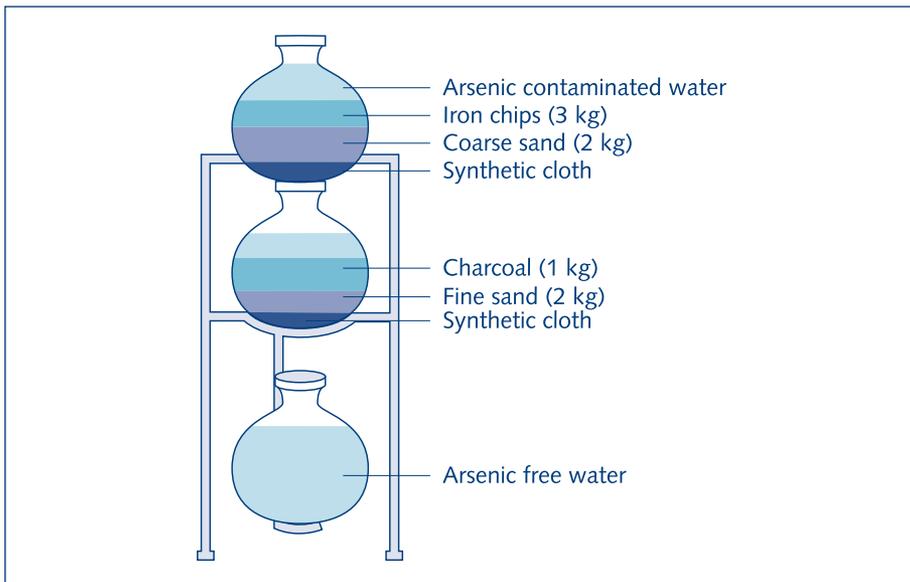


Fig. 23.7. Construction details of a three-pitcher filter

1 For SORAS (Solar Oxidation and Removal of Arsenic) see website:
<http://www.eawag.ch/research/arsen/d-index.htm>

23.10 Uncertainties and considerations for arsenic removal technology

There are several uncertainties and considerations related to the arsenic problem in drinking water supply. Often there is no systematic pattern in spatial distribution of arsenic contamination. In a village one tubewell may have arsenic concentration above the allowable limit whereas the neighbouring one may have no contamination. Therefore, all the existing tubewells need to be tested for their arsenic contamination level. Although research and investigation is going on, the exact mechanism of arsenic release into the groundwater and its mobility remain unclear, so there is an uncertainty in further spreading of arsenic.

Several technical and social uncertainties prevail in selecting an appropriate technology. Many technologies are being piloted and have promising results. They need to be properly tested over time and validated before large-scale replication. These technologies have to be socially acceptable and user friendly. And the filter materials (such as sand and iron chips) and chemicals packed for household use need to be available and affordable, otherwise their sustained operation and use will remain questionable.

The effect of arsenic on human health and the acceptable standard for drinking water are being debated globally and in specific countries. Several developing countries still have the standard as 50 µg/L but in the near future it may be reduced to the WHO recommended level of 10 µg/L. In that case many of the presently suggested technologies may become obsolete or will require modification.

The safe disposal of arsenic sludge is an environmental issue. The standards and methods are yet to be determined.

Finding an acceptable water source is an important issue. There are two main options: find a new arsenic-free water source or use the existing arsenic-contaminated water source and treat that water. In either case, the supplied drinking water has to be free from arsenic contamination, i.e. below the acceptable limit as well as free from bacteriological and other chemical contamination.

The new source may be surface water but that water needs to be treated. Rainwater harvesting is feasible in some areas. The new water source may also be groundwater that is arsenic safe and located in a different aquifer (e.g. a deep aquifer that appears to have water with no or low arsenic levels) or come from a suitable aquifer in a nearby location. In some areas dug wells with depths up to 10 metres are found to be free of arsenic contamination. Locating an arsenic-safe aquifer that will remain arsenic free in the long run requires substantial hydrogeological investigations.

When designing the production capacity of a treatment unit a decision has to be made about whether the amount of treated water is to be enough for all purposes or only for drinking and cooking. The plant type and the number of people using a unit have to be determined, i.e. will it be a community, a neighbourhood or a household level technology? In many instances, a central village level treatment unit and piped distribution system is preferred. This is because people like to have a higher, or at least equal, level of service compared with the existing contaminated one. Other advantages are that water quality monitoring can be done in one place instead of several and that the per capita costs can be comparable to other options. If a better technology emerges, or if water quality deteriorates, or the water quality standard is made more stringent in future, then the technology replacement or modifications can be conveniently done in one place. The private sector can be involved through the treatment and sale of the treated water in larger containers.

Hence, all the considerations, including possible future developments, have to be carefully weighed and discussed with the users; only then can they make a well-informed and balanced decision on the most suitable technology for their specific conditions. On the other hand, immediate supply of arsenic-free water is needed in arsenic-affected areas to prevent further spreading of arsenic-related diseases. Governments may make a decision to provide arsenic-safe water immediately by applying a proven technology. In parallel, field tests need to be carried out in search for the best long-term sustainable technologies.

Table 23.1 Summary of processes for arsenic removal

Physical-chemical processes	Removal efficiency		Experience with different scale of operation and remarks
	As (III)	As (V)	
Oxidation	n.a.	n.a.	This is not a removal process, but converts As(III) to As(V) for subsequent efficient removal. Oxidation by air is very slow and as such chemical oxidation is preferred. Chemical oxidation is a proven technology and requires moderate skill. Moderately expensive.
Adsorption and co-precipitation - using iron salts	++	+++	Well proven at central level, piloted at community and household levels. Phosphate and silicate may reduce arsenic removal rates. Generates arsenic-rich sludge. Relatively inexpensive.
Adsorption and co-precipitation - using alum	-	+++	Proven at central level, piloted at household levels. Phosphate and silicate may reduce arsenic removal rates. Optimal over a relatively narrow pH range. Generates arsenic-rich sludge. Relatively inexpensive.

Physical-chemical processes	Removal efficiency		Experience with different scale of operation and remarks
	As (III)	As (V)	
Naturally occurring Fe removal	?	+/ ++/ +++	Little application in central systems, limited studies at community and household levels. More research is needed on which hydrochemical conditions are conducive for good arsenic removal. Inexpensive.
Sorption on filter media – activated alumina	+/ ++	+++	Tested in a pilot scale in community and household systems, in industrialised and developing countries. Can remove both forms of arsenic. In most areas chemical oxidation is not required. Regeneration requires strong acid and base and produces arsenic-rich waste. the long-term performance of regenerated media needs documentation. Water rich in iron and manganese may require pre-treatment to prevent media clogging. Moderately expensive.
Sorption on filter media - ion exchange resins	-	+++	Tested on a pilot scale in central and household systems, mostly in industrialised countries. Interference from sulphate and high TDS. High adsorption capacity, but long-term performance of regenerated media needs documentation. Water rich in iron and manganese may require pre-treatment to prevent media clogging. Chemical oxidation required for water with high As(III) proportion. Moderately expensive. Regeneration produces arsenic-rich brine.
Sorption on filter media – other media	+/ ++	++/ +++	Has been shown to be effective in laboratory studies in industrialised and developing countries. Needs to be evaluated under different environmental conditions, and in field settings. Sometimes chemical oxidation required. Simple media are inexpensive, advanced media can be relatively expensive.

+++	Consistently > 90% removal
++	Generally 60-90% removal
+	Generally 30-60% removal
-	< 30% removal
?	Insufficient information
n.a.	not applicable

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Web sites

Bangladesh Arsenic Mitigation Water Supply Project <http://www.bamwsp.org>

WaterAid reports: <http://www.wateraid.org.uk/research/BangladeshA.htm>

SANDEC on SORAS technology: <http://www.eawag.ch/news/arsen/d-index.htm>

British Geological Survey: http://www.bgs.ac.uk/arsenic/bphase1/b_intro.htm

WHO: <http://www.who.int/inf-fs/en/fact210.html>

http://www.who.int/water_sanitation_health/Arsenic/ArsenicUNReptoc.htm

WELL: <http://www.lboro.ac.uk/well/resources/fact-sheets-htm/aidw.htm>

Environmental Protection Agency (USA): <http://www.epa.gov/safewater/arsenic.html>

24 Water supply in disasters and emergencies

Tom de Veer

24 Water supply in disasters and emergencies

24.1 Introduction

This chapter focuses on the provision of drinking water supplies in emergency situations. It suggests both immediate and longer-term inputs that can be implemented by external agencies or by communities themselves.

