

Artificial Recharge A Technology for Sustainable Water Resources Development

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Report to the Water Research Commission
by the
Cape Water Programme
Division of Water Environment & Forestry Technology
CSIR

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ARTIFICIAL RECHARGE

**A TECHNOLOGY FOR SUSTAINABLE WATER
RESOURCE DEVELOPMENT**

Final report to the
WATER RESEARCH COMMISSION

by

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EXECUTIVE SUMMARY

Artificial groundwater recharge has gained acceptance worldwide as an effective method of conserving water for future use, for improving water quality, for averting saline water intrusion, and for many other uses. In Southern Africa it has found only a few full scale applications but these have proved the viability of the technique, especially for primary aquifers.

Artificial recharge is considered as a practical, cost-effective and environmentally acceptable water management alternative for water supply authorities, rural communities and farmers. The aim of artificial recharge for water supply purposes is to rapidly replenish aquifers with water that would otherwise be lost through evaporation and stream flows. The subsurface conservation of water is of special significance in semiarid to arid areas.

RESEARCH OBJECTIVES

The aim of this project was to assess the feasibility of using artificial recharge technologies in South Africa for community water supplies. In order to meet the aim, the following objectives were identified:

- those artificial recharge techniques appropriate to South African conditions and needs must be determined;
- the assessment must include concepts such as conjunctive use, groundwater catchment management and groundwater dams (sub-surface and sand-storage dams);
- the appropriate technologies must be linked to areas/regions/hydrogeological settings;
- basic guidelines must be developed for establishing artificial recharge schemes; and
- a pilot scheme must be designed to test the concept (field site identification, and pilot scheme and test programme designed).

RESEARCH METHOD

The project was planned to consist of a desk study which would draw on international experience with respect to topics such as the hydrological factors which affect the potential for artificial recharge, and guidelines for establishing artificial recharge schemes. Suitable situations for pilot artificial recharge scheme applications were to be identified in consultation with experienced local geohydrologists.

Besides identifying pilot artificial recharge schemes, the key issues identified, and the tasks which were to form the basis of the research plan, included:

- identify potential artificial recharge users;
- identify the factors which affect artificial recharge potential;

- describe the hydrogeological requirements for artificial recharge;
- describe the water resources needed in order to artificially recharge an aquifer;
- describe the artificial recharge methods appropriate to South Africa;
- discuss the costs associated with developing an artificial recharge scheme;
- describe the management implications of operating an artificial recharge scheme.

Special attention was to be paid to the identification of suitable fractured aquifers for pilot recharge studies as large parts of the subcontinent are underlain by such aquifers.

ARTIFICIAL RECHARGE TECHNIQUES

A variety of artificial recharge systems and combinations of systems are in use. Selection of system type and design is dictated either by the purpose(s) or by the characteristics of the situation. With direct artificial recharge methods, water is conveyed to a suitable site where it is made to enter the aquifer. Indirect or induced recharge is obtained by locating the means of groundwater abstraction as close as practicable to areas of reject recharge or natural discharge. *Unconfined, primary aquifers* such as unconsolidated alluvial or dune sands are usually recharged by inducing river or lake water into the aquifer, or by infiltrating water from seepage basins or trenches into the aquifer. *Unconfined secondary aquifers* like surface calcrete and weathered-rock aquifers can also be recharged in this way. In the case of *confined primary or secondary aquifers* the recharge water needs to bypass the confining layers in order to come into contact with the most permeable part of the aquifer, therefore, seepage trenches or boreholes are needed. The creation of artificial aquifers, or sand storage dams, is an innovative way to create subsurface storage space in ephemeral river beds, particularly in hot, semiarid to arid regions.

RECHARGE WATER SOURCES: QUANTITY, QUALITY AND RELIABILITY

Recharge water can be obtained from a variety of sources, but mostly from surplus surface water, which cannot be used or reused directly for a variety of reasons, and is lost as outflows into rivers or the sea, or evaporates. Water for aquifer recharge purposes has to have a consistent high quality and a fairly predictable quantity over time. Sources considered are municipal wastewaters, storm runoff, rainfall harvesting, river flows, and water releases from dams. Municipal wastewater has a predictable quantity and quality, but it may not necessarily be suitable for artificial recharge purposes for unrestricted reuse. They invariably require significant chemical treatment before being considered of sufficiently high quality for aquifer recharge. Although some 33 000 million m³ of mean annual runoff can, on average be exploited for water supply purposes every year, the total volume of sewage effluent produced annually is only in the region of 300 million m³ (DWAF, 1986). Of the sewage effluent actually produced, 24 percent was reused (DWAF, 1986).

Storm runoff is highly variable in quantity. In comparison with rural storm runoff, urbanised catchment runoff may be up to 50 percent more in total volume, and up to 500 percent more in peak discharge. For stormwater management, e.g. a lower volume and lower peak discharges, infiltration and stormwater retention ponds, porous pavements and wetlands are recommended for catchment areas. In numerous examples worldwide stormwater is used for artificial recharge.

Due to the highly variable flow rate, runoff is usually collected in an impoundment or basin from which controlled release of water into recharge basins takes place after settling of the bulk of the suspended solids. The Omdel scheme in neighbouring Namibia operates in this way.

Urban storm runoff is usually highly variable in quality, but with the exception of industrial runoff, is on average of higher chemical quality than municipal waste water and agricultural runoff.

The aim of rainfall harvesting is to collect as much high-quality water from rainfall events as possible. Thus the objectives are to minimise loss to infiltration and evaporation as well as to minimise contamination. Hence rainfall harvesting has volume-flow characteristics similar to, if not more extreme than, urban storm runoff.

Rivers have a more consistent quantity of flow than storm runoff in their catchment areas, due partly to a base-flow contribution from groundwater/ interflow, and a wide range of rainfall-response times. Compared to most countries, South Africa has a low average annual rainfall. The climate varies from humid in the east and in the southern coastal areas to arid in the central, north-western and western areas. In the arid areas of South Africa, river flows are far more variable than those in the humid areas. In dry areas river flows are below one percent of the mean annual runoff for 50 to 70 percent of the time.

In the report the quality-related characteristics of various water sources are compared for the USA. The advantages and disadvantages of using various source waters for ground water recharge are also listed.

SOIL MATRIX CHEMISTRY AND WATER QUALITY ISSUES

Various factors influence the quality of the water when recharging an aquifer via an artificial groundwater recharge system. Infiltrating water, derived from a surface recharge facility, in moving through the unsaturated zone, may undergo substantial quality changes before reaching the aquifer. The design of the system should be such that these changes will improve the quality of the water, e.g. degradation of organics, before it reaches the saturated zone. However, the initial quality of the recharge water often remains the main characteristic determining the final water quality. In the case of direct injection into the aquifer, quality changes are usually limited to phenomena such as calcium dissolution from a carbonate aquifer.

Chemical equilibrium modelling should be used to ascertain if any undesirable reactions, e.g. precipitation, could take place in the aquifer during artificial recharge.

The quality of the recharge water is a key variable in the decision on the type of artificial recharge system to be employed. High quality, low turbidity water can be utilised successfully in any kind of recharge system. However, when the water quality needs improvement, e.g. nutrient or organic compound removal, a surface infiltration system involving soil aquifer treatment (SAT) may be indicated. Alternatively, pretreatment options may be exercised to reduce turbidity and improve quality for direct recharge. Also, aquifer storage and recovery (ASR) systems sometimes allow for a controlled degeneration in aquifer water quality where the recovered water

is to be used for restricted purposes. In such cases the turbidity must still be low, but the chemical and/or microbiological quality may be impaired. In general, the extent of post treatment of water recovered from an artificial recharge operation, will mainly depend on the intended use.

Chemical quality changes taking place in the recharge process can be simulated by mass transport modelling following successful simulation of the groundwater flow with suitable models. For most of the groundwater flow models an appropriate mass transport model exists.

HYDRAULIC FACTORS WHICH AFFECT THE POTENTIAL FOR ARTIFICIAL RECHARGE

In order to understand the processes and mechanisms involved in artificial groundwater recharge the report briefly covers unsaturated and saturated flow as related to basin infiltration and subsurface injection. In the case of basin infiltration, the success of recharge schemes is largely dependent upon the ability of the unsaturated zone to transmit water. In addition, the soil characteristics should be such that while the optimal infiltration rate is obtained, the contact time with the soil and aquifer material should also be sufficient for ensuring the desired quality improvements by sorption, degradation and other processes. The theory of soil water movement is given with emphasis on the driving force of soil water movement, the unsaturated hydraulic conductivity, the effect of layering and the quantification of infiltration rates. The implications of the infiltration theory to recharge schemes are also discussed.

The hydraulic conductivity function and its relation to the moisture content of the system is discussed followed by the mathematics of soil water flow, Darcy's law and the Richards equation. Infiltration into layered soils present special problems and the theory is briefly described. It is also important to be able to estimate the ability of the unsaturated zone to transmit water. This is described on the basis of the Delta function model and its calculation in a stepwise process.

Subsequently the aquifer's hydraulic suitability to receive recharge water is discussed in terms of its hydraulic conductivity and storage capacity. A third important factor is the aquifer's hydraulic gradient, which relates mostly to the recovery of the recharged water. The key questions are:

- Will the recharge water be able to flow into the aquifer (hydraulic conductivity)?
- Will the aquifer have sufficient space to accept the water (storage)?
- Will the water be recoverable?

CLOGGING POTENTIAL

Correctly dealing with the phenomenon of clogging of the artificial recharge system, plays a decisive role in determining the success or failure of a scheme. Clogging of the system is due to mechanical, physical, chemical and biological processes, as well as a combination of these. It can take place at the infiltration surface, in the unsaturated zone, or in the aquifer itself. In the case of injection it could block the fractures leading away from the borehole. A thorough

understanding of the processes involved and the consequent reversibility or irreversibility of the situation is needed in order to be able to manage the clogging phenomenon.

Clogging or plugging is defined as a significant increase in injection head for a constant injection rate, and is due to an increased resistance to flow near the well or borehole. Clogging also negatively impacts on recovery of recharged water, since it increases drawdown during abstraction.

The processes that are primarily responsible for clogging are: air entrapment and gas binding; deposition of suspended solids from the recharge water; biological growth of bacteria on or within the infiltration media and the surrounding formation; and chemical reactions between recharge water, groundwater and the aquifer material.

Basin recharge remains the most attractive option if the water has any (physical) clogging potential as restoration of the infiltration surface is a relatively simple task provided compaction of the filtration medium by heavy surface scraping equipment is prevented. Different recharge systems and the associated potential clogging phenomena are discussed in the report with reference to preventive and remedial measures where applicable.

MODELLING ARTIFICIAL GROUNDWATER RECHARGE

Mathematical models can be used to quantify artificial recharge processes, thereby optimizing recharge rates. There are a wide variety of mathematical models which are designed to represent actual physical processes, including the flow mechanisms. Once these mechanisms have been defined, calculations can be carried out to estimate artificial recharge. The ultimate goal of any such process is usually to convey the maximum volume of water to groundwater storage as efficiently as possible.

MODFLOW can be used to simulate the borehole injection method or the surface water/river option with a coefficient for leakage. This option is generally available in most three dimensional groundwater modelling packages for example *SUTRA*, a saturated / unsaturated transport numerical modelling package. Examples of packages that calculate infiltration from seepage basins are *FASTSEEP* and *SEEP/W* which are finite element programmes developed specifically for simulating seepage from spreading basins. *SEEP/W* is the most popular of the software mentioned above as it can simulate both infiltration due to spreading basins and recharge by means of injection boreholes for saturated and unsaturated conditions.

Mass transport models are linked to individual hydraulic models. Most of these models are capable of modelling advection in complex steady-state and transient flow fields, anisotropic dispersion, first order decay and production reactions, and linear and nonlinear sorption. Examples of commercially available software, are *MT3D*, which is linked to *MODFLOW* and *CTRAN/W*, which is linked to *SEEP/W*.

Models are useful tools for the management of artificial recharge. However it is important to note that there is no ideal model, and that none of the models mentioned above were developed to simulate the fractured rock conditions which are common in South Africa. Therefore, the most

suitable one has to be selected for the particular investigation.

RECOVERY EFFICIENCY

Based on the literature, the efficiency of groundwater recovery in artificial groundwater recharge systems seldom seems to be an issue. Where the artificial recharge scheme affects only a part of the aquifer which is centrally located and hydraulically up-gradient of the production wellfield, losses of recharged water from the aquifer are, in general, regarded as being insignificant. In the case of the Atlantis aquifer for example, only a general water balance was carried out with respect to the artificial recharge facility, accounting for the volume of water actually added to the groundwater reserve. On the other hand, subsurface injection of water into a fractured aquifer may lead to losses of water from the aquifer via fractures or springs, sometimes issuing on neighbouring properties. Such losses need to be taken into account when optimising the system.

Where water is to be obtained from a surface reservoir, the potential water losses from artificial recharge should be considered in relation to evaporation losses. Because of South Africa's high evaporation rates, water should be stored underground where possible, even if not all of it can be retrieved.

In the case of aquifer storage and recovery (ASR) systems recovery efficiency is found to be dealt with very specifically in the literature. Recovery efficiencies on ASR schemes are most often defined in terms of water quality. It, therefore, requires a significant difference in composition between the injected water and the native aquifer water to be applied successfully. ASR recovery efficiency was *defined* as follows: *the percentage of water volume stored that is subsequently recovered while meeting a target water quality criterion*. The water quality criterion is typically the TDS, EC or chloride concentration. The limit is generally set at a higher salinity than that of the injected water. This reflects the typical current ASR practice of injecting fresh waste water into saline or brackish coastal aquifers. This means that what is being abstracted is not only the injected water but a mixture of injected and native water until the proportion of native saline water becomes unacceptable.