To distinguish between a disaster and an emergency, we use these common definitions:

A **disaster** is a natural or manmade event that causes physical loss or damage, social and/or economic disruption and threatens people's lives either directly or indirectly.

A community¹ confronted with a disaster will need to react in order to save the lives of its members and prevent suffering as much as possible. However, the disaster may stretch beyond the capacity of a community and cause an emergency.

An **emergency** is a situation of hardship and human suffering arising from a disaster, which develops if the organisational infrastructures in place cannot cope with the situation.

In an emergency the affected population will need external assistance to ease the hardship and suffering to a bearable level and to minimise mortality and morbidity levels.

Disaster and emergency response operations may be very different, depending on a wide range of aspects, such as:

- scope and duration of the response needed: ranging from only required immediately after the event for a small number of specific aspects, to broad interventions required for several years after the event;
- presence of local and national authorities;
- level of operational capacity and effectiveness of authorities;
- degree of effective response of local authorities and other local actors/leaders;
- high security risks in area (especially in conflict areas);
- logistical and resource problems for agencies and authorities involved;
- location of affected populations (i.e. displaced, partly displaced or not at all);
- availability of water sources and their quality and quantity.

From a technical point of view drinking water systems used in response to an emergency may not differ much from similar systems used in development situations.

1 In this chapter a community is regarded as a group of people with some kind of longer-term coherence, like a town, a village, a neighbourhood, or a group of refugees.

However, in emergencies the factors that affect the choice of the drinking water systems² usually differ considerably from the factors influencing such choices in development situations. This often leads to the choice of a different drinking water system in an emergency from that in a non-emergency. In emergencies the most important decisive factors for drinking water interventions often are:

- security situation regarding accessibility to the area for national/international organisations and peoples freedom of movement;
- access to the area in terms of roads and topography;
- socio-political, legal and cultural constraints;
- availability of water sources and their characteristics;
- time required to develop the water sources;
- time required to mobilise the required resources;
- characteristics of the affected population (number of people, displaced or not, extent to which coping mechanisms³ are still in place, etc.).

The initial aim of the intervention will be to achieve a survival supply of drinking water quickly so as to keep drinking water-related morbidity and mortality rates among the affected population within acceptable limits. If this is not possible, the affected population has to be moved to another, more favourable location. Once the survival supply has been achieved, work can start on realising a *longer-term supply* to provide people with more sustainable (less costly) and better drinking water facilities for as long as they need assistance.

Sometimes survival and/or longer-term supply systems can form the basis for the development of more permanent supply systems. Examples are the repair of existing water systems or the construction of new systems suitable for providing drinking water to the existing local population after the emergency has passed. Planners need to take account of this potential in choosing appropriate emergency interventions in situations where this is applicable. Efforts made to bridge the “gap” between the emergency and the “normal” or “development” situation are often described as *rehabilitation*. Repair of existing systems is an important option, especially in emergencies where the population has not been displaced.

Participation of beneficiaries in planning emergency supplies should be enhanced where possible. Beneficiaries are often well able to identify the most suitable locations for drinking water points, have local knowledge about the drinking water situation,

2 A “water system” in the context of this chapter includes all facilities, services and management inputs required to obtain, transport and treat water, and deliver it to the affected population.

3 The term ‘coping mechanisms’ refers to both the physical structures and means to survive, and the social relations with relatives and others that give basis for using and/or developing water sources.

hydrological circumstances, and existing water points. They are usually ready and able to provide their labour in the construction/rehabilitation of drinking water systems, and may be very effective in quickly spreading important hygiene messages. Important advice and inputs often come from key informants, local persons in powerful positions and local drinking water institutions. On many occasions these groups may be largely able to implement the required inputs themselves, needing only some external assistance with, for instance, the provision of equipment. On the other hand, effective participation by beneficiaries and local institutions may sometimes be difficult and time consuming in emergency situations. Facilitating participation requires the kind of skills that not all aid workers possess. Consequently, in emergency work, there are success stories but also failures with regard to participation. The critical question is whether participation can be made to work properly in the limited time frame available. Sometimes this is the case, sometimes not, and it is often not easy to predict beforehand what the best solution is.

The following points may provide some guidance for those in charge of an emergency intervention:

1. It is vital to obtain as much information as possible about the existing situation and immediate needs right at the start. That means gathering information quickly from the expected beneficiaries (women are particularly important informants with regard to drinking water needs), and from local institutions, key resource persons, community representatives and the people in power. Based on these consultations and other information (e.g. personal observations), a quick decision should be made, involving all the parties insofar as it is possible to do so. If time is very pressing this stage may have to be finalised within a day, sometimes even hours.
2. Those in power have to agree to the proposals before action can start. Assess to what extent those in power demand to be involved and judge what decisions you can make yourself and what decisions you cannot.
3. Depending on the situation, labour contributed by beneficiaries may be paid or voluntary. In camps, often labour provided for construction works is paid, but looking after tap stands is a voluntary input. Those involved in the operation of the system are paid. In most cases salaries are lower than usual, with the argument that beneficiaries already obtain food and other services free in the camp. It is important to have salary scales similar to other NGOs active in the area.
4. Active involvement of existing local drinking water institutions in the interventions may be very effective, as such institutions are usually familiar with the existing systems, have knowledge about suitable water sources, have key staff, equipment, and access to a local network of decision makers, and so on. However, local institutions may be overwhelmed by the events and/or be so weak in their performance that they can also be more of a threat than an asset in the initial

disaster response. A key consideration is whether the institutions are likely to provide the required inputs within the available timeframe. This is often not the case because local institutions are in most cases not set up for rapid interventions. Often it is easier, quicker and better to hire a number of beneficiaries for the work and supervise them directly yourself, than to provide guidance to a slowly operating local institution.

5. Participation of beneficiaries in the construction of survival supplies is possible and advisable if it contributes to increased quality of the intervention and/or time gain. It may apply to the siting of the water service points and the control over their use. Consultations with women and children help to adjust the design and location of the facilities to match their needs. The involvement of representatives from the community and, if it is a displaced community, persons with specific knowledge and skills on water system construction and management, helps to achieve sustainable water services⁴.
6. Managing community participation requires a high professional level of planners and implementers (including sometimes professionals from among the beneficiaries). Decisions are needed on whether participation will provide the expected results and how such participation should be organised within the given circumstances and time limits.

Minimum requirements for longer-term supply interventions in response to emergencies have been developed by the Sphere project (WHO, 1993), a collaborative initiative of a large number of international humanitarian organisations. Table 24.1 shows some of the most important minimum standards for both survival and longer-term emergency drinking water systems in comparison to guidelines often used in development situations.

In chapter 2 the importance of linking water supply with hygiene and sanitation was explained. The environmental sanitation conditions in emergency and refugee situations are usually very poor. The risks of transmission of water and sanitation related diseases are very high because human waste disposal containing human faecal borne pathogens are often indiscriminately spread around in the area. Therefore, apart from the provision of water supply – the focus of this publication and chapter – sanitation improvement and hygiene promotion are key actions to prevent the outbreaks of diseases in these conditions of despair, uncertainty, poverty and crowding.

4 Women need to be brought into the picture as well as men. Emergency workers are often men and local women who have been affected by the disaster will be largely occupied with care for their children. This creates a risk that only local men's knowledge about water supply will be included and not the quite different perspective of women.