Typical ASR recovery efficiencies are found to be up to 70 percent, however, it has been stated that most schemes can be developed to 100%. Exceptions are very transmissive, highly saline aquifers which reached 70 to 80 percent. These efficiencies should be compared to the efficiency of surface storage facilities, such as impoundments, which may be significantly less than 50 percent.

Many factors control recovery efficiency, several of which are interdependent. These mainly relate to aquifer characteristics of the receiving zone around the borehole, such as: aquifer configuration (dip), hydrodynamic dispersion, permeability (degree of permeability and relationship between primary and secondary permeability), hydraulic gradient, concentration gradient between the injected water and the native aquifer water, heterogeneity, anisotropy, and aquifer thickness. Operational characteristics that influence recovery efficiency include inter alia: borehole siting, borehole design (screen position and total screen length), rates of injection and abstraction, volume injected, storage time, the target limit, number of successive cycles, etc.

SOCIO-ECONOMIC AND LEGAL ISSUES

The cost effectiveness of an artificial recharge scheme is usually a key consideration. Often water supply facilities are expanded to overcome the shortage experienced during peak demand periods. Conjunctive use of artificially recharged water is an ideal solution for this purpose as no additional facilities will be required to recover the stored water.

Surface recharge, if feasible, is usually the most cost-effective option of getting water into the aquifer. If surface recharge is not feasible then ASR can also achieve this objective at a marginally higher unit cost. ASR operations are invariably more cost-effective when compared with conventional water supply alternatives involving the development of new water sources.

In the case of Kenhardt calculations showed that an artificial recharge scheme would be the most cost-effective method of augmenting the water supplies, and it would be the method of choice provided surface water was available for subsurface storage.

Artificial recharge schemes commonly involve surface or waste water capture, treatment, pumping, distribution and water quality monitoring. In order for these processes to be efficient, careful planning and management is needed. Maximum benefit from an artificial recharge scheme usually involves integrating the scheme into the planning and management of the entire groundwater basin. This includes optimising both surface and groundwater resources, and their storage capacities.

Artificial recharge is one of the developments in water resource management that is challenging the legal fraternity to respond to advances in the water resources sector. Issues, that need to be addressed within a legal framework, are for example policies for protecting public health, safety, property and ecological interests; ownership of water proposed for recharge; laws for ensuring that the water quality and storage capacity of an aquifer does not deteriorate as a result of artificial recharge operations; and other matters.

Public opinion about artificial recharge could be a controlling factor in the successful implementation of this technology. Public attitudes cannot be disregarded because the public ultimately bears the burden of the costs of such operations. Concern regarding artificial recharge centres primarily around the perceived quality of the water and whether it might serve to transmit pathogens, viruses or harmful trace chemicals. Public perception about reuse of stormwater or surface water is not likely to be a problem. However problems may arise when waste water is to be used for artificial recharge purposes. A public participation strategy for overcoming problems associated with public perceptions has to form part of the planning of the recharge scheme.

GUIDELINES FOR ESTABLISHING ARTIFICIAL RECHARGE SCHEMES

Establishing an artificial recharge scheme needs a phased and multi-disciplinary approach. In the report two examples of planning processes are described. The first example applies to basin recharge schemes and the second one to borehole injection schemes. The level of effort and associated financial investment in obtaining data in each phase should be related to the degree

of risk, both technical and nontechnical. If land availability and hydrogeology are favourable, basin recharge as opposed to borehole injection is usually the most cost-effective method.

In the planning process for establishing a basin recharge scheme, eight phases are identified. These commence with a precursory site evaluation, followed by detailed site investigations, recharge water evaluation, mathematical modelling, pilot test studies, and other tasks, before finalising construction plans and developing operation, maintenance and performance evaluation plans. These could also be grouped differently in three phases, as suggested in the planning of an aquifer storage and recovery scheme (ASR), or borehole injection system. The main difference is, however, that full scale injection boreholes already need to be installed for the testing phase. Once the testing is completed, the facility then has to be expanded to full design capacity.

ARTIFICIAL RECHARGE EXPERIENCES ELSEWHERE IN THE WORLD

Artificial recharge is practised in many countries around the world, and in some countries, such schemes were introduced over a century ago. A survey of artificial recharge practice in fourteen European countries concluded that:

- ▶ artificial recharge is important in ten of the fourteen countries surveyed;
- ▶ the use of artificial recharge is increasing within the countries in which it is operating;
- ▶ artificial recharge schemes are operating successfully, and in some cases have been in use for nearly a century;
- ▶ in addition to increasing water supplies, artificial recharge often has environmental and water quality aims and benefits.

Examples of artificial recharge schemes are listed in the report. Some show how artificial recharge forms part of bulk water supply schemes, and the others show that it is possible to recharge relatively low permeability aquifers as well.

THE POTENTIAL FOR ARTIFICIAL RECHARGE IN SOUTH AFRICA

Artificial recharge to South African aquifers is not a new concept. Scattered throughout the country are small earth dams which farmers have built to augment their borehole supplies. In all but one case, so it seems, the impact of these artificial recharge schemes on groundwater has not been established. The one study where records were kept was in the Soutpansberg District of the Northern Province, but unfortunately the information obtained is insufficient to establish the true effect of the recharge dams. A substantial data base does however exist in the case of the artificial recharge scheme at the town of Atlantis in the Western Cape. In this case, the effectiveness of the scheme has been demonstrated. Two large recharge basins feed the primary, dune-sand aquifer, supplying in the region of 2×10^6 m³/a of recharge water. The source is storm water runoff and treated domestic wastewater.

The two main hydrological factors which determine the potential for artificial recharge in South Africa are the availability of raw water and the ability of the aquifer to physically receive surplus

water. In relation to the water source, the reliability of the raw water and its quality is of prime concern. Possible water sources include ephemeral and perennial rivers, dams, municipal waste water and storm runoff. In view of the high evaporation rates it can be cost effective to store water underground rather than at the surface. If rivers or dams are to be used for artificial recharge, it may be necessary to reduce the turbidity of the water in order to prevent clogging.

In relation to the aquifer acceptance potential, the permeability of the aquifer (and the soil horizons above the aquifer if infiltration methods are applicable) and the storage potential of the aquifer are the key factors which will determine the suitability of an aquifer for artificial recharge. Although aquifers which have high storativity values and which are highly transmissive are most suitable for receiving additional water, aquifers with low storativities and low permeabilities like the hard rock aquifers commonly found in South Africa, can be artificially recharged.

South African aquifers vary considerably from those with high a permeability and storativity like the primary aquifers at Atlantis and the Cape Flats in the Western Cape and some of the dolomitic aquifers in the Northwest Province, to those with low permeability and storativity like most of the hard rock aquifers which are found throughout the country. Whereas aquifers with high permeabilities and storativities are most suitable for receiving recharge water, aquifers with limited permeability and storativity can also be artificially recharged.

Spreading basins should be considered as the first choice as long as the permeability of the soil horizons and aquifer material is sufficient for allowing rapid infiltration and percolation. If not, trenches which cut through the impermeable layers, or boreholes which penetrate the most permeable parts of the aquifer, should be considered as the more appropriate methods.

South African environments potentially suitable for artificial recharge are listed, as well as the key prerequisites for such applications. Existing Southern African artificial recharge schemes, both pilot and full scale facilities, are also summarised in the report.

PILOT ARTIFICIAL RECHARGE SCHEMES

It was an objective of this study to conceptually design one or more pilot artificial recharge schemes which could serve to demonstrate the applicability of artificial recharge in South Africa. In view of the scarcity of schemes utilizing a fractured aquifer, it was considered a priority to have at least one scheme in such a geohydrological setting. A further criterion for the selection of potential test sites was that the community or town was needing to obtain additional water supplies. This was intended to ensure the interest of the community in developing, maintaining and monitoring of the recharge facility. The availability of a potentially suitable aquifer and a source of recharge water were basic requisites for considering any site.

A large number of pilot study sites were proposed, but for only ten of these enough information was available for evaluation. Brief information is provided in the report on each of these. Of these, six have tentatively been identified for further study. These are Kenhardt, Calvinia, Windhoek, Williston, Karkams, and a farm near Rustenburg. For the first three more detailed information is provided. In all cases, further investigations, including field studies are required

in order to establish whether they are indeed suitable for artificial recharge. Four of the sites are located in the arid Northern Cape and one in the Northwest Province. Further suitable study sites may be considered, but selection will also depend on factors such as the local infrastructure, as well as participation and support by the local authority.

Kenhardt

Kenhardt is likely to face severe water shortages in the near future. Domestic water is currently obtained from the Driekop wellfield which has sufficient capacity to supply the current average winter demand, but not the peak summer demand. The severity of this problem is clearly shown by a dramatic drop in the water table since 1991. There are a number of water sources which could be used to meet the town's additional water needs. The most economical solution would probably be by implementing an artificial recharge scheme using an existing, but largely unused dam, linked to the existing production wellfield.

A pilot artificial recharge study is proposed for deriving design and operational criteria. At this stage it appears as if the Rooiberg Dam may be a suitable water source. Because of the dam's unreliability, it is not used for domestic or farming purposes, however, this does not mean that it cannot be used for artificial recharge. Its net capacity in 1983 was calculated as $3.7 \times 10^6 \text{ m}^3$, and the Department of Water Affairs and Forestry's records show that it receives water most years. Should this site prove suitable for artificial recharge, then water could be gravitated down the existing furrow, filtered and then pumped to the wellfield. The filtered water could also be pumped directly to the town's treatment and distribution systems.

The method of artificial recharge is likely to be either recharge basins of about 20 m x 20 m, or subsurface trenches filled with coarse sand and slotted casing. All of these facilities would be constructed up gradient of the production boreholes.

If an average electrical conductivity of 68 mS/m could be maintained, the water quality in the Driekop aquifer would improve.

Calvinia

Calvinia relies on the Karee Dam, which is the main domestic water source, and on groundwater for its water supply. Recently a new wellfield was developed in order to alleviate the water reliability problems. Although this will improve the situation for the town, a reserve sub-surface storage compartment could be added to the water supply system, at little extra cost. This would help provide a longer term solution to water supply problems.

The aquifer to be recharged forms a sub-surface reservoir, consisting of a highly porous and permeable cylindrical brecciated plug. The plug is believed to be well brecciated, with open spaces, to about 240 metres below ground level. It is located about 12 km east of Calvinia, near the newly developed wellfield. A total of four boreholes have been drilled into the plug and two outside the plug. Two of those drilled into the plug will be used for injecting the recharge water.

Surplus treated water from the Karee Dam, when available, and groundwater from other aquifers could be used for storage within the sub-surface reservoir. The groundwater can be obtained

from new production boreholes and an existing production borehole, which are not being used continuously for town water supply purposes. When groundwater from these aquifers is not needed, the water could be pumped into the subsurface reservoir for use at a later stage.

As borehole water may serve as part of the source for the recharge water, chemical equilibrium modelling was carried out, using the modelling package MINTEQA. The purpose was to determine whether mixing of aquifer and recharge water may lead to chemical precipitation, which, in turn, may cause blockages in the aquifer. The equilibrium modelling showed that in all mixing scenarios the solutions would be in equilibrium with the potential solid phases. Only ferric solids would show a degree of super saturation and thus have a possible precipitation potential. It depends, however, on the oxidation state of the iron in solution. If it is assumed that iron occurs exclusively as Fe^{2+} , i.e. in the reduced form, no precipitation potential exists.

An implementation plan has been compiled and was submitted to the Calvinia Municipality.

Windhoek

Windhoek receives most of its water supply from surface water resources, but a vital contribution is made by groundwater and water reclamation. Groundwater from the quartzite beds of the Windhoek aquifer provide approximately ten per cent of the total supply, but can yield substantially more in periods of drought. Temporary over-exploitation in such periods can be balanced by the subsurface storage of surplus surface water when available, preventing evaporation losses, and providing additional reserves. A preliminary test has already indicated that artificial recharge could be feasible and a longer term injection run on a larger scale is planned to determine the full scale feasibility and response of the aquifer.

Williston

This artificial recharge scheme would involve transferring water from one groundwater compartment to another groundwater compartment. The second compartment is the one which supplies the town of Williston.

The water level in the main production borehole of the town (G33076) has declined steadily over the years. This borehole was shown to be hydraulically linked to another borehole (G39975) some 4 km away. This is due to a horizontal fracture pattern which resulted from dolerite sill intrusions into the Karoo sedimentary rocks. The boreholes both intersect the main horizontal fracture at approximately 70 m below ground level. Thus water injected into the distant borehole (G39975) would find its way to the main town production borehole. Another borehole (G39976) exists 50 m from the distant borehole but they seem to be separated by an impermeable boundary. Water from borehole G39976 could, therefore, be injected into G39975 for abstraction at G33076 near the town. The injection rate is estimated to be in the order of 6 L/s.

Karkams

In the case of Karkams, a spring in the granitic terrain was dammed and piped into a production borehole via a sand filter. The spring flows for a number of months each year and the water is largely lost down the river. The system was intended to intercept this water and to store it in the

subsurface to augment the existing reserves. Flood waters damaged the system and it is presently in disrepair. The success of the system could not be determined as no water level measurements or other readings have been taken.