Table 24.1 Important guidelines for survival supply, longer-term supply and development drinking water projects

Standard	Survival supply in response to an emergency	Longer term supply after an emergency	Development supply
Water availability per beneficiary ⁵	3-5 l per person per day (for survival supply)	15 l per person per day	20-50 l per person per day
Number of water collection points	1 point per 500-750 persons	1 point per 250 - 500 persons	1 point per 200-300 persons
Distance from water collection points	1 km	500-700 m	100-400 m
Maximum waiting time at water collection points	2 hours	20 minutes	no guideline
Turbidity	< 20 NTU	< 10 NTU	< 5 NTU
Residual free chlorine at water collection point	0.3 -1.0 mg/l	0.2-0.5 mg/l	if water is chlorinated: 0.2 mg/l
Conductivity	< 3000 μ S/cm	< 2000 μ S/cm	< 1400 μ S/cm
pH	No restriction	6-8 for coagulation with aluminium sulphate; < 8 for disinfection	Preferably < 8 for effective disinfection with chlorine
E. coli or thermotolerant coliforms	Always aim to disinfect supplies. If this is not possible then use the best available source and apply solar disinfection for the drinking water	Always aim to disinfect supplies. If this is not possible then: < 10 thermotolerant coliform (E. coli)/100 ml (and apply solar disinfection for the drinking water)	0 thermotolerant coliform/100 ml (also in this case: apply solar disinfection for the drinking water)
Household collection and storage capacity	1 container of 10-20 l for collection per family. The container has a narrow neck and/or a cover	2 containers of 10-20 l for collection and 0-1 storage container of 20 l per family. The containers have a narrow neck and/or a cover	no guideline

(these guidelines have an unofficial status and come from many sources)

5 There may be considerable water needs as well for livestock, anal washing, medical centres, feeding centres, or even irrigation.

The rest of this chapter deals with technical aspects of drinking water systems used in survival and/or longer-term supply interventions insofar as they differ from “regular water supply development”. For a comprehensive methodology for the selection of drinking water systems for both survival and longer-term supply in response to emergencies reference is made to House and Reed (1997). De Veer (1999) suggests quality systems for the operation of a number of different drinking water systems in camps. Davis and Lambert (2002) provide a comprehensive overview of drinking water techniques used in emergency response operations.

24.2 Water tankers

The provision of drinking water by tankers is a solution for survival supply when time is very limited and other systems cannot be realised within the time limits. Water supply by tankers is only possible when certain requirements are fulfilled (see below). Usually these requirements can be met locally. Otherwise trucks, repair facilities, etc. have to be brought in by road or air from elsewhere. Water supply by tankers is almost always planned and managed by specialised external agencies because of the complexity of its management and the high costs. This is not, therefore, a solution for longer-term water supply, although examples exist where large water tanker operations have continued for many months. Water tankers are also often used to supplement other water services.

Water tanker requirements

- Water tankers are normal trucks or trailers with storage vessels mounted on them (Fig. 24.1). If hiring or purchasing water tankers or storage tanks locally, you must know what they have been used for. Before use, rigorous cleaning and disinfection (super chlorination) is needed. Regular cleaning/disinfection during operations is also a good practice.
- The water source must have adequate yield and water of sufficient quality (biological contamination is usually not a direct problem because the water will be chlorinated). The place should allow for pumping into the water tanker. Permission will need to be obtained from the owner and water rights from the authorities.
- Water pumps are needed at the source and perhaps at the delivery location; a pump to empty the tanker quickly can save a lot of time.
- Water storage capacity and some tap stands need to be provided at the delivery location.
- Fuel and lubricants need organising for the trucks.
- A sufficient supply of chlorine has to be available for chlorination of the water;
- There must be enough truck drivers.
- O&M facilities include mechanics and tools, equipment and spare parts.
- Road access has to be available between the source and the beneficiaries (check road conditions, bridges, permission to use the road from local authorities, owners or those in power in the area).

- It may be necessary to equip the trucks with communication equipment, especially where security can be a problem or where the situation can change very quickly.
- Managing the water tankering operation properly demands professional skills and experience.

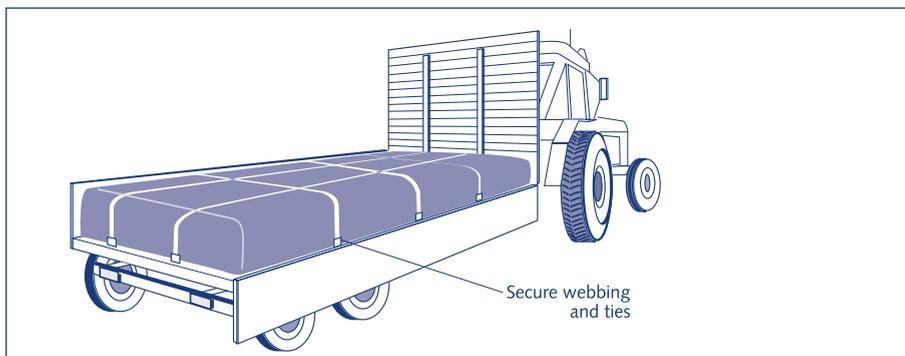


Fig. 24.1. Collapsible pillow tank mounted on a trailer
Source: Davis, J. and Lambert, R., 2002

Water tanker management

The effective operation of water supply by tankers requires good planning and management of staff, logistics and stocks. There are many pitfalls. Table 24.2 is an example of a water tanker schedule, illustrating the basic assumptions that need to be verified and the way that the required supply capacity is calculated.

Water should be chlorinated at the filling point if possible. A fixed amount of chlorine (usually sodium-hypochlorite) should be added to each tanker, according to the volume of water in the tanker. The journey will allow the chlorine to mix well with the water. The contact time should be at least 30 minutes. The chlorine dosing may vary depending on the turbidity and organic content, and therefore the dosing needs to be determined regularly. Monitoring of residual chlorine levels is recommended.

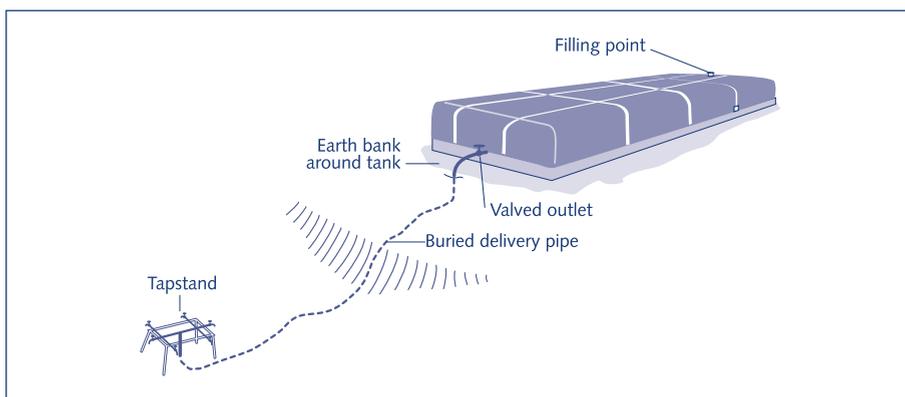


Fig. 24.2. Typical set-up of a distribution point in a tankering operation
Source: Davis, J. and Lambert, R., 2002

Water delivery points should be well organised with sufficient space for the trucks to off-load quickly in large enough water tanks. Crowding of people at the water distribution point needs to be prevented. A better solution is to have a storage tank with a number of tap stands situated at a sufficient distance from the tank, as in figure 24.2.

Table 24.2 Example of a water tanker supply schedule

Basic assumptions:

Population:	10,000 persons
Minimum water requirement:	15 lcd
Distance filling station to distribution point:	15 km
Speed of water tanker:	20 km/h
Time required to fill the tanker at filling point:	20 minutes
Working hours per day:	12 hours
Water tanker capacity:	10,000 litres

Calculation of amount of water to be tankered:

Daily water requirement:	150,000 litres
Allow 20% extra for wastage and for new arrivals:	30,000 litres
Water to be tankered:	180,000 l/day

Calculation of gross turnaround time:

Time to fill the tanker:	20 minutes
Journey from filling station to distribution point:	45 minutes
(assuming that all delivery points are at approximately the same distances, which is not always the case)	
Time for emptying water tanker at distribution point:	8 minutes
Return journey to filling station:	45 minutes
Net turn around time:	118 minutes
Add 30% for contingencies:	35 minutes
Gross turnaround time:	153 minutes, say 2.6 hrs

Calculation of the number of water tankers required:

Number of deliveries per tanker per day = working hours in a day/gross

turnaround time = $12/2.6 = 4.6$

Amount of water delivered per water tanker per day = $4.6 \times 10,000 = 46,000$ litres

Numbers of water tankers required: $180,000/46,000 \approx 4$

24.3. Water intake, storage, treatment and distribution systems

Water transport and distribution systems constructed in response to a disaster/emergency situation follow the same principles as those constructed under normal conditions (see chapters 20 and 21). They may differ in certain details due to the factors described in section 24.1. Also the choice of a specific type of system may be dictated by the special circumstances prevailing in an emergency.