It is possible to redesign the facility and to set up a system with a water meter and water level recorder in order to establish the success of the recharge operation. This could serve as an example for using other non-perennial springs.

Rustenburg (farm)

This facility is situated in the Rustenburg area, in igneous rocks at the base of the Bushveld Igneous Complex. A farmer gravitates clear spring water during the summer rainfall period into 4 or 5 boreholes. In winter, the water is abstracted from the boreholes for irrigation purposes. The boreholes yield (and receive from the springs) on average about 1 L/s, and yield approximately the same amount of water that is injected. The scheme has been operational for three years.

CONCLUSIONS

The project set out to assess the feasibility of using artificial recharge as a means of enhancing groundwater resources in South Africa. South African hydrogeological environments which are suitable for artificial recharge were identified, and four possible sites are presented where artificial recharge could be implemented. These sites include the aquifers which support the towns of Kenhardt, Calvinia and Williston, and the city of Windhoek in Namibia.

Success factors which affect the potential for artificial recharge are:

- the water source: quantity, quality and reliability
- the hydraulic characteristics of the aquifer;
- the quality of the recharge water;
- clogging of the recharge basins, trenches or boreholes;
- groundwater recovery;
- economic factors;
- management requirements.

The main conclusions drawn for each of these success factors are set out in the report.

Both primary and secondary aquifers are suitable for artificial recharge in South Africa. Suitable primary aquifers include sandy alluvium (which may feed underlying weathered and fractured hard-rock aquifers), and coastal sand aquifers. Examples of alluvial aquifers include the Mogol River, near Ellisras and the Sand River near Pietersburg in the Northern Province. Suitable coastal sand aquifers include those in the Cape Flats and Atlantis in the Western Cape Province, and those which support coastal towns which experience high peak demands in summer.

Secondary aquifers with high permeability and storativity are most suitable for receiving additional recharge water. Such aquifers include the dolomitic aquifers in the Northwest

Province, and the intensely weathered and fractured hard-rock aquifers which are found in various parts of the country. In many parts of the country, however, only low permeability hard-rock aquifers exist. Artificial recharge may still be an appropriate method to enhance limited natural groundwater resources. In such areas, average borehole yields may only be in the region of 1 L/s. If these aquifers were artificially recharged at 1L/s from a number of recharge points, it could make a considerable difference to the exploitable groundwater resource. The geohydrological environments which may be suitable for artificial recharge are listed and the following types of systems may be appropriate:

- induced recharge schemes (eg. river bank filtration)
- infiltration schemes (eg. infiltration basins, land flooding and subsurface trenches)
- borehole injection
- artificial aquifers (sand storage dams)

The overall conclusion is that artificial groundwater recharge is an internationally recognised method for managing water resources. Its main uses are for conserving water for future use, for improving water quality and for averting saline water intrusion into over-exploited aquifers. At present there are only a few fully operational artificial recharge schemes in Southern Africa, but sufficient motivation exists to support the continuation of pilot scale artificial recharge studies in secondary aquifers. Sites which have been identified, and which seem suitable with respect to the success factors mentioned above, include Kenhardt, Calvinia, Williston and Windhoek.

RECOMMENDATIONS

It is recommended to:

- test artificial recharge in secondary, fractured rock aquifers, since in most of South Africa these hydrogeological conditions occur. Existing schemes throughout the world are mostly in highly permeable, porous aquifers, thus lessons learnt are not always applicable to South African conditions. Pilot studies should include borehole injection schemes.
- study borehole clogging using surface water;
- carry out equilibrium modelling to predict the outcome of chemical reactions between recharged water, native groundwater and the host rock;
- study factors which affect recovery efficiency during the implementation of pilot artificial recharge schemes in secondary aquifers; and
- establish the fate of microorganisms when using water of impaired quality for recharge of fractured aquifers.

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Chapter 1

INTRODUCTION

Artificial recharge is a practical, cost-effective and environmentally acceptable water management alternative for rural communities, water supply authorities and farmers. The aim of artificial recharge for water supply purposes is to rapidly replenish aquifers with water that would otherwise be lost through evaporation and streamflows. A tremendous amount of water is lost through evaporation alone, from the thousands of dams scattered throughout the country. The challenge for water supply planners, as part of the Department of Water Affairs and Forestry's campaign for water conservation, is to minimise this waste, and to maximise sub-surface storage.

The best known South African example, is the artificial recharge facility in the town of Atlantis, near Cape Town. Approximately 20 percent of the town's water supplies are obtained via artificial recharge of stormwater runoff and treated wastewater. The scheme has been successfully in operation for more than 15 years. Many of the other existing artificial recharge schemes in the country, have either been poorly designed, are not operating efficiently, or they have not been monitored, and thus their effectiveness has not been established.

If artificial recharge is to become a recognised method for ensuring reliable water supplies, then geohydrologists and water resource planners will need to become familiar with the factors which affect the suitability of a given site for artificial recharge. This study presents the issues which need to be considered when planning an artificial recharge scheme.

1.1 RESEARCH OBJECTIVES

The aim of this project was to assess the feasibility of using artificial recharge technologies in South Africa. In order to meet the aim, the following objectives were identified:

- those artificial recharge techniques appropriate to South African conditions and needs must be determined;
- the assessment must include concepts such as conjunctive use, groundwater catchment management and groundwater dams (sub-surface and sand-storage dams);
- the appropriate technologies must be linked to areas/regions/hydrogeological settings;
- basic guidelines must be developed for establishing artificial recharge schemes; and
- a pilot scheme must be designed to test the concept (field site identification, and pilot scheme and test programme designed).

1.2 RESEARCH METHOD

The project consisted of a desk study which drew on: (a) international experience with respect to topics like the hydrological factors which affect the potential for artificial recharge and guidelines for establishing artificial recharge schemes; and (b) local experience with respect to identifying possible pilot artificial recharge schemes.

Besides identifying pilot artificial recharge schemes, the key issues identified, and the tasks which were to form the basis of the research plan, included:

- identify potential artificial recharge users;
- identify the factors which affect artificial recharge potential;
- describe the hydrogeological requirements for artificial recharge;
- describe the water resources needed in order to artificially recharge an aquifer;
- describe the artificial recharge methods appropriate to South Africa;
- discuss the costs associated with developing an artificial recharge scheme;
- describe the management implications of operating an artificial recharge scheme.

The research was carried out by means of a detailed literature study and discussions with a number of people who have had, or may have in future some association with an artificial recharge project. These people (listed in the acknowledgements) included geohydrologists from DWAF, private consulting companies and research institutions, and people who are responsible for the delivery of water, like municipal engineers. Besides sharing their experience, many expressed a need to know how to go about assessing whether a particular aquifer would be suitable for receiving artificially recharged water. For this reason, a substantial proportion of this report is devoted to: describing the hydrological factors which affect the potential for artificial recharge; and a step by step process which can be followed when assessing the suitability of a particular site for artificial recharge.

In order to assess the potential pilot artificial recharge schemes, many of the proposed sites were visited, and in the case of Kenhardt and Calvinia, field tests including particle size analyses and borehole test pumping were conducted (Chapter 7).

1.3 REPORT LAYOUT

This report is aimed at two main audiences: water resource planners and geohydrologists. It contains general information for those who need to be aware of the issues which affect the success and failures of artificial recharge schemes, and it contains specialist information for the geohydrologist who needs to design an efficient artificial recharge scheme. The report reflects current international experience, and where possible, it relates hydrological and non-hydrological factors to South African conditions.

Chapter 2 contains a description of artificial recharge methods and the key tasks which are

necessary for efficient management. Chapter 3 is aimed at geohydrologists - it describes the hydrological factors which affect the potential for artificial recharge in South Africa. These factors include:

- The quantity, reliability and quality of the water to be used for recharge;
- The suitability of the unsaturated zone for transmitting recharge water to the aquifer;
- The suitability of the aquifer to receive additional recharge. This relates to the aquifer's permeability and storage capacity;
- The change in water quality as the recharge water mixes with the native groundwater;
- The factors which affect the reduction in infiltration or injection efficiency (clogging);
- How the success of an artificial recharge scheme is measured in terms of its recovery efficiency;
- The role of modelling in the design of an artificial recharge facility.

Chapter 4 discusses socio-economic factors which need be considered when planning an artificial recharge scheme. Chapter 5 describes the steps to take when planning an artificial recharge scheme. This chapter, which is aimed at both water resource planners and geohydrologists, contains the basic guidelines which need to be considered when establishing the suitability of a particular site for artificial recharge.

Chapter 6 gives examples of artificial recharge schemes from around the world. Case studies have been selected where aquifers have similar characteristics to most South African aquifers. That is, hard rock, secondary aquifers which have relatively low permeabilities. This chapter also lists existing artificial recharge schemes in Southern Africa, and it describes typical South African geohydrological environments which may be suitable for artificial recharge. Chapter 7 describes possible pilot artificial recharge sites, and the concluding chapter, Chapter 8, summarises some of the main points captured in this report.

Chapter 2

ARTIFICIAL RECHARGE METHODS

A variety of artificial recharge systems and combinations of systems are in use around the world. Selection of system type and design is dictated either by the purpose(s) or by the characteristics of the situation. Purposes could be the augmentation of supplies, storage of water for peak (seasonal) demands, or the treatment of drinking water or wastewater. The methods can be classified into two main groups, namely indirect and direct artificial recharge. Indirect or induced recharge is obtained by locating the means of groundwater abstraction as close as practicable to areas of reject recharge or natural discharge. With direct artificial recharge methods, water is conveyed to a suitable site where it is made to enter the aquifer.

Unconfined, primary aquifers such as unconsolidated alluvial or dune sands are usually recharged by inducing river or lake water into the aquifer, or by infiltrating water from seepage basins or trenches into the aquifer. *Unconfined secondary aquifers* like surface calcretes and weathered-rock aquifers can also be artificially recharged by means of infiltration basins and trenches. In the case of *confined primary or secondary aquifers* the recharge water needs to bypass the confining layers in order to come into contact with the most permeable part of the aquifer. In these cases, seepage trenches or boreholes are needed. Seepage trenches may be appropriate if the confining layer is a thin, near surface horizon. Boreholes are required if the confining layer is near surface but more than about 5 m thick, or if the confining layer is deep seated. These methods are described below.

2.1 INDUCED RECHARGE METHODS

The most common method of induced recharge, commonly called (river) bank filtration, consists of setting a gallery or a line of wells at a short distance (say 50 m) parallel to a river or lake (Figure 2.1). When pumping commences, water abstracted from the wells initially comes from natural groundwater. As pumping continues the water table near the shore line will drop below the water level in the river/lake, and surface water from the river/lake will be induced to enter the aquifer and flow to the wells. While this method is applied throughout the world, there is limited local potential for induced recharge because many South African rivers are non perennial and have poor river bed permeabilities.

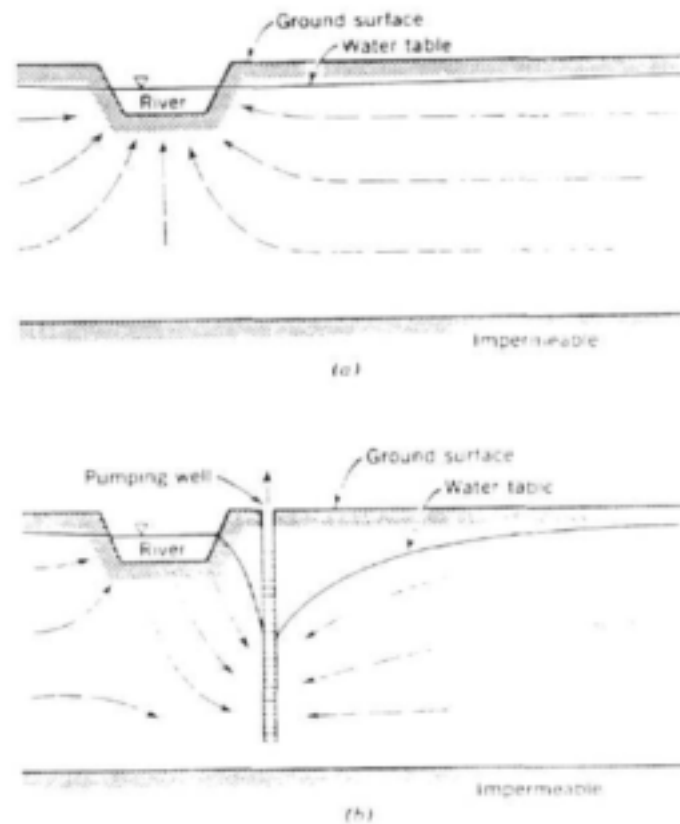


Figure 2.1 Induced recharge resulting from a well near a river. (a) Natural flow pattern. (b) Flow pattern with pumping well (after Todd, 1980)

The amount of water induced into the aquifer depends on:

- the rate of pumping;
- permeability;
- type of well;
- distance from surface water and wells;
- natural groundwater movement.

Huisman and Olsthoorn (1983) state that the success of this approach depends primarily on the permeability of the river bank or river bed. In South Africa, the reliability of the water source would be of equal concern. A significant benefit from induced recharge is that it can provide water which is free of organic matter and pathogenic bacteria (Todd, 1980).

No urban bulk water supply schemes of this nature are in use in Southern Africa. The method has however proved effective in other parts of the world where unconsolidated formations of permeable sand and gravel are hydraulically connected between the surface water source and the aquifer. Many cities bordering rivers in the Mississippi River basin have developed wellfields along river banks because of the assured supply of high quality water (Todd, 1980). One of the

more well known schemes is that designed by G. Thiem, the "father" of groundwater. The original system, which supplied the city of Prague, consisted of 684 shallow wells in Quaternary alluvial sediments stretching 22 km along the Jizera River. It was put into operation in 1914 and gave 800 - 1 100 l/s. It has since been upgraded to include infiltration basins and deeper boreholes, and now gives about 10 000 l/s (Knezek and Kubala, 1994).

Management requirements for river bank filtration

Although this is a simple form of artificial recharge, the system can become inefficient and poor quality water can be abstracted if it is not managed properly. The key management tasks include:

- water quality monitoring;
- water treatment;
- scraping of river banks and beds during periods of low flow in order to remove clogging clays, silts and organic matter.

2.2 DIRECT RECHARGE

Direct artificial recharge takes place when water is conveyed to a suitable site where it is made to enter the aquifer. The main advantages of direct recharge over induced recharge are:

- the water source and aquifer can be far apart;
- before entering the aquifer the raw water can be treated;
- the system can be designed so that clogging is minimal and clogging material can be removed;
- water of unacceptable quality can be left out of the recharge system.

Three groups of direct recharge methods can be recognised. They depend on the depth to the aquifer and the existence of impermeable or confining soil/rock layers:

- i. The aquifer extends to or near to ground surface and all soil layers are permeable (or impermeable, near-surface layers can be scraped away). Here, surface infiltration systems like spreading basins may be applied. These are usually the cheapest systems.
- ii. The aquifer is situated at a moderate depth below ground surface and/or impermeable soil layers are too deep to be scraped away. Pits and subsurface seepage trenches can be used. They are expensive and can have small unit capacity.
- iii. Aquifer is deep seated and/or is confined. In such circumstances, injector wells are required. The most common problem associated with this technique is with clogging, where the ability of the aquifer to receive water from the borehole is reduced as a result of suspended matter clogging up the rock openings adjacent to the borehole.

2.2.1 Surface infiltration systems

Recharge by surface infiltration methods, whereby water moves from the land surface to the aquifer by infiltration and percolation through the soil matrix, is practised throughout the world. Its ease of use, robustness towards water quality, and relatively low cost makes it an attractive method. Techniques include surface flooding, ridge and furrow systems, stream channel modifications, and infiltration basins.

The amount of water entering the aquifer depends on three factors (Figure 2.2):

- the infiltration rate;
- the percolation rate;
- the capacity for horizontal water movement.

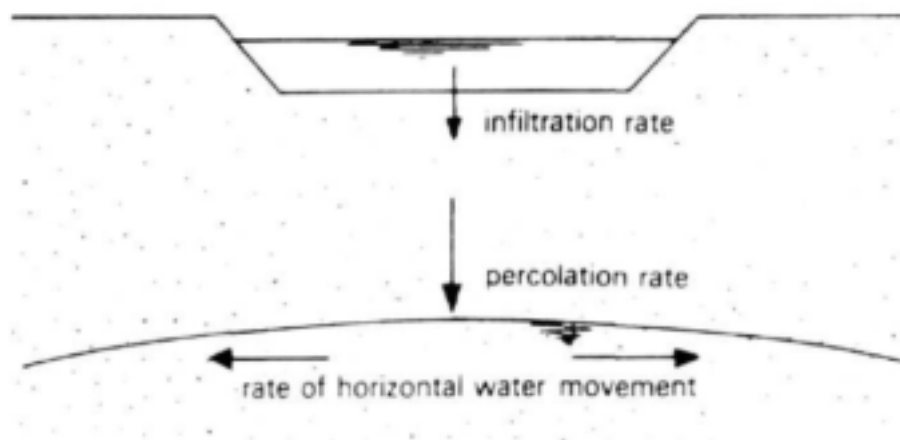


Figure 2.2 Direct recharge by spreading (after Huisman, Olsthoorn, 1983)

The infiltration rate: This is also called the entry, intake or acceptance rate, and is the rate at which water is picked up by the soil. When artificial recharge commences, the infiltration rate is equal to the percolation rate (described below). At the point of entry into the aquifer, clogging occurs by the deposition of particles carried in suspension or in solution, by algal growth, colloidal spreading and soil dispersion, microbial activity, etc. The entrance resistance increases as soil pore openings become sealed. With time the entry velocity is only a fraction of its original value. Figure 2.3 shows this relationship, and includes the planned duration of each recharge run and the probable production entry rate.

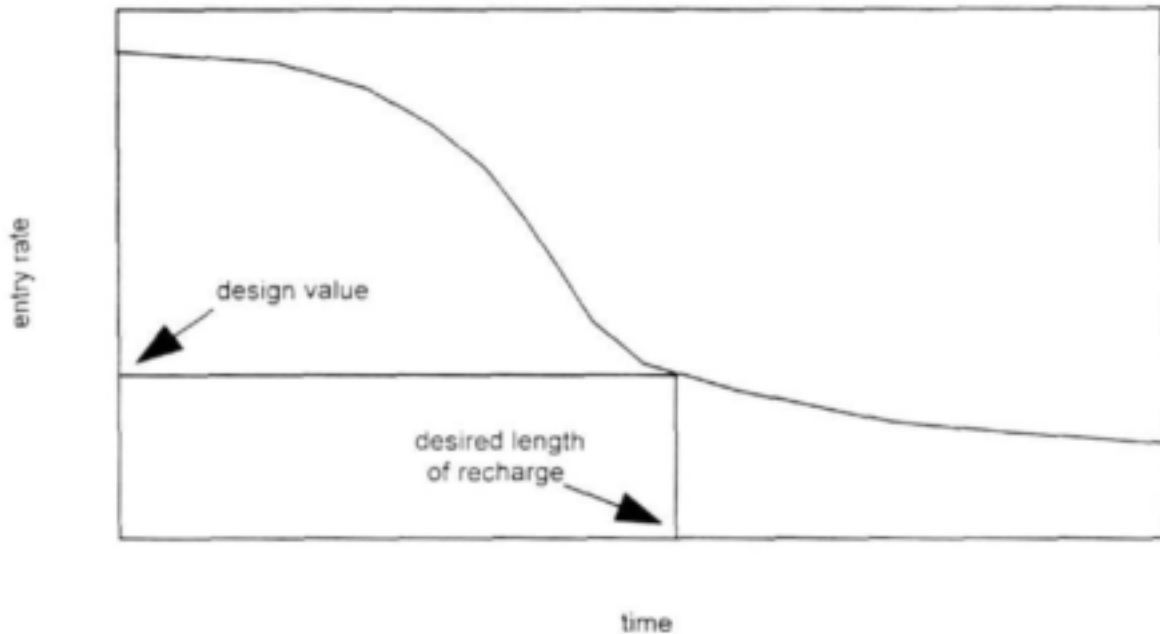


Figure 2.3 Decrease of entry value with time

The percolation rate: This describes the rate at which water is able to move downward through the soil, and it is dependant on the coefficient of permeability in the vertical direction.

The capacity for horizontal water movement: This depends on aquifer transmissivity and on the flow pattern.

Surface infiltration systems require the availability of:

- adequate land with permeable soils;
- vadose zones without restricting layers that produce excessive perched water mounds;
- unconfined aquifers of sufficient transmissivity to prevent undue rises of groundwater mounds;
- vadose zones and aquifers free from undesirable chemicals.

Some commonly used methods include:

Infiltration Basins: Infiltration basins are the most widely used method of groundwater recharge. Basins afford high loading rates and relatively low maintenance and land requirements. Basins consist of bermed, flat-bottomed areas of varying sizes. Long, narrow basins built along topographic contours have been used effectively.

Infiltration basins require permeable soil for high hydraulic loading rates, yet the soil must be fine enough to provide sufficient soil surfaces for biochemical and microbiological reactions, which provide additional treatment to the water. Some of the best soils are in the sandy loam, loamy

sand, and fine sand range. The hydraulic loading rate is preliminarily estimated by soil studies, but final evaluation is done by operating in situ test pits or ponds. Hydraulic loading rates for rapid infiltration basins vary from 20 to 150 m/y (\pm 50 - 400 mm/d), but are usually less than 90 m/y (Bouwer, 1985).

Flooding: Water is spread over a large, gently sloped area (1 to 3 percent gradient). Ditches and berms may enclose the flooding area. Advantages are low capital and O&M costs. Disadvantages are large areal requirements, evaporation losses, and clogging.

Ridge and furrow: Water is placed in narrow, flat-bottomed ditches. Ridge and furrows are especially adaptable to sloping land, but only a small percentage of the land surface is available for infiltration.

Stream channel modifications: Berms are constructed in stream channels to retard the downstream movement of the surface water and, thus increase infiltration into the underground. This method is used mainly in ephemeral or shallow rivers and streams, where machinery can enter the stream beds, when there is little or no flow, to construct the berms and prepare the ground surface for recharge. Disadvantages may include a frequent need for replacement due to washouts and possible legal restrictions related to such construction practices.

Management requirements for surface infiltration systems

Management techniques are site specific and vary accordingly, but some common principles are practised in most systems. A wetting and drying cycle with periodic cleaning of the bottom is used to reduce clogging by accumulated suspended material. Microbial populations are maintained in the subsurface near the recharge facility to break down organic matter, and to promote nitrification and denitrification processes for nitrogen removal. The key management tasks include:

- maintaining acceptable infiltration rates by drying and scraping;
- maintaining the quality of the recharge water (prior to entering the recharge facility), for example by settling and filtration. If slow sand filters are used to pre-treat the recharge water, then the sands and gravels used in the filters need to be cleaned and topped up from time to time.

2.2.2 Subsurface infiltration systems

Pits: Pits or large diameter wells are excavated through low permeable soil horizons (Figure 2.4). Even abandoned excavations have been used as artificial recharge facilities. An advantage of these schemes is that the steep sides provide for high silt tolerance, since silt usually settles to the bottom of the pit, leaving the walls relatively unclogged.

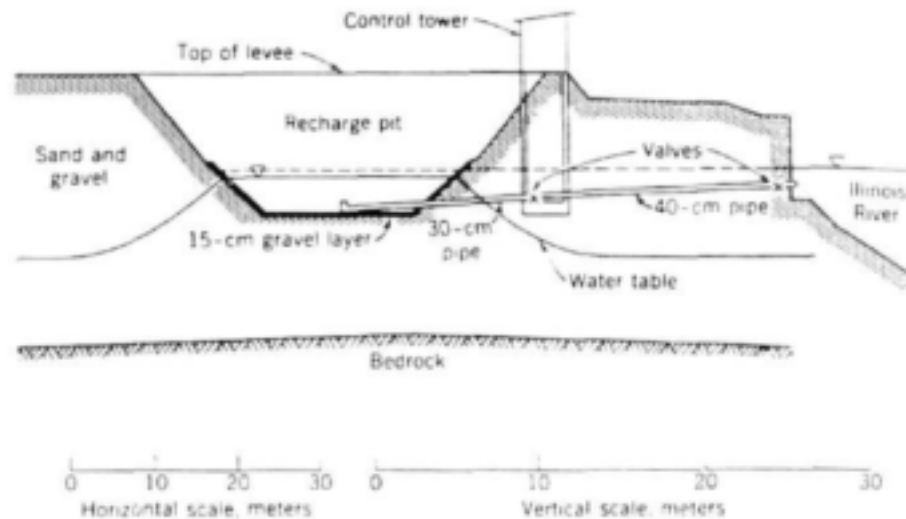


Figure 2.4 Cross section of a recharge pit at Peoria, Illinois, USA (after Suter and Harneson, 1960)

Seepage trenches: Seepage trenches are built to cut through impervious topsoil or subsoil layers; or to maximise the sidewall surface area and minimise the bottom surface area in order to facilitate horizontal movement of recharge water.

A particular design is described Wolters and Hantke (Hantke & Schlegel, 1995). It consists of a trench that is one metre wide and several metres deep (Figure 2.5). It could be up to 100 m long and is filled with coarse sand with a covering. It cuts through (shallow) impermeable layers and the covering prevents algal growth. According to the designers it combines easy regeneration, durability, and high infiltration rates. This method seems to be gaining popularity in Germany. In 1983 there were none of this type listed amongst 54 recharge installations in that country, however, in 1992 there were six amongst the 63 installations listed. The technique has also been combined with injection boreholes where the boreholes extend through the bottom of the trench. The idea is to extend the advantages of the trench to those cases where the confining layers are too deep and can only be pierced by boreholes.