Construction of water transport and distribution systems in response to a disaster/emergency is usually done through paid labour, while in development situations much more use is made of voluntary labour from the beneficiary community. The common reasons are that, following a disaster, social coherence among the affected population is less than in "normal" communities, there is lack of a common feeling of responsibility, and people may believe that the facilities will not be beneficial to them for a long time. These factors can make it more difficult to mobilise beneficiaries on a voluntary basis. This is not a rule, though, and exceptions do exist.

Intake

If the water source is surface water then a temporary intake structure can be created by building a small weir made of bags filled with sand. The end of the inlet pipe needs to be covered with some kind of filter (netting) and positioned in the river directly upstream of the weir. The pumping equipment can be whatever is commonly available in the country; straightforward technology gives the best chances for reliable functioning. A source caretaker (a paid staff member) is required to make sure that the pump does not operate when the water level is too low.

Advantages of simple intake structures are that the materials required can be easily obtained locally or flown in quickly, construction requires little time and can be done without highly skilled technicians, management is easy, costs are low and the structures can be dismantled easily. Upgrading of the system may involve the use of floaters attached to the inlet, in case the water level rises, and small diversion structures to cope with flow variations in the river (Fig. 24.3).

Wherever possible, the water catchment area should be protected to minimise the risks of pollution. Human settlement, agricultural and livestock activities should be prevented as much as possible anywhere near or upstream of the intake. Part of the source assessment should be the identification of present or potential pollution sources upstream of the intake. If pollution threats are found, then the intake may have to be positioned further upstream (beyond the polluting source), or another reliable and safe water source needs to be found, or the people need to be settled elsewhere.

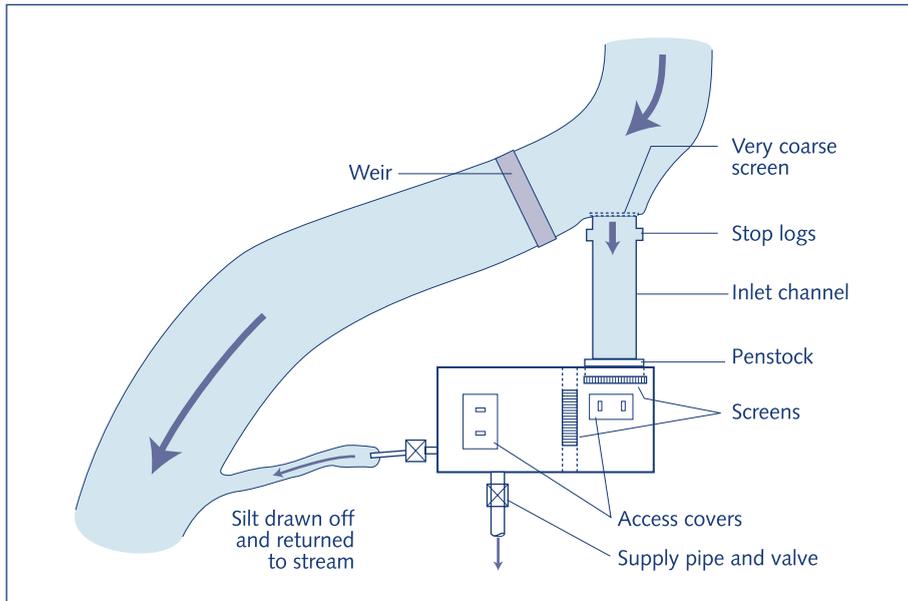


Figure 24.3. A diversion structure for a stream intake
Source: Davis, J. and Lambert, R. 2002

Pipes

The quickest response is achieved by using pipes that are locally available even if they are not an ideal solution.

Flexible hoses are suitable for survival supply. They usually have to be brought in from abroad but can be flown in and are easily transported over land, can be installed on site very quickly (rolled out) and connect easily to other types of pipe. However, flexible hoses are more expensive, more vulnerable and less durable than other pipes. Some flexible hoses, though, can resist quite large pressures. Two types are distinguished: lay-flat hose made of canvas or PVC material (collapsed when not in use, making it easy to handle and transport), and PVC suction and delivery hose made of flexible smooth bore PVC, reinforced with a semi-rigid PVC spiral.

Polyethylene (PE) pipes are suitable for survival supply due to the ease of jointing and the flexibility of the pipe. Jointing can be done through fusion welding or push-fit or mechanical couplings. All these jointing options are easy and quick and give very strong joints that can withstand end loading. They therefore do not need thrust blocks at points where the direction of the pipeline changes. Several couplings also allow connection to galvanised iron (GI) pipes and it is possible for low-pressure applications to screw PE pipe onto GI threaded pipe and fittings. The flexibility of PE makes it possible for smaller-diameter pipes to be delivered in long coils. This way only few joints are required and pipe can be laid very quickly. Even in freezing conditions PE remains flexible and can be laid easily. It also slows down freezing of the water in the pipes due

to its thick wall and low thermal conductivity. Even if the water does freeze, the pipes will not break easily. Transport of the coils can sometimes be difficult because they require quite a bit of space. PE is often locally available. PE can initially be laid over land if necessary because it is quite robust and not very sensitive to sunlight (use the black pipes, not the blue ones). If laid over land, it is of course more vulnerable to damage by people walking over it and vandalism – and this danger should not be underestimated. Pipes should therefore be buried in trenches as soon as possible.

PVC materials may be used in survival supply situations when they are locally available and if there are no better alternatives. Large diameter and/or high-pressure class pipe often needs to be PVC. PVC is light and can be more easily transported to site than the heavier GI pipes. GI pipes are also much more expensive. However, during transport and storage, PVC is vulnerable to breakage and it becomes brittle at low temperatures or when exposed to sunlight. It should therefore not be laid over land or used in areas where it can freeze unless it is buried below the frost line. PVC pipes with push-fit joints are appropriate for survival supply because they can be laid quickly. They can also be easily dismantled compared with PVC pipes with solvent cement joints. This can be important for instance when refugees have returned to their home areas, leaving the infrastructure in the camp behind. However, push-fitted PVC pipes cannot sustain end loads; they must be anchored in position by backfilling the trench before pressurising the pipeline. Bends, blank ends and fittings must also be adequately anchored. If necessary it is possible to bend a PVC pipe by filling the pipe with sand and heating the area to be bent, although this will affect the strength of the pipe.

Water tanks

A typical water tank for disaster/emergency situations should be easily and quickly installed, should be suitable for quick transportation to the site (so light and easy to pack materials are needed) and should be easy and quick to dismantle at the end of the operation. Two types of tanks are specifically suitable for emergency response operations: OXFAM tanks and collapsible tanks.

OXFAM tanks

OXFAM tanks are made from corrugated steel sheets that can be bolted together to form the round casing for the tank. A synthetic rubber lining is hung inside and a plastic sheet can be put on top to form a roof (Fig. 24.4). The tank can easily be brought in parts and erected on site. An experienced team can erect an OXFAM tank within a day and it is fairly easy to learn the job (each tank comes with a self-instruction manual). There are OXFAM tanks with volumes ranging from 10.5-95 m³. The tanks are very durable and can last for many years. They usually have to be brought in from abroad. Main uses are water storage and treatment in both survival (if the required materials are on site in time) and longer-term supply systems.