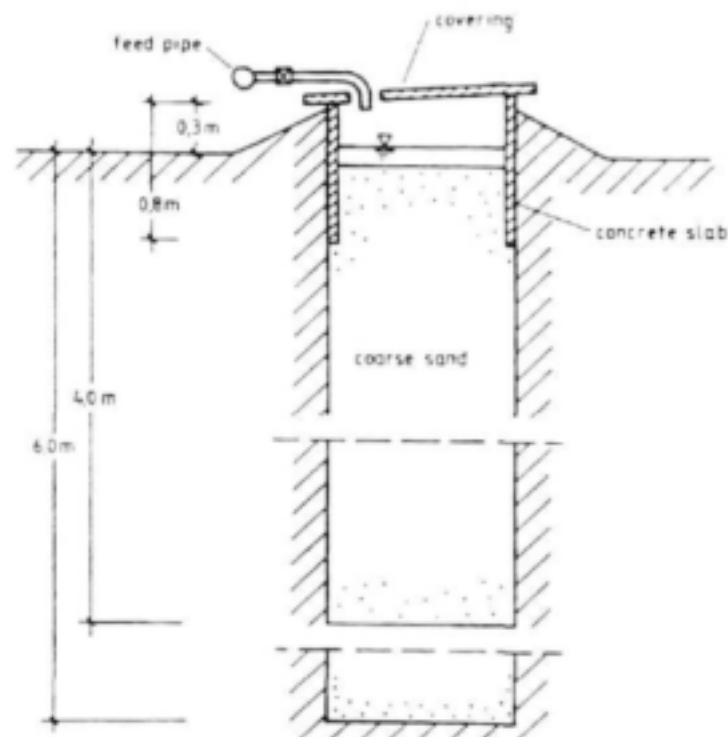


Figure 2.5 Cross section of a seepage trench (after Hantke and Schlegel, 1995)

Management requirements for subsurface infiltration systems

Clogging is the main problem with subsurface infiltration systems. Because of the high costs associated with developing such systems, it is essential that poor quality water be kept out of the pits/trenches. The key management tasks are:

- monitoring the quality of the recharge water;
- maintaining good quality recharge water.

2.2.3 Borehole injection

A rapidly growing technology is borehole recharge. This is commonly referred to as direct subsurface recharge or DSR. The advantage of this technology over other artificial recharge methods is that recharge water can bypass confining layers, it is suitable for deep seated aquifers, and the most permeable portions of the aquifer can be targeted for artificial recharge. Water can be stored during times when it is available and recovered when needed to meet seasonal, long-term, emergency or other needs. The main drawback of this approach is that water quality requirements are usually significantly higher for borehole injection than for groundwater recharge by means of basins.

Clogging around the borehole can cause a dramatic decline in aquifer entry rates. Three main

clogging parameters have been identified for predicting the clogging potential of recharge water (Peters & Castell-Exner, 1993). These are the membrane filtration index (MFI), the assimilable organic carbon content (AOC), and the parallel filter index (MLFI, "Meeloop Filter Index"). These three indices test different aspects of clogging. In the case of MFI the blocking of a filter (usually 0,45 micrometre) is tested - this relates to suspended material. The AOC tests the microbiological fouling of the aquifer material by actually testing the bacterial response to the water. The MLFI is determined by passing the recharge water through columns filled with the appropriate aquifer material. The flow rate is maintained higher than through the aquifer around the borehole. Thus, clogging occurs faster in the columns and the MLFI serves as an early warning of things to come for the recharge borehole, enabling preventative action to be taken early (Bouwer, 1996). Even under conditions where clogging rates are low, injection boreholes need to be backflushed periodically to remove accumulated solids.

The radius of the storage bubble from an injection borehole is typically less than 200 m (Pyne, 1995). During seasonal storage it will move away from the borehole at a rate dependent on:

- the aquifer's regional hydraulic gradient;
- the aquifer's permeability;
- the aquifer's porosity.

Borehole injection is often carried out by means of dual purpose boreholes, where the same borehole is used for injection and recovery. These boreholes are commonly referred to as Aquifer Storage Recovery boreholes, or ASR boreholes. The number of recharge installations using injection boreholes is increasing. In the United States of America 24 injection schemes exist, 20 are ASR schemes, and 11 of them have been developed since 1990 (Pyne, 1995). In Germany the number increased from four in 1983 to nine in 1992 (Hantke & Schlegel, 1995). Borehole injection schemes have also been developed in Canada, Israel, England, The Netherlands and Australia.

Management requirements for borehole injection systems

The potential for borehole clogging or plugging is especially high in injection wells. For this reason it is essential that the high quality of the recharge water be maintained. The main causes of clogging in injection boreholes are:

- the presence of suspended matter in the recharge water;
- the presence of air bubbles in the recharge water;
- the growth of bacteria in the gravel pack and adjacent formations;
- chemical reactions between recharge water and native groundwater which produce insoluble deposits.

While the clogging by suspended matter, air bubbles and bacteria must be minimised by correct design and operational procedures, chemical clogging can only partially be prevented by adding chemicals to the recharge water. This may however adversely affect the groundwater.

The key management issues are:

- maintaining the water treatment works which supplies recharge water;
- monitoring the recharge water quality;
- monitoring injection rates (injection efficiency);
- restoring injection efficiency by backflushing, adding chemicals, etc.;
- if the injection borehole is to be used for abstraction, then abstracted water quality needs to be closely monitored.

2.3 ARTIFICIAL AQUIFERS (SAND STORAGE DAMS)

Sand dams are typically constructed in sandy, dry river beds. They consist of a dam wall, constructed to bedrock across an ephemeral river bed, behind which layers of sand are deposited by successive floods (Figure 2.6). The artificial aquifer therefore consists of natural sand materials and natural runoff water which have accumulated as a result of an artificial wall. Sand dams have been used in suitable areas of Namibia, where extensive research was done by Wipplinger (Wipplinger, 1953). The outline of sand dams given below is taken mainly from a dissertation by Hartley, 1997.

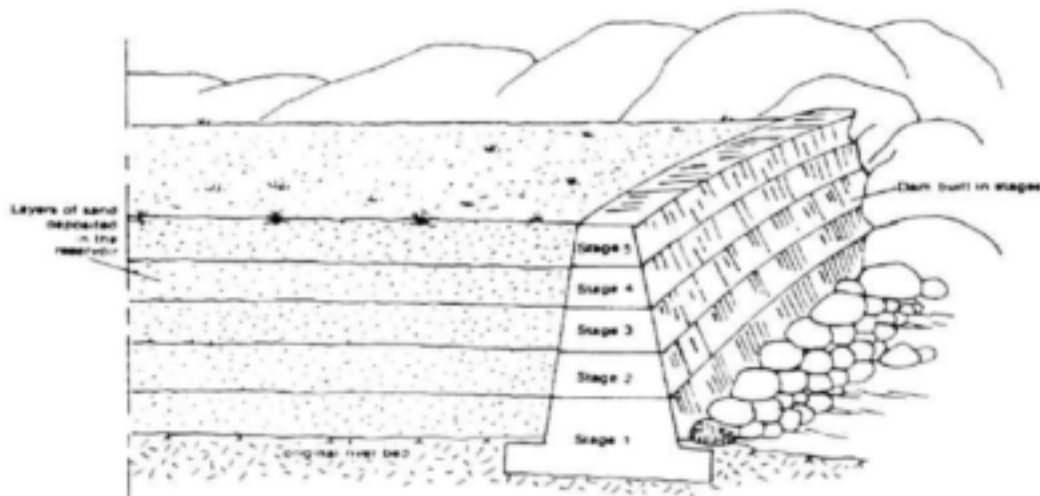


Figure 2.6 Construction principle of a sand storage dam (from Nilsson, 1988)

2.3.1 How sand dams work

Sand dams are best sited in rugged, arid areas with high runoff. Ephemeral river beds in these areas are usually sandy with a bedrock base. Flash floods, or longer ephemeral flow events, are slowed down behind the wall allowing coarser sand particles to settle out. Water infiltrates the porous, coarse sand and is prevented from migrating downstream (as would happen in a natural alluvial aquifer) by the wall.

The height of the wall is crucial in determining how much the flood flow is slowed. The rate of flow should be slowed enough to encourage the deposition of the coarser sand fraction behind the wall but the wall should allow sufficient overflow to carry away finer material. Depositions of silt will reduce the rate and amount of infiltrating water. Fine grained sand also results in slower infiltration and higher losses due to evaporation where the water table is close to the surface due to the greater capillary action in finer grained sediments. The dam wall should be raised after each successive flood.

Depending on the porosity and composition of the sediments, the storage volume of a sand-storage dam can be up to 25% of the volume of the storage basin. Water stored is available for abstraction either from a shallow well or a feeder pipe. Sometimes sand dams are sited over permeable bedrock and act as a source to recharge a deeper secondary aquifer.

2.3.2 Pre-requisites for sand dam construction

Sand dams can be constructed in areas with the following features:

- hilly or mountainous topography with well defined valleys
- arid area with high runoff resulting in ephemeral river flow
- the predominant parent rock in the area should weather to produce a coarse sandy sediment (eg. granites, sandstones, quartzites) which will act as the aquifer material
- the bedrock in the valley should be able to provide a solid rock foundation for the wall
- the dam should be underlain either by a low permeability bedrock to prevent seepage losses, or bedrock which hosts a secondary aquifer to be recharged by the sand dam
- building material for the wall should be readily available (eg. hand sized rocks for stone masonry) as construction of the wall is an ongoing process (raised after each flood).

2.3.3 Advantages of sand dams

Sand storage dams are an appropriate method of water storage in dry areas with high evaporation rates where surface dams incur high losses of stored water. The rate of evaporation from a sand dam decreases with increasing depth to the water table and increasing grain size of the sand (Figure 2.7). Fine grained sands have greater capillary action and this is relevant if the water table is at a shallow depth beneath the surface as is shown below.

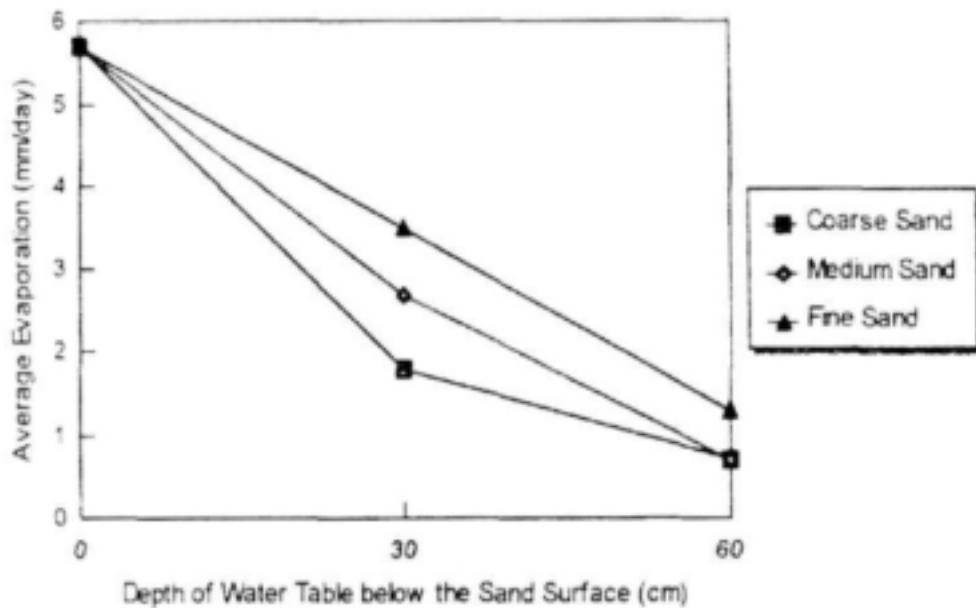


Figure 2.7 Evaporation from a sand dam as a function of the depth of the water table for different grain sizes of sand (from Hellwig, 1973)

The quality of water stored in a sand dam, as with all groundwater, is typically better than that from a surface water dam due to the protection from surface contamination.

The cost of establishing a sand dam is usually low since locally available materials, like river boulders, clay and concrete are used. The technology necessary to abstract water from a sand dam is usually simpler than that required for a deeper groundwater resource. This means that it can be more easily managed and maintained by rural communities, which are often fairly isolated.

The process of developing a sand dam, as opposed to siting and drilling a borehole, allows for greater community involvement, particularly in determining a suitable location for the dam and the construction of the dam wall. The level of technology required to abstract water from a sand dam need not be high. Motorised pumps can be used, however, hand pumps or a gravity system is usually sufficient (Figure 2.8).

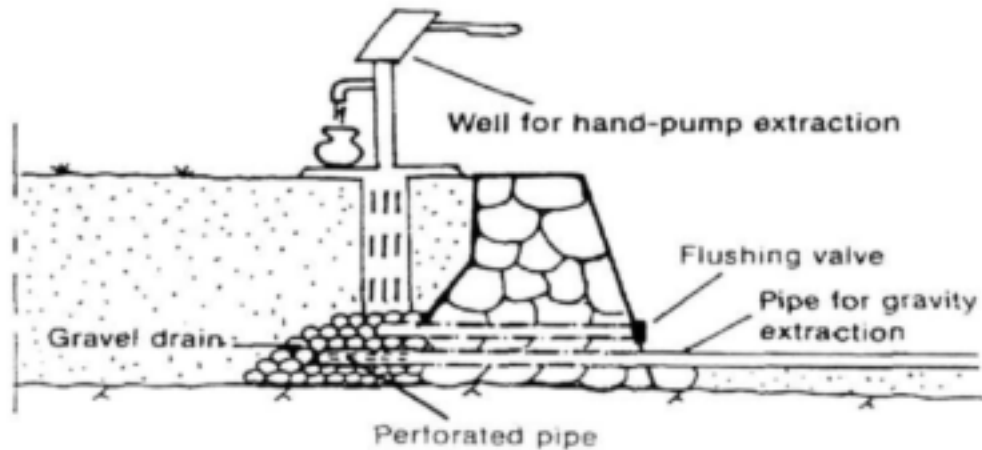


Figure 2.8 Extraction alternatives from a sand storage dam (from Nilsson, 1988)

2.3.4 Disadvantages of sand dams

Sand dams typically take a long and unpredictable period of time to reach their maximum storage capacity as they are dependent on the sediment load and water of flash floods. This means that they are a long term investment for a community and not an immediate solution to a supply problem.