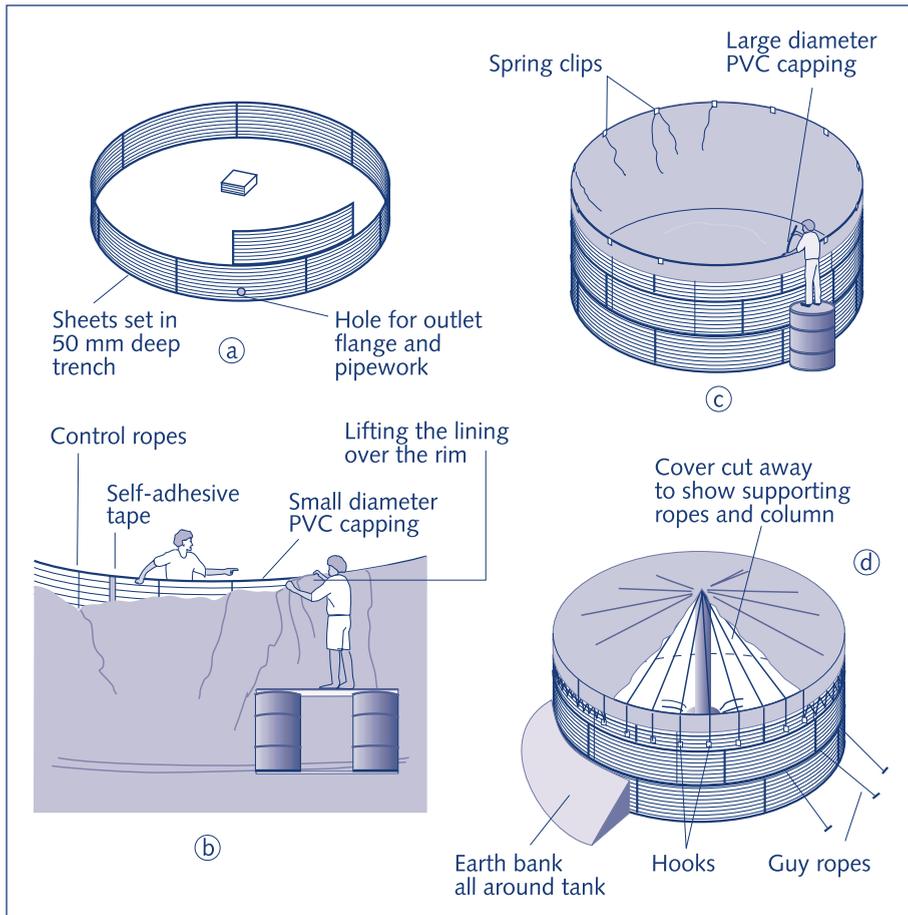


Fig. 24.4. Erection of an OXFAM tank
(Adapted from: Davis and Lambert, 2002)

Collapsible tanks

These tanks are made of collapsible fabric, without any supporting framework. They have a low packed volume and are easy to handle. An advantage is that they are very light and can thus be transported to site easily (over land or by air). However, they are usually not locally available and therefore have to be brought in from abroad. They are not very durable either and are vulnerable to vandalism (sticking a knife in causes major leakages, although the hole can be repaired relatively easily). Main uses are in situations where very quick action is required (within hours/days) and where no other tank materials are available. The tanks have an outlet that can be connected to a flexible hose leading to a tap stand at a slightly lower elevation than the tank (Fig. 24.2). In areas where the temperature can fall below zero, collapsible tanks should be placed inside buildings (e.g. in warehouses). There are two types of collapsible tanks:

a. Bladder tanks (also called "pillow tanks") are available in volumes of 2.5-20 m³.

The material is UV resistant high tensile EVA or PVC coated polyester. They should be positioned on a properly levelled piece of land from which all stones and sharp

obstacles have been removed (Fig. 24.2). It is advisable to place a groundsheet under the tank. Sometimes the tank can be fixed to the ground with pegs, otherwise it is advisable to form an earth bund around it to avoid any movement.

A disadvantage is that it is difficult to remove silt and sludge from the tank.

- b. Flotation collar tanks (also called “onion tanks”) are available in volumes of 1.8-90 m³. The material used is synthetic rubber. The tank has a self-supporting buoyant foam-filled or inflatable rim and a loose cover. This tank can to a certain extent also be used on slopes.

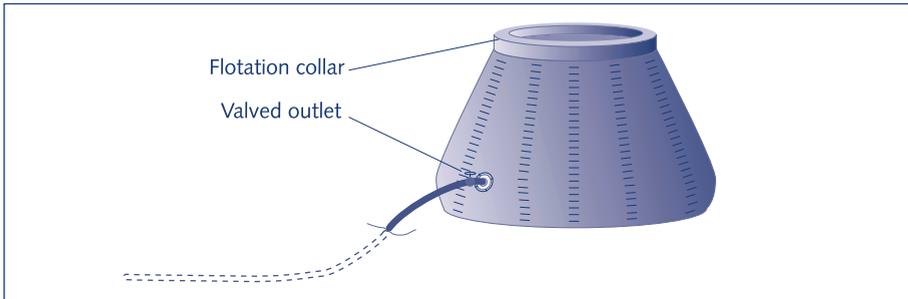


Figure 24.5. A flotation collar tank
(Adapted from: Davis and Lambert, 2002)

Water treatment

In disaster/emergency situations, water treatment complicates the water delivery process and should therefore be minimised and always be simple. It is better to look for a water source supplying water of safe quality wherever possible. When treatment is found to be necessary, deciding which treatment system is most suitable depends not only on the raw water quality but also on the availability of construction materials and chemicals.

Simple water treatment systems include the following:

1. **Sedimentation** using a large OXFAM or other type of tank.
2. **Coagulation/Flocculation.** Often used where no time is available to wait for the construction of filtration units and where the required chemicals (usually aluminium sulphate) can be made available. The system can be operated as a batch or continuous system. Staff should be well instructed and be monitored on their performance.
3. **Multi-stage filtration** (two coarse material filtration systems followed by slow sand filtration – see also chapter 16). These filters can be constructed fairly quickly (within a week) if use is made of OXFAM tanks and if the required coarse gravel and sand can be found nearby (figs. 24.6 and 24.7). If the turbidity of the water is not too high, large amounts of water can be effectively filtered. A well-trained and supervised crew is required. Also small multi-stage filtration units can be set up for specific applications (for instance for health and feeding centres) (Fig. 24.8). Other designs are given in chapter 16.

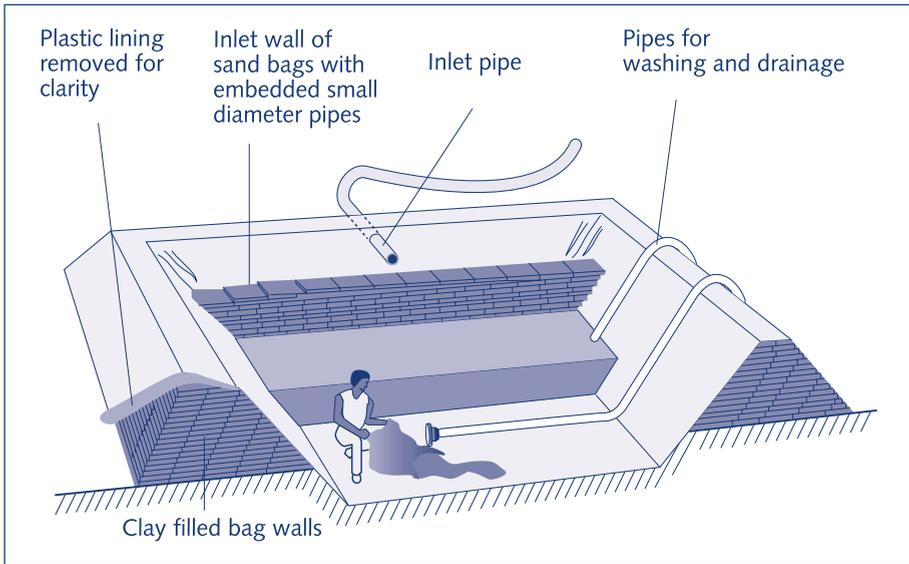


Figure 24.6. Cross-section of an excavated horizontal flow roughing filter (Adapted from: Davis and Lambert, 2002)