The reduction in surface and subsurface water flow down-stream and the removal of much of the coarser sediment fraction may have deleterious environmental impacts. The rate of recharge of alluvial aquifers down-stream is likely to be reduced and previously permeable alluvial deposits may silt over.

The fixed storage volume of a sand dam is likely to be significantly smaller than that of a borehole in a reasonable alluvial aquifer or secondary aquifer.

Sand dams are vulnerable to silting up. This leads to lower rates of recharge. Some siltation is inevitable in most sand dams, and will not significantly alter the efficiency of the dam if it is limited in extent and forms a layer which is thin enough to be washed away by the next flood. The maintenance of silt free channels across the sand dams is advised. This can be achieved by constructing openings in the wall through which water will flow during periods of low-flow at sufficient velocity to keep fine particles in suspension (Beaumont and Burger, 1970). In some areas where significant sand is already present to host the recharging water, siltation ponds may be constructed upstream. Once the fine particles have settled out water is released to recharge the sand dam.

Chapter 3

HYDROLOGICAL FACTORS WHICH AFFECT THE POTENTIAL FOR ARTIFICIAL RECHARGE

This chapter is aimed at geohydrologists and people who have a reasonable understanding of hydrology, geochemistry and groundwater flow mechanisms. The chapter opens with a discussion on the quantity, reliability and quality of the water to be used for recharge. With South Africa's predominantly semi-arid climate, in most areas it will not be possible to depend on natural runoff for recharging aquifers - not only because of the irregularity of stream flow, but also because of the high concentration of suspended solids in these waters. In some areas, particularly where vast volumes of dammed water is lost to evaporation, it may be worthwhile considering storing dam water underground - like the Windhoek and Calvinia municipalities are currently considering (see Chapters 6 & 7).

The conjunctive use of surface and groundwater is increasingly becoming part of the thinking of water resource planners. The emphasis is on how to maximise the available water resources, and since groundwater is largely protected from contamination and evaporation losses, the concept of storing surface water underground is receiving attention. Another underutilised water resource is municipal waste water. This water source has the advantage of being available on a daily basis. The Pietersburg municipality, for example, has for a number of years, pumped treated waste water into an alluvial aquifer. During droughts, this aquifer has served as one of the most reliable sources for the town's municipal water supplies.

The next section deals with the hydraulic factors which affect the potential for artificial recharge. It opens with a theoretical description of water movement through the unsaturated zone. An understanding of this topic, including the mathematics of soil water movement, is essential when planning and designing infiltration basins. The main issue is whether the hydraulic conductivity of the soil layers will be sufficient to allow for the recharge water to move through this zone and enter the aquifer.

Will the aquifer be able to receive the recharge water? This is a concern raised by many South African geohydrologists who have experience in secondary aquifers. The issues discussed in this section relate to the suitability of an aquifer to receive additional recharge water, as a function of the aquifer's hydraulic conductivity and storage capacity.

The following section on water quality changes includes a discussion on: the change in water quality as the recharge water mixes with the natural groundwater; the precipitation potential as a result of water blending; and the role that the unsaturated zone plays in treating the recharge water.

The most critical issue with many artificial recharge schemes is clogging, or the reduction in infiltration or injection efficiency with time. Because of the importance of this issue, a sizeable portion of this chapter is given to describing the clogging processes.

The next section briefly describes the role of modelling in the design of an artificial recharge facility.

The last section describes how the success of an artificial recharge scheme is measured in terms of its recovery efficiency. The question commonly asked is: What percentage of the recharge water will be recoverable? Usually the same volume of water injected into a borehole injection and recovery scheme can be recovered (Pyne, 1995). It may also be possible to recover a greater volume than the amount injected, so long as the mixing of injected and natural groundwater provide a blend which is acceptable. This section describes how recovery efficiency is defined and the hydraulic and design factors which affect recovery efficiency.

3.1 RECHARGE WATER SOURCES: QUANTITY, QUALITY AND RELIABILITY

Recharge water can be obtained from a variety of sources, but mostly from surplus surface water, which cannot be used or reused directly for a variety of reasons, and is lost as outflows into rivers or the sea, or evaporates. It is important, however, that water for aquifer recharge purposes has a consistent high quality and a fairly predictable quantity over time (NRC, 1994). Water used for recharge purposes must be suitable for the ultimate use of the recharge water after abstraction. Municipal wastewater, for example, has a predictable quantity and quality, but it may not necessarily be suitable for artificial recharge purposes for unrestricted reuse. Stormwater runoff usually has a suitable chemical quality but other characteristics, for example sediment load and quantity variations, need special consideration. In the paragraphs below a variety of water sources are discussed and a number of general trends and characteristics identified.

3.1.1 Municipal wastewaters

Municipal wastewaters, derived from sewage treatment works, are usually of predictable volume and are a fairly uniform rate of flow over time (NRC, 1994). In South Africa, the official viewpoint of the Department of Water Affairs and Forestry is that sewage effluent is a valuable resource and should be considered as much part of a water supply system as surface water or groundwater (DWAF, 1986). Sewage effluent that is not returned to rivers or disposed to sea, is mainly used to irrigate pasture (DWAF, 1986).

Although some 33 000 million m³ of mean annual runoff can, on average be exploited for water supply purposes every year, the total volume of sewage effluent produced annually is only in the region of 300 million m³ (DWAF, 1986). Of the sewage effluent actually produced, 24 percent was reused (DWAF, 1986). The remaining effluents was retained in oxidation pond or other systems, or was not treated, soaked away or evaporated. About two-thirds of the latter was used on pasture and about one quarter was applied to crops and trees.

Wastewaters usually have a fairly consistent quality over time, but they invariably require significant treatment before being considered of sufficiently high quality for aquifer recharge (NRC, 1994, Bouwer, 1996b). A further issue to be addressed is the nutrient content. High nutrient waters present potential problems relating to algal growth and biological clogging of the recharge zone (Bouwer, 1996b).

If recharge waters are to be used for potable purposes, disinfection of water after abstraction is a requirement (Bouwer, 1985). For injection purposes, pre-chlorination of recharge water is recommended to prevent borehole clogging. When injected waters are to be used for potable water supplies, disinfection byproducts could become a human health and aquifer treatability issue.

In South Africa, legislation requires that waste-waters be treated to a high standard before being released back to the river from which the original water was abstracted. As a result wastewater is allowed to be discharged to land where insufficient treatment facilities are available and provided a permit was obtained from the Department of Water Affairs and Forestry.

Wastewater containing industrial effluents may not be suitable for artificial recharge for eventual unrestricted use. At the Atlantis artificial recharge scheme which is described in Chapter 6, separate plants were constructed for treating domestic sewage and industrial wastewater. Only the treated domestic effluent is used for artificial recharge up-gradient of the production wellfield. The treated industrial effluent is used for maintaining a hydraulic barrier along the coast for trapping fresh water inland and preventing seawater intrusion (Tredoux & Wright, 1996).

3.1.2 Storm runoff

Storm runoff is highly variable in quantity. In comparison with rural storm runoff, urbanised catchment runoff may be up to 50 percent more in total volume, and up to 500 percent more in peak discharge (Batchelor, 1997; NRC, 1994). In order to obtain a lower volume and lower peak discharges, infiltration and stormwater retention ponds, porous pavements and wetlands are recommended for catchment areas (Batchelor, 1997).

In numerous examples worldwide stormwater is used for artificial recharge. Due to the highly variable flow rate, runoff is usually collected in an impoundment or basin from which controlled release of water into recharge basins takes place after settling of the bulk of the suspended solids. The Omdel scheme in neighbouring Namibia operates in this way (Tordiffe, 1996).

Storm runoff is usually highly variable in quality, but with the exception of industrial runoff, is on average of higher chemical quality than municipal waste water and agricultural runoff (NRC, 1994). Due to the highly variable nature of its quality, storm runoff is difficult to treat in order to obtain a consistent standard (NRC, 1994). In order to obtain high quality storm runoff with little need for treatment, two management options are available, namely:

- Divert poor quality stormwater such as that which occurs as a "first flush".

- Install natural treatment basins in the catchment area, so that solid particles and trace elements are precipitated and oils, tars and waxes are trapped.

Storm runoff typically contains animal wastes, fertilisers, domestic detergents, lead, cadmium from car tyres and fertilisers, copper and zinc from roofing materials and downpipes, pesticides and herbicides, solvents and oils (Oliver *et al.*, 1996). Rainfall harvesting, in the form of roof catchment runoff is most likely the best quality storm runoff water to use.

3.1.3 Rainfall harvesting

The aim of rainfall harvesting is to collect as much high-quality water from rainfall events as possible. Thus the objectives are to minimise loss to infiltration and evaporation as well as to minimise contamination. Hence rainfall harvesting has volume-flow characteristics similar to, if not more extreme than, urban storm runoff.

In South Africa, a rainfall harvesting facility exists at Paulshoek near Kamieskroon in Namaqualand. The runoff surface consists of granitic rock with a collection wall at the lower end channelling the water into a subsurface storage basin (Roberts, 1998). Regular cleaning of the rock face is required to prevent the runoff from becoming contaminated with dirt and other materials collecting over the long dry spells.

3.1.4 River flows

Rivers have a more consistent quantity of flow than storm runoff in their catchment areas, due partly to a base-flow contribution from groundwater/interflow, and a wide range of rainfall-response times.

Compared to most countries, South Africa has a low average annual rainfall (DWAF, 1986). The climate varies from humid in the east and in the southern coastal areas to arid in the central, north-western and western areas. In the arid areas of South Africa, river flows are far more variable than those in the humid areas (DWAF, 1986). The following information is derived from duration curves for river catchments presented in a report by Midgley *et al.* (1983). From the report it can be seen that there is a high variability of flows in rivers with catchments in arid areas compared with those in humid areas. The information presented below is derived from duration curves for rivers with small to medium-sized catchments in arid areas and in humid areas. It is presented in terms of percentage, (y %), of mean annual runoff (MAR) not being exceeded for a specified percentage of time (x %) :

Dry areas: 1 % of MAR is not exceeded for about 50 to 70 % of the time.

Humid areas: 40 % of MAR is not exceeded for about 10 % of the time.

The time being considered is the time over which records have been kept (generally, between 30 to 50 years).

Thus, in the more arid areas (where MAR is small), less than 1% of MAR may be expected at a pre-specified point in a river for most of the time. (This assumes that the water is not being abstracted or "consumed" by man-induced development upstream.) In areas other than winter rainfall areas, records show that years with higher than average rainfall tend to be grouped together as part of a "wet cycle" (Alexander, 1984). Records show that flows down rivers in arid areas tend to be restricted to those years which occur during a "wet" cycle (Pitman *et al.*, 1983). This would appear to be especially applicable to rivers with large catchment areas.

At a specified point in a river system, MAR information is mostly used for water storage and water supply management purposes (Midgley *et al.*, 1983). The Hydrological Research Unit at the University of the Witwatersrand produced a series of reports whose main purpose is to present MAR statistics and relevant information for water supply planning purposes (HRU, 1981; Midgley *et al.*, 1983). In the reports, the authors present a procedure for addressing site-specific water resources planning:

- i. Obtained the necessary rights to abstract the water for storage, to store the water, to transport the water (servitudes) and to use the stored water for a specific purpose. A further addition here relates to the need to consult with interested and affected parties as well as to ensure an EIA process is set in motion.
- ii. Carry out the necessary hydro-geological and hydrological investigations.
- iii. Conduct a cost-benefit analysis.
- iv. Ensure alternative options have been investigated.
- v. Carry out a risk assessment in terms of, for example, a failure in water supply, and the cost implications of this.

The authors also present concerns that need to be addressed when estimating actual MAR statistics for the river at the point of abstraction:

- i. Determine storage (and required yield) characteristics. (For groundwater recharge projects, storage would most likely relate to temporary in-channel/ off-channel storage as well as to longer-term storage within an aquifer.)
- ii. Determine flow-duration curves for the river at the proposed point of abstraction. (See Midgley *et al.*, 1983.)
- iii. Determine how upstream uses are likely to affect the MAR characteristics at the proposed point of abstraction. Afforestation, irrigation, inter-basin transfers and upstream impoundments will all have significant affects on actual downstream MAR characteristics.

Regarding the last point, the authors suggest a precaution when estimating MAR characteristics

at a site:

Published river flow records should be compared with on-site flow-gaugings. These comparisons should take into account the fact that flow records do not represent stationary time series. The latter is partly because most catchments have changing land uses. The MAR data in reports are mostly synthesized, and represent catchments with minimal water usage in terms of afforestation, irrigation, etc.

Generally, rivers have higher water quality and are more consistent in quality than storm runoff. The major quality concerns are suspended solids, dissolved solids, pathogens and nutrients (Stuyfzand and Kooiman, 1996). The relevance of these concerns are dependent to a large extent on soils and geology as well as on land-use and vegetation cover in the catchment area (DWAF, 1986). In South Africa, the saline soils and geology of the Karoo areas contribute significantly to the high salinity of rivers with catchments dominated by saline Karoo formations (DWAF, 1986).