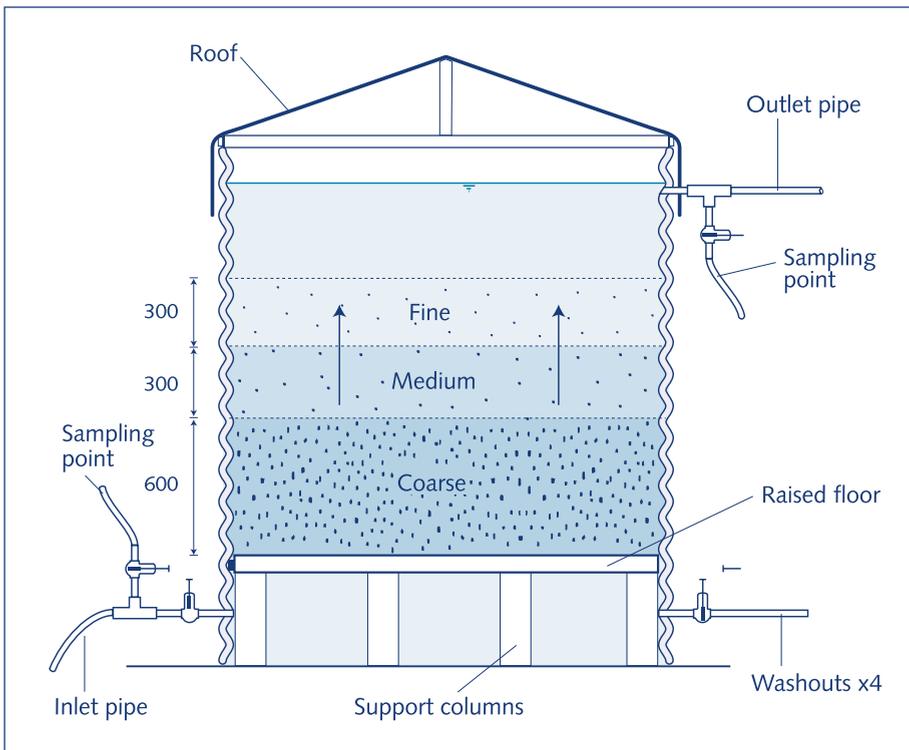


Figure 24.7. A vertical flow roughing filter (Source: Oxfam, 2000) (Adapted from: Davis and Lambert, 2002)

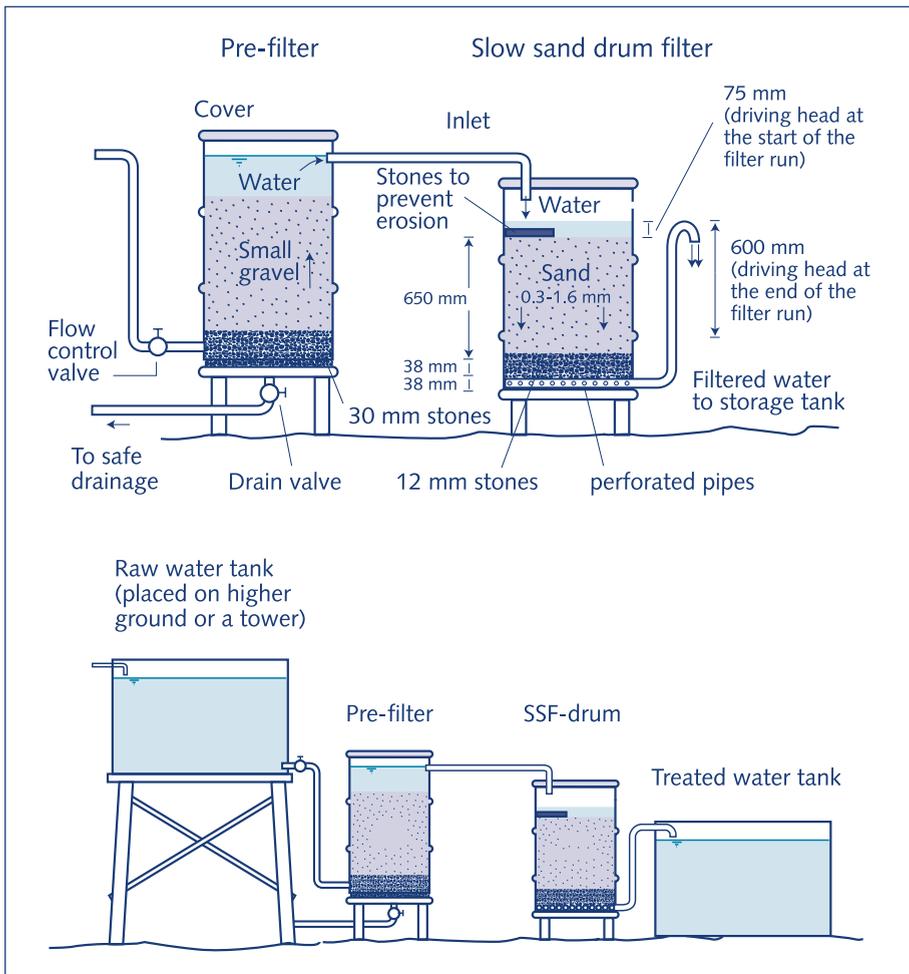


Figure 24.8. Small-scale multi-stage filtration units
Source: Davis, J. and Lambert, R. 2002

4. **Chlorination.** It is recommended always to chlorinate the water in disaster/emergency response operations. In a piped water supply system this is usually done as the last step of the treatment process. For survival water supply a batch system is the easiest: two small OXFAM tanks (of about 10 m³ each) are filled with water and the required amount of chlorine is added. The chlorinated water should stand for 30 minutes before it is released into the distribution system. The most suitable chlorine solution is hypochlorite dissolved in water. A trained staff member should add the chlorine to the chlorine storage tank wearing protective clothes and goggles. Appropriate chlorine dosing equipment is shown in chapter 19.

The number of consumers and the changes in water quality determine the frequency of checking for residual chlorine at the tap point. It should be done at least once a month, but if there are thousands of people using the water supply, it should be checked at

least daily. The required dose of chlorine may change over time due to changing characteristics of the water, for instance when the rainy seasons starts with more risk of contamination. If the residual chlorine is less than 0.3 mg/l then a trained staff member should determine how much chlorine is actually required and adjust the amount of chlorine to be added to the tanks. Free residual chlorine is an indicator that there is still disinfecting capacity in the water. The residual chlorine content will drop during transport between the point of adding the chlorine and the tap point as the chlorine kills micro-organisms. If the distribution pipes are not permanently under pressure, contaminated water may enter the system at points where leaks are present. That consumes more chlorine, and cause the residual chlorine level to drop.

Public standposts

In most disaster/emergency response operations water distribution is through public standposts. Private connections are not common unless existing systems are used. Special connections will be needed to public facilities such as field hospitals and health posts, feeding centres and market places. These facilities have their own water requirements, which should be included in the calculation of the total water demand and the design of the water supply.

For public standposts, usually push taps, also called self-closing taps, are used. These are robust taps that close automatically under their own weight after a user stops pushing the weight up. The taps help in this way to diminish water wastage, if users do not close the taps due to negligence. Sometimes people do keep the taps open on purpose by attaching a piece of rope between the weight and the pipe above. They may do so for instance if they want to use the water for irrigation, small industrial activities (beer brewing), etc. Particularly in situations where water of drinking water quality is scarce - which is usually the case in disaster/emergency situations – this practice should not be allowed and needs to be strongly controlled.

Each standpost should have a voluntary caretaker elected by the people using the water from it. The caretakers should be trained, monitored and guided by higher-level staff. Proper selection and training of caretakers can be quickly arranged, even in survival supply systems. It is, for instance, important to have the beneficiaries discussing the character and qualifications of the person they need as a caretaker. If this is done well, people often choose women to be caretakers. It is worthwhile to pay regular attention to and motivate the caretakers. This can be done by organising small workshops for them that may also provide good feedback to the programme, and by giving them small incentives (e.g. soap) once in a while. It is also important to give them a recognised status and acknowledge the work they do, for instance by inviting them to meetings and not taking their work for granted. But the main assurance that things will go well at the tap stand is regular supervision by higher-level staff.

Management

Larger piped water supply systems are used mostly for longer-term supply. Where these systems consist of repaired, rehabilitated and/or upgraded existing drinking water systems all efforts should be made to involve the local community and the responsible local water agency (which could be under the local government) from the beginning and throughout the project cycle. Always make sure that the different user groups are represented and inform and consult them. Depending on the size of the new system and the increase in the number of temporary consumers, the community and/or the water agency may be fully in charge or contribute to the management of the systems and service. In general management is carried out by external staff, to avoid learning time that may cost unnecessary lives.

Where large pipeline systems have been realised under other circumstances, such as in camps, management is often executed by external agencies. Sometimes this situation may last for many years. Only in a few cases has the management of such systems been handed over successfully to local beneficiary organisations.