3.1.5 Water released from dams

Dams are designed to store water, and so the assured yield of a dam over time to water users will be more consistent than that of a river with no dam. The main effect that dams have on downstream river flows is to reduce the average annual flow volume and to make it more consistent in quantity (Davies and Day, 1986). Farm dams whose sole purpose is irrigation, tend to present a greater problem than state-owned dams regarding downstream water flows because the former have high evaporative losses due to a large surface-area-to-depth ratio, and because there is often no requirement to release a minimum assured flow to downstream users. (DWAF, 1986).

The quality of water in dams in terms of organic contaminants, suspended sediments and pathogens should be inversely correlated to the hydraulic retention time of water in a dam, in a similar way to which the relationship holds true for sewage water in oxidation and stabilisation ponds (this is certainly true of the Vaal Dam - see Rand Water Board monitoring records). In terms of other quality parameters, comparing them to upstream river waters, the following is likely to hold true:

- Due to a portion of water in a dam being lost to evaporation, the total dissolved solids concentration of water in a dam would be higher than that of upriver waters, on average (DWAF, 1986).
- Water at the bottom of a dam is colder and contains little dissolved oxygen (Davies and Day, 1986).
- Water at the surface of a dam is warmer, contains a lower suspended solids concentration and has a higher nutrient concentration (Davies and Day, 1986).

3.1.6 Agricultural return flows

Agricultural return flows do not include stormwater runoff. Agricultural return flows will be less variable in flow characteristics than stormwater as they are dependent on rainfall as well as on irrigation, and consist of both surface as well as subsurface runoff (interflow).

Agricultural return flows generally contain high concentrations of dissolved solids, suspended solids, nutrients, pesticides and trace elements (NRC, 1994; DWAF, 1986). In some developed countries, trace elements which require special consideration include selenium, uranium, boron and arsenic. Total dissolved solids (TDS) and nitrates are also of concern since TDS is increased by up to ten times that of irrigation water, and nitrate concentrations of above 100 mg/l are not uncommon (NRC, 1994). Agricultural return flows are generally not recommended for recharge purposes because of their poor quality (NRC, 1994).

3.1.7 Comparison of the water quality characteristics of selected sources

In Table 3.1 the quality-related characteristics of irrigation return flow, urban storm runoff and treated municipal wastewater in the USA are compared. (Source: NRC, 1994).

Table 3.2 shows the advantages and disadvantages of using various source waters for ground water recharge (Sources: NRC, 1994; DWAF, 1986; Midgeley, 1983; Davies and Day, 1986).

Table 3.1 Comparison of Quality Parameters for Irrigation Return Flow, Urban Storm Water Runoff, and Treated Municipal Wastewater (Source: NRC, 1994)

IRRIGATION RETURN FLOW	URBAN STORMWATER	SECONDARY TREATED MUNICIPAL WASTEWATER
<p>General</p> <ul style="list-style-type: none"> ➤ TDS (total dissolved solids) > 500-10,000 mg/l. (There is extreme variability, low TDS is largely Ca²⁺, Mg²⁺, HCO₃⁻, and high TDS largely Na⁺, Cl⁻, SO₄²⁻, depending on source quality and location) ➤ Total suspended solids 7-3,000 mg/l ➤ Nitrate, up to 10-20 times drinking water standards 	<ul style="list-style-type: none"> ➤ Turbidity 10-100 Nephelometric turbidity units (> 1,000 NTU in construction area runoff) ➤ Total suspended solids 25-1,000 mg/l (> 10,000 mg/l in construction runoff) ➤ Total dissolved solids 50-100 mg/l ➤ Chemical oxygen demand 50-100 mg/l ➤ BOD₅ (5-day biological oxygen demand) 10-25 mg/l (ultimate biochemical oxygen demand is several hundreds mg/l) ➤ Nitrate 0.5-10 mg/l 	<ul style="list-style-type: none"> ➤ Total suspended solids 10-25 mg/l ➤ Total dissolved solids 200-5,000 mg/l ➤ Total organic carbon 15-25 mg/l ➤ Chemical oxygen demand 40-70 mg/l ➤ BOD₅ 15-30 mg/l ➤ Nitrate 0.4-30 mg/l
<p>Biological</p> <ul style="list-style-type: none"> ➤ Inadequate data 	<ul style="list-style-type: none"> ➤ Human pathogens <i>Pseudomonas aeruginosa</i> 10³-10⁷/100 ml <i>Shigella</i>, protozoa, and viruses likely present (limited data) ➤ Faecal coliforms 10³-10⁷/100 ml 	<ul style="list-style-type: none"> ➤ Human pathogens: Almost complete removal with disinfection ➤ Faecal coliforms < 10/100 ml
<p>Organics</p> <ul style="list-style-type: none"> ➤ No extensive data currently available ➤ Pesticides, dependent on local practice, some notable concentrations observed (USA data) terbacel 10 µg/l 2,4-D 110 µg/l atrazine < 10-1,000 µg/l chlorobanfol 0.04-3.7 µg/l DCPA 150 µg/l (mean) EPTC 1,250 µg/l (mean) Prometryn 180 µg/l (mean) Methomyl 120 µg/l (mean) 	<ul style="list-style-type: none"> ➤ Pesticides, mostly in residential base flows DDT up to 1 µg/l lindane up to 1 µg/l endrin up to 1 µg/l chlordane up to 10 µg/l methoxychlor up to 10 µg/l ➤ Volatile organic compounds (benzene and toluene present in industrial runoff) ➤ Polycyclic aromatic hydrocarbons (mostly in particulate forms) benzo(a)pyrene 10-100 µg/l fluoranthene, phenanthrene, chrysene, pyrene, and anthracene 1 to 25 µg/l ➤ bis (2-ethylhexyl) phthalate up to 60 µg/l ➤ Others 1,2-dichloroethane, ethylbenzene pentachlorophenol, and tetrachloroethylene up to 10 µg/l (mostly in industrial runoff) PCBs (Poly-chlorinated biphenyls) up to 10 µg/l (industrial runoff) 	<ul style="list-style-type: none"> ➤ Volatile organic compounds methylene chloride < 1.5 µg/l chloroform 3.5-7.5 µg/l ➤ Polycyclic aromatic hydrocarbons benzo(a)pyrene 0.1-0.4 µg/l fluoranthene 0.01-0.35 µg/l pyrene 0.2-1 µg/l ➤ Phthalates up to 200 µg/l ➤ Others ethylbenzene 11 µg/l pentachlorophenol 4 to 300 µg/l 1,1,2,2-tetrachloroethylene 1-60 µg/l
<p>Metals and other inorganics</p> <ul style="list-style-type: none"> ➤ Mostly associated with soluble phase ➤ Selenium up to 300 µg/l ➤ Uranium up to 300 µg/l ➤ Boron 190-28,000 µg/l ➤ Arsenic 1-190 µg/l 	<ul style="list-style-type: none"> ➤ Mostly associated with particulate phase ➤ Antimony 2-100 µg/l ➤ Chromium 1-200 µg/l (25% filterable) (mostly in industrial runoff) ➤ Arsenic 1-50 µg/l (average 1 µg/l) ➤ Beryllium 1-50 µg/l ➤ Cadmium 1-15 µg/l (25% filterable) ➤ Selenium 1-100 µg/l ➤ Zinc 200 µg/l average, can be 1,000 µg/l in runoff from galvanized metal (mostly filterable) ➤ Mercury 0.1-200 µg/l ➤ Nickel 1-200 µg/l (50% filterable) ➤ Cyanide 1-300 µg/l ➤ Lead 10-500 µg/l, has decreased by about 10 x over past 20 years (only 10% filterable) 	<ul style="list-style-type: none"> ➤ Arsenic 5-23 µg/l ➤ Boron 300-2,500 µg/l ➤ Cadmium < 5-200 µg/l ➤ Chromium < 1-100 µg/l ➤ Copper 6-50 µg/l ➤ Lead 3-350 µg/l ➤ Mercury < 2-10 µg/l ➤ Nickel 3-600 µg/l ➤ Zinc 4-350 µg/l

Table 3.2 Advantages and disadvantages of using various source waters for ground water recharge purposes
(Sources: NRC, 1994; DWAF, 1986; Midgeley, 1983; Davies and Day, 1986)

	ADVANTAGES	DISADVANTAGES
River/dam Water		
River water	<ul style="list-style-type: none"> ➤ Fairly low pathogen content in rural areas. ➤ Low TOC (total organic carbon) for reduced disinfection by-product formation potential. ➤ Low contaminant load in mountain catchments. 	<ul style="list-style-type: none"> ⚡ Generally, variable in flows, suspended solids and TDS (total dissolved solids). ⚡ Highly intermittent flows in more arid areas. ⚡ High TDS base-flows in Karoo areas. ⚡ High suspended solids loads for rivers in eroding catchments.
Water released from dams	<ul style="list-style-type: none"> ➤ More consistent in quality than upstream river. ➤ More assured yield than upstream river. ➤ Lower TOC, pathogens, suspended solids, heavy metals and nutrients than upstream river. 	<ul style="list-style-type: none"> ⚡ High evaporative losses in shallow dams. ⚡ Higher average TDS concentrations than upstream river. ⚡ High storage costs.
Municipal Wastewater		
Advanced treated municipal wastewater	<ul style="list-style-type: none"> ➤ Best quality municipal wastewater. ➤ Low TOC for reduced disinfection by-product formation potential. ➤ Relatively constant flows. 	<ul style="list-style-type: none"> ⚡ High treatment cost.
Secondary-treated municipal wastewater	<ul style="list-style-type: none"> ➤ Most common. ➤ Relatively constant flow. ➤ High volume. 	<ul style="list-style-type: none"> ⚡ Moderate to poor water quality.
Primary-treated municipal wastewater	<ul style="list-style-type: none"> ➤ High TOC for possible improved denitrification. ➤ Relatively constant flow. 	<ul style="list-style-type: none"> ⚡ Poor water quality; higher toxicants, nutrients, BOD (biological oxygen demand), and suspended solids than other municipal wastewaters. ⚡ High disinfection by-product formation potential.
Urban Stormwater		
Industrial area stormwater		<ul style="list-style-type: none"> ⚡ Highly irregular toxicant quality (likely contamination from industrial processes and contact with grossly polluted soils).
Residential area stormwater	<ul style="list-style-type: none"> ➤ Fair quality water after "first flush". 	<ul style="list-style-type: none"> ⚡ Irregular flows (highly intermittent).
Agricultural Irrigation Return Flows		
Irrigation return flow		<ul style="list-style-type: none"> ⚡ High pesticides and herbicides. ⚡ High nutrients and salts. ⚡ Fairly irregular flows.

3.2 SOIL MATRIX CHEMISTRY AND WATER QUALITY ISSUES

The quality of the water which reaches the aquifer via an artificial groundwater recharge system is influenced by a number of factors. The initial quality of the recharge water, however, often remains the main characteristic determining the final water quality. In the case of injection into the aquifer, quality changes are usually limited to phenomena such as calcium dissolution from a carbonate aquifer. On the other hand, seepage from a surface recharge facility moving through the unsaturated zone, may undergo substantial quality changes before reaching the aquifer. The design of the system should be such that these changes would improve the quality of the water, for example degradation of organic compounds, before it reaches the saturated zone.

3.2.1 Soil matrix

In arid areas such as the Karoo in South Africa, saline soils are common (HRU, 1981). Due to high potential evaporation rates in warm, arid areas, low infiltration rates of less than 0,3 m per day could present problems relating to increased TDS levels of recharge waters (Bouwer, 1985). This is especially relevant for areas in the Karoo, as soils in this region are usually either impermeable or else have impermeable horizons, making those areas unsuitable for artificial recharge using surface infiltration.

Soils containing significant levels of iron and manganese can release iron and manganese into infiltrating recharge waters under anaerobic conditions which occur when recharge waters has a sufficient concentration of nutrients and dissolved organic carbon (Ineson, 1970; Pavelic and Dillon, 1996).

Soils which have a clay component generally tend to become less permeable when irrigated/inundated by waters with a relatively high sodium adsorption ratio (SAR) and low TDS value (Bouwer, 1985). The higher the SAR and the lower the TDS, the more clays tend to disperse and clog up soil pores (DNDE, 1982).

Soils which contain organic carbon are able to attenuate organic chemicals in recharge waters (Oliver, *et al.*, 1996). Soils which contain iron, aluminium and manganese oxides and oxy-hydroxides are effective at attenuating anionic and cationic contaminants in recharge waters (Domenico & Schwartz, 1990). Soils containing organic carbon and clays are very effective at attenuating dissolved heavy metals (Adriano, 1986; Fuller, 1978). Soils containing calcium and magnesium carbonates are effective in causing specific heavy metals and anions to precipitate out of solution (Adriano, 1986; Fuller, 1978). An important factor which influences the mobility of ionic contaminants in soil is pH, where low pH mobilises cationic contaminants and high pH mobilises anionic contaminants in a soil solution (Allen, 1997).

Chemical interaction between the recharge water and the aquifer material or the *in situ* groundwater could be of significance. When the recharge water is under saturated with respect to calcium carbonate, which is often the case with surface water, it will dissolve limestone, calcite, or dolomite which may form part of the aquifer material, until equilibrium is reached.

In the case of borehole injection, the pressure differences are such that water that may be in chemical equilibrium at the surface, will be under saturated once injected into the aquifer. Carbonate mineral dissolution will, in general, not create a serious hazard, except in isolated cases, for example when sinkhole formation could be a possibility.

Other types of chemical interactions, for example those that cause precipitation, could have serious clogging consequences. This is possible, for example, in the case of water with different redox potentials. If oxygenated water is recharged into an aquifer with reducing conditions, dissolved iron and manganese will be precipitated in the aquifer.