An example of a successful handing-over of the management of a water supply system to beneficiaries is the camps for Afghan refugees near Peshawar in Pakistan. Here external organisations simply did not have the resources anymore to pay for the operation and maintenance of the pipeline systems. Another factor was that to a high degree the Afghan refugees were incorporated into the local economic system, which enabled them to pay for the water services and motivated them to get a grip on their own situation.

24.4 Spring protection

Where springs are a suitable source they can be quickly protected by installing a *mini-filter* (Fig. 24.8). If the amount of storage in the collection pit with the filter layers is small it is better to have an open outlet instead of a tap at the collection point to avoid possible build-up of back-pressure with the danger of the spring diverting to an alternative route. Instead of a tap that may break easily if used heavily, a simple wooden plug can be used. Drive a small bar horizontally into the ground to mark the level of the groundwater before disturbing the site. The bar will show the level below which the filter element with the outlet pipe has to be placed. The spring site should be cleaned and vegetation cleared. Care must be taken not to disturb the underlying impermeable layer; the spring could be lost by the diversion of groundwater away from the original eye. Ensure good drainage at the collection point. Fence the area 10 m around the spring to prevent contamination from beneficiaries and/or their livestock in the immediate surroundings of the spring. If the catchment area is in danger of contamination a larger area may have to be fenced (and/or guarded). A diversion ditch

drains the rainfall runoff away from the spring. To avoid losing good quality water the outlet pipe can be connected to a water tank (for instance a collapsible tank (Fig. 24.2 and 24.5).

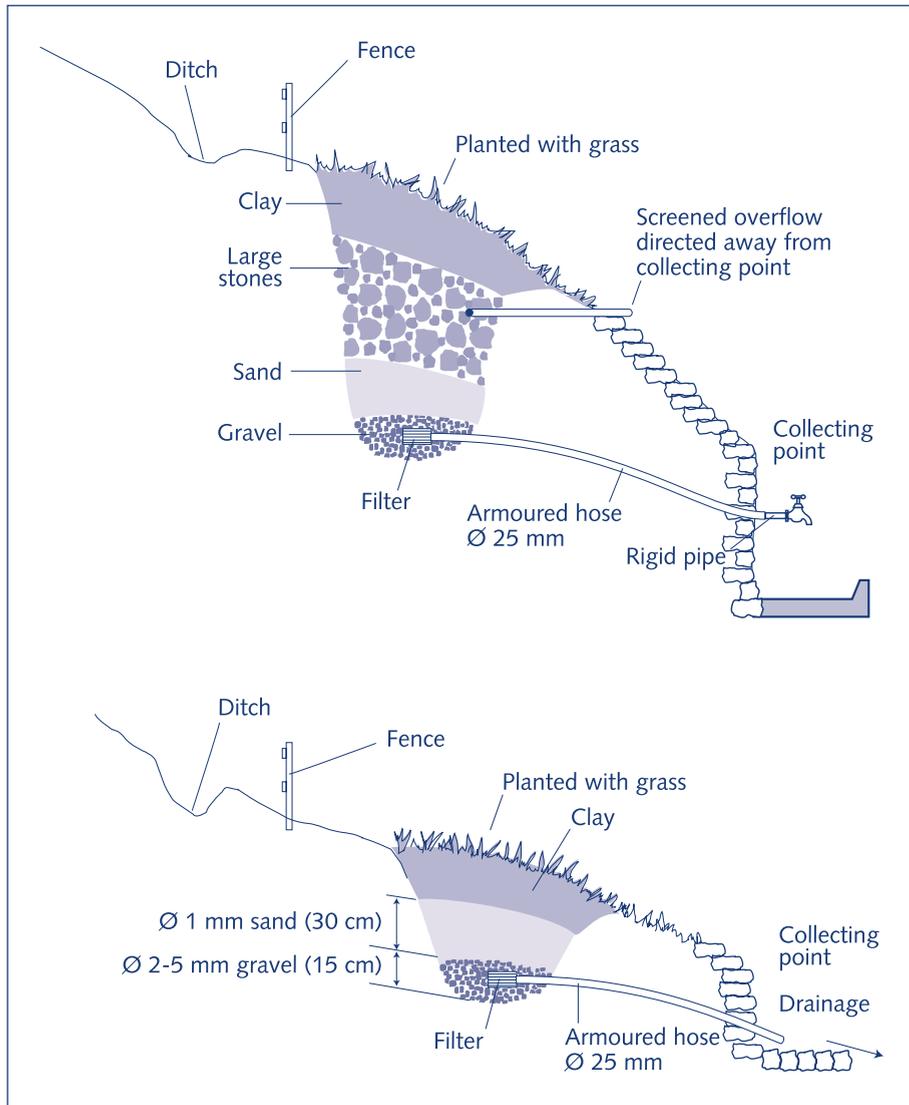


Fig. 24.9. Two "emergency" type spring protections
Adapted from: Davis and Lambert, 2002

Diffuse groundwater seepage can be collected through a "seepage spring protection" consisting of carefully placed infiltration trenches as shown in figure 24.10.

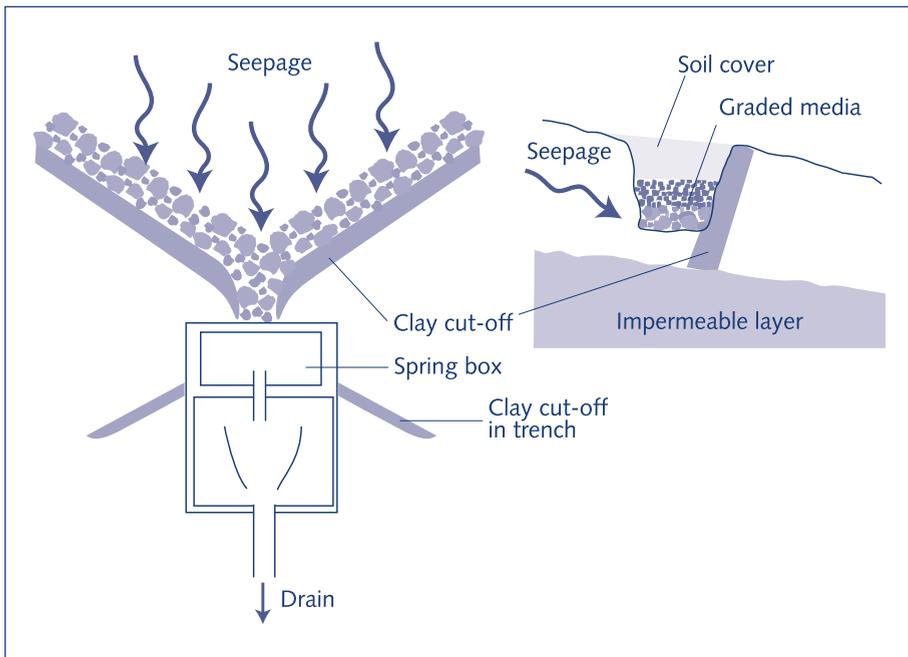


Fig. 24.10. Seepage spring protection
Source: Davis, J. and Lambert, R. 2002

24.5 Wells and boreholes

An important factor in the initial stages of an emergency response operation usually is whether wells or boreholes can be developed in time. From a technical point of view there is no difference between an emergency or a development situation with regard to wells and boreholes, although some methods for the development of wells or boreholes are quicker than others. Typical factors that play a role in the decision-making to consider wells/boreholes as an option are:

- availability of suitable companies for the drilling of boreholes; they may be there but already be hired by other organisations;
- the hydro-geological characteristics: do they allow for quick development of a well or borehole?
- available equipment and expertise.

In emergencies it may initially not be possible to develop a well/ borehole because of the time factor, but it may be a very good option for longer-term supply following on survival supply. For survival supply it is important to look at existing wells/boreholes. There may be existing wells/boreholes that are not in use; sometimes they only need to be cleaned and disinfected. Or existing wells/boreholes can produce much more water than they actually do; this extra production capacity may be temporary, for instance during the rainy season, but this will then at least provide a suitable source for some time, allowing time to develop other water sources. It may also be possible to increase

the capacity of existing wells/boreholes, for instance by digging or boring deeper. Obviously these measures and uses have to be discussed with and approved by the local authorities and the owner.

In a non-emergency situation a point source such as a well or borehole usually provides up to a maximum of 300 people with water. For survival supply, a well or borehole can provide water to 900 people at a normal pumping rate because the collection of water will be more equally spread over the day and not with the high collection peaks of more normal situations. The amount of water required per person is also less. The production of a large diameter well can be increased by placing several handpumps on the same well, provided the aquifer allows for more abstraction. To avoid contamination of the groundwater the direct area of a well or borehole will need to be fenced and possibly guarded to keep away people and/or their livestock. Also protection against theft and vandalism is often required. If pumping is by motorised means, the standposts should be more than 10 m away (preferably more than 50-100 m), to prevent pollution of the direct well/borehole environment.

24.6 Rainwater catchment

Rainwater can be collected from roofs – these can also be tents or plastic sheeting - or ground surfaces. To avoid contamination plastic sheeting can be laid on the ground to catch the water. Rainwater can be an important source for survival supply, but the problem is that one cannot be sure that sufficient rain will fall just during the days it is required. Also, those who catch the water need to have sufficient storage capacity. See also chapter 7 on rainwater harvesting. Disinfection is usually needed to ensure good quality of drinking water. This can be done by using chlorine tablets, solar disinfection or boiling. Boiling is often not a feasible option because of lack of firewood and the flat taste of the water. See also chapter 19 on disinfection technologies.

24.7 Household facilities

In disaster/emergency situations affected households always need drinking water containers for fetching water and for storage in their house or shelter. Often people have lost these items during the disaster and they need to be made available. An important criterion is that the containers can be well closed and easily carried, by children as well as adults.

Typical containers are

1. Collapsible containers of 5-15 litres with a narrow neck and screw cap: easy to transport to site quickly and in large numbers, but often with a very short life time; they can break easily, sometimes even within days. These containers usually are not available locally and have to be brought in from abroad.

2. Jerry cans (usually plastic) of 15-25 litres with a narrow neck and screw cap. They have a long lifetime but are often difficult to transport to site in large numbers. Often these jerry cans can be purchased locally at low cost.
3. OXFAM bucket (Fig. 24.11). A bucket of 15 litres with a cover with a small hole plus attached push cap. Because the buckets can be stacked inside each other they can be more easily transported to site in large numbers quickly. While they are quite durable, they are expensive and not available locally. The form (round) makes them less easy to carry and the cover may get lost.

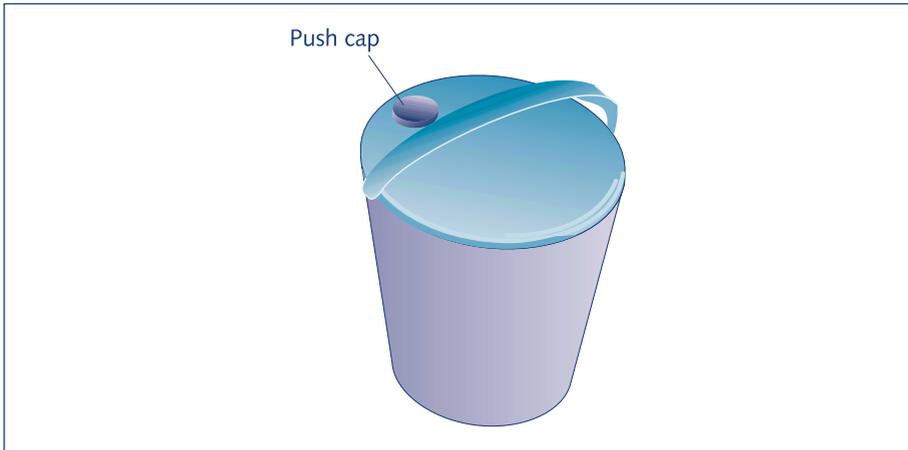


Fig. 24.11. OXFAM bucket.
Source: <http://www.oxfam.org.uk>

It may also be necessary to provide households with chlorine tablets, particularly when the water cannot be purified centrally and where water is contaminated to a dangerous level. Proper instructions and regular follow-up are of utmost importance to prevent people using too little or too much chlorine or even consuming the chlorine tablets thinking it is some kind of medicine. It is useful to try to size the tablets such that only one is needed for the size of water container that people have available. However, the amount of chlorine may change over time, the size of the containers people have may vary and the disinfecting capacity of the chlorine tablets may differ. If households are to boil their drinking water they should have access to sufficient amounts of fuel wood. This can have large environmental implications.

24.8 Emergency preparedness

To be prepared for emergencies actions are required at different levels:

- The international community should have the organisational infrastructure to provide assistance rapidly and effectively in case of disasters/emergencies, but should also assist poor countries to prepare themselves for coping with disasters and emergencies.

- From the national to the district level, authorities should work out disaster/emergency preparedness plans that indicate what assistance the national level should provide to local authorities and what assistance those levels should provide to the lower levels, etc., how to support NGOs, companies, etc. to cope and provide assistance when a disaster strikes.
- NGOs need to prepare themselves to provide assistance to communities struck by disasters/emergencies. They may also choose to play a role in facilitating authorities to prepare themselves, make them aware about the need for such preparations, and possibly help them with the resources required.
- Companies can prepare themselves for disasters by being ready to provide services and goods related to their business activities.
- Communities need to prepare themselves to be able to cope as well as possible with disasters themselves.

Persons with certain skills may want to assist in emergency work. People with specific useful skills can register with RedR⁶. This is an international organisation assisting in staff recruitment and training for disasters/emergencies. It is important for each of the parties involved in disaster preparedness that they work with scenarios for the disasters they may have to cope with and the specific roles they will have to, or want to, fulfil in such scenarios. An important principle in emergency work is to be as self-reliant as possible. If you rely on others for the execution of your tasks and the others don't perform well, you will also not be able to perform your tasks well. This is the reason why organisations choose to be independent from each other with, for instance, their logistical operations. It may cost more, but significantly reduces the risks. Especially for a community in a disaster prone area, it is important to assume that it will be alone when facing a disaster and it should use this assumption in the scenarios it defines when preparing for disasters.

Some specific measures that communities can take in order to be prepared for disasters that can potentially affect their drinking water systems:

1. **Ensure communication with the outer world.** Telephone lines and roads are often disrupted or not accessible after a disaster and the community could be cut off from the world, not even being able to request help. Therefore, it is helpful if there is radio equipment in or near the community. There should be a simple protocol agreed between the community and people who can be reached with the radio equipment on how to respond to and communicate in disaster situations. Both parties should have the protocol attached to the wall directly beside the radio equipment.

6 <http://www.redr.org>

2. **Ensure contingency supplies.** Sufficient supplies of spare parts, tools, equipment and chemicals, and construction materials for the drinking water system will enable continued operation and/or repair of the system even if supply lines are cut off for some time due to a disaster. The supplies can even be stored at different places if areas are prone to disasters that can affect restricted areas (e.g. where an area is prone to earthquakes, flooding or mud slides).
3. **Ensure sufficient skilled people.** When several people have knowledge about operational and maintenance tasks in the system, the chance is high that even if caretakers, bookkeepers, etc. become unavailable due to a disaster, other people are still available who can take over these tasks. Such 'spare' people should not only be trained initially, but should also be actively involved on a regular basis in executing the tasks they are supposed to be able to do.
4. **Place water tanks at safe locations.** Water tanks should not be placed where they could easily be affected by a natural disaster (for instance in locations prone to mud slides) or where water flushing out of breached tanks could cause great damage (for instance where water tanks are placed directly above inhabited areas).
5. **Have a contingency plan.** There should be a contingency plan that stipulates what needs to be done in case of a disaster and people in the community should have designated tasks for such occasions. A drinking water contingency plan should indicate who should do what. The involved key persons should discuss and go through the plan on a regular basis and adapt it to the circumstances whenever needed.

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http://www.sphereproject.org/handbook_index.htm

Web sites

REDR: <http://www.redr.org>

International Committee of the Red Cross: <http://www.icrc.org/>

Médecins sans Frontières: <http://www.msf.org/>

OXFAM International: <http://www.oxfam.org/>

Disaster Relief: <http://www.disasterrelief.org/Links/>

The Water Page: http://www.thewaterpage.com/response_and_relief.htm