3.2.2 Water quality

The quality of the recharge water is a key variable in the decision on the type of artificial recharge system to be employed. High quality, low turbidity water can be utilised successfully in any kind of recharge system. However, when the water quality needs improvement, for example, when nutrient or organic compounds need to be removed, a surface infiltration system involving soil aquifer treatment (SAT) may be necessary. Alternatively, pretreatment options may be exercised to reduce turbidity and improve quality for direct recharge. Also, aquifer storage and recovery (ASR) systems sometimes allow for a controlled degeneration in aquifer water quality where the recovered water is to be used for restricted purposes. In such cases the turbidity must still be low, but the chemical and/or microbiological quality may be impaired. In general, the extent of post treatment of water recovered from an artificial recharge operation, will mainly depend on the intended use.

In the paragraphs below, water quality aspects pertaining to the above are discussed in broad terms in the context of literature information and examples.

Chemical quality changes taking place in the recharge process can be simulated by mass transport modelling following successful simulation of the groundwater flow with suitable models. For most of the groundwater flow models an appropriate mass transport model exists. The most popular of these are discussed in section 3.5.

3.2.2.1 Pretreatment of recharge water

Recharge waters generally need to receive a sufficient degree of pretreatment to minimise degradation of groundwater quality, and to minimise the need to extensively treat the water after abstraction (NRC, 1994). From an operational point of view, pretreatment to reduce the extent of clogging problems (see section 3.4), is a further requirement (NRC, 1994; Bouwer, 1985).

In the case of direct subsurface injection, the recharge water generally needs treatment to near potable water quality standards and chlorination before injection into an aquifer. Advanced water treatment to remove suspended solids, assimilable organic carbon, nutrients, and microorganisms is required for pretreatment of recharge waters, then disinfection (Bouwer, 1996b). Possible exceptions are high transmissivity aquifers, when recharge water is not required for potable use (Pyne, 1995). When water of potable quality is recharged, it only needs chlorination after

abstraction (Pyne, 1995).

An issue which needs to be addressed regarding pretreatment is one relating to using chlorine disinfection to promote the potential of an aquifer to perform as a bioreactor without attendant clogging problems (Bouwer, 1996b). Using disinfection to cause bioreactor activities to move away from the recharge zone, out into the aquifer, could promote the aquifer as an effective treatment medium for organic contaminants, disinfection byproducts, nitrates and other contaminants (Bouwer, 1996a,b). This could then reduce the pretreatment requirements. Bouwer (1996a,b) suggests that the amount of disinfection needed for this will be related to the Total organic carbon concentrations in the recharge water. Disinfection with chlorine reduces the pH, making the water more corrosive (Pyne, 1995).

3.2.2.2 Surface infiltration systems

This technique involves application of recharge water to the soil surface above an unconfined aquifer. As explained elsewhere, application of recharge water occurs through channels, spreading basins or through ephemeral stream beds which are in hydraulic connection with aquifers (DNDE, 1982). Soil-aquifer treatment should be considered an important part of the pretreatment process (Bouwer, 1985). The unsaturated zone can bring about considerable quality improvements to infiltrating recharge waters and so reduce pre and post treatment requirements (NRC, 1994). This is true especially for suspended solids, total organic carbon, biological oxygen demand, organic chemicals, heavy metals and pathogens (Chang and Page, 1985; Crites, 1985).

Apart from its use to replenish an aquifer, for aquifer storage purposes, and to maintain base flows in rivers, artificial recharge can also serve to improve water quality in an aquifer, to treat wastewater, to improve goal-use quality of surface water, and to create hydraulic barriers against saline water intrusion (NRC, 1994; DNDE, 1982; Bouwer, 1996a,b).

Most issues of concern may be expressed in terms of treatment capacity versus infiltration rates. The treatment capacity of soils for recharge water should be high if potentially toxic trace elements (PTEs) and organics are an issue (DNDE, 1982; Oliver *et al.*, 1996). A good treatment capacity is needed for PTEs, pathogens and phosphorus. Requirements generally are a high clay percentage, high organic content, and soils of low permeability and high porosity. However, such soils do not aid infiltration rates (DNDE, 1982). Low infiltration rates in soils are commensurate with high treatment capacity, but land area requirements are then large. High infiltration rates reduce the treatment potential of soils, which in turn increases the need for further pretreatment of source waters prior to recharge. High infiltration rates also reduce the portion of recharge water which is lost to evaporation (Bouwer, 1996b; DNDE, 1984).

Shallow ponds (less than 0.3 m) help to increase the dissolved oxygen of recharge waters (NRC, 1994; Bouwer, 1985). A high dissolved oxygen content assists in the nitrification of ammonia and stimulates organic contaminant degradation processes in the soil (Frank, 1970).

The presence of algae in recharge water accelerates the clogging rate (Frank, 1970; NRC, 1994).

Seasonal algal growth and die-off causes seasonal changes in pH and in the dissolved oxygen content of pond water (NRC, 1994). Changes in pH affect the chemical stability of the water, and so also the potential of the soils to treat the recharge water (NRC, 1994; Bouwer, 1996a).

Algal die-off promotes anaerobic conditions which in turn promote a rapid clogging rate in the soil (NRC, 1994; Okubo & Matsumoto, 1983). Anaerobic conditions stimulate denitrification of nitrates in the recharge water, provided there is sufficient dissolved organic carbon (DOC) present (Bouwer, 1996a). Anaerobic conditions stimulate the dissolution of iron and manganese from soils, giving potential problems in the post treatment of extracted water (Ineson, 1970; Pavelic and Dillon, 1996).

There is a need to compromise between best infiltration rate and optimum treatment capacity. Loamy sands (a fairly high infiltration rate) to clay loams (good treatment capacity) are the most suitable, dependent on recharge water quality and end use of recharge waters (DNDE, 1982; NRC, 1994).

The development of a biological film on the soil surface helps to treat recharge waters, but it reduces the infiltration rate over time while the land is inundated, and treatment capacity is improved proportionately (DNDE, 1982). This is true even for sandy soils. Introducing a drying cycle helps to restore the original soil permeability (Bouwer, 1996a; NRC, 1994). An important issue that needs to be addressed is how to manage the wetting/drying cycles so as to obtain satisfactory infiltration rates and treatment capacities over the wetting period (NRC, 1994; Bouwer, 1996a; DNDE, 1982). If managed properly, soils may be used to remove a large proportion of the nitrogen from recharge waters (Bouwer, 1985). More than half of the total nitrogen will be removed under a proper balance of aerobic/anaerobic conditions (dependent on flooding/drying cycles) and with sufficient carbon availability, but even under optimum conditions only up to 90 percent of the nitrogen may be removed (Bouwer, 1996a; DNDE, 1982). Pathogen removal is dependent in part on wetting/drying cycles, and on the hydraulic retention time (HRT) of recharge waters in the subsurface system. A retention time of at least six months for sufficient pathogenic organism die-off is recommended, but there could still be some risk in terms of viruses (Bouwer, 1996a; DNDE, 1982).

Phosphates in waste waters are generally readily sorbed or precipitated in soils (Bouwer, 1985). In soil, phosphate precipitates out as complexes of iron, aluminium or calcium, depending on soil pH and cation availability (Bouwer, 1985).

3.2.2.3 Subsurface injection systems

Direct subsurface recharge (DSR) systems involve the injection of water into an aquifer via a well or borehole. DSR projects generally apply to confined and semi-confined aquifers (Bouwer, 1996b; Pyne, 1995). Recently a subcategory of DSR systems has been extensively researched, where injected water is abstracted from the injection point (aquifer storage and recovery, ASR systems) as opposed to systems where recharge water is abstracted from another point. ASR systems are far less complex than other DSR systems, and are likely to give fewer problems relating to clogging (Sibenaler, 1996, Vecchioli *et al.*, 1975; Gerges *et al.*, 1996). As residence

time and travel distance of recharge water in ASR systems is much lower than in other DSR systems, it is likely that ASR systems will not have as effective "polishing" effects, especially in terms of pathogen treatment (Bouwer, 1996a).

The quality of water which may be injected depends firstly on the suspended solids content of the recharge waters and the transmissivity of the aquifer, and secondly on the required use of the recharge water, the quality of water in the aquifer, the chemical compatibility between the groundwater and injected water in terms of sustainability of the aquifer under recharge use, biological clogging potential, and the requirements of applicable regulations (Gerges *et al.*, 1996; Pyne, 1995; NRC, 1994; Bouwer, 1996a; DNDE, 1982).

Technically, issues of concern are generally in terms of borehole clogging (recharge/abstraction efficiency) and geochemical reactions in the aquifer (Oaksford, 1996; NRC, 1994; Pyne, 1995). Very little treatment to the recharge water should be expected from ASR schemes, except for reduction in disinfection byproducts, phosphates and pathogens when long hydraulic retention times are involved (DNDE, 1982; Vecchioli *et al.*, 1975; Bouwer, 1996a). Pyne (1995) states that ASR systems' ability to treat recharge waters, depends on characteristics of the recharge water, the native groundwater, the aquifer media and injection/ abstraction of stored waters, but insufficient information is currently available on this to derive guidelines. The treatment potential of other DSR systems would depend on the same characteristics, but should be more effective than ASR systems due to longer residence times and travel distances (Bouwer, 1996a).

Oxygenated waters injected into anoxic groundwater generally present geochemical problems (Pyne, 1995; DNDE, 1982). The consequent oxidation of anoxic groundwater lowers pH, converts sulphide to sulphate, and causes soluble iron and manganese to precipitate as oxyhydroxides. Iron oxyhydroxides precipitate very rapidly, to the extent they may control equilibrium reaction paths in an aquifer (Pyne, 1995). If the aquifer media contains iron sulphide or iron carbonates, iron oxyhydroxides can rapidly form on oxygenation and may cause clogging (Pyne, 1995). It may be of interest to note that oxygenated recharge waters are beneficial for soil aquifer treatment systems, but this is not necessarily true for recharge waters which are to be injected into aquifers.

If the recharge water is disinfected, then the production of disinfection byproducts (DBPs) becomes an issue, albeit not a major one (Bouwer, 1985).

Aquifers which have a clay component are generally sensitive to the sodium adsorption ratio (SAR) and total dissolved solids (TDS) values of recharge waters. The higher the SAR and the lower the TDS of recharge water, the more clay particles tend to disperse and clog up pores or apertures in an aquifer media matrix (DNDE, 1982).

3.2.2.4 Post treatment of recovered water

For properly designed and managed soil-aquifer treatment systems, results show that for infiltrated sewage effluents, abstracted waters are no less suitable *for potable use* than currently accepted potable water source supplies (Bouwer, 1996a). Treatment would be similar to that

required for normal water sources.

If the recharge water becomes anoxic in the aquifer, it is likely that aeration would be needed on extraction. If the extracted water is potentially corrosive or else supersaturated with regard to calcium/magnesium carbonate, it will need to be chemically stabilised. Occasionally sodium bisulphite, calcium chloride and other chemicals are formed, and so must be treated (Pyne, 1995).

If manganese is present in the storage zone and the pH of recharge waters is less than 8, manganese may be present in the abstracted waters in sufficiently high concentrations so as to cause taste problems and black discolouration of wetted surfaces on oxygenation.

Chlorination causes potentially carcinogenic disinfection by-products to form. The ones of most concern are trihalomethanes (THMs) and halogenated acetic acids (HAAs). These are generally removed from chlorinated drinking water during aquifer storage (Bouwer, 1985; Pyne, 1995).

For properly designed and managed soil aquifer treatment systems, experience shows that *no pretreatment or post treatment is required for non-potable use of abstracted recharge waters* (Bouwer, 1996a,b).

3.3 HYDRAULIC FACTORS WHICH AFFECT THE POTENTIAL FOR ARTIFICIAL RECHARGE

3.3.1 The unsaturated zone

One of the methods that is often used for groundwater artificial recharge is basin infiltration, whereby water is applied at the surface and allowed to infiltrate to the aquifer. The success of infiltration recharge schemes is largely dependent upon the ability of the unsaturated zone to transmit water. In addition, the soil characteristics should be such that while the optimal infiltration rate is obtained, the contact time with the soil and aquifer material should also be sufficient for ensuring the desired quality improvements by sorption, degradation and other processes.

In this section on the unsaturated zone, the theory of soil water movement is given (mainly from Sililo, 1994). Emphasis is on the driving force of soil water movement, the unsaturated hydraulic conductivity, the effect of layering and the quantification of infiltration rates. The implications of the infiltration theory to recharge schemes is given in the last section.

3.3.1.1 The mechanism of soil water movement

In the unsaturated zone, the driving force of water movement is the gradient of potential energy. Water will tend to move in the direction of decreasing energy status. The energy status of soil water is usually referred to as the water potential. This potential expresses the specific potential energy of soil water relative to that of water in a standard reference state. The standard state that is generally used is a hypothetical reservoir of pure and free water at atmospheric pressure

defined as having a potential of zero. Physically, the water potential consists of three major force fields which result from the attraction of the solid matrix for water, presence of solutes and action of external gas pressure. Thus the equation for water potential can be written as

$$\Psi_w = \Psi_m + \Psi_{ex} + \Psi_{os} \quad [1]$$

where Ψ_m is the matric potential, Ψ_{ex} is the potential arising from external gas pressure and Ψ_{os} is the osmotic potential. Matric potential arises from the interaction of water with the matrix of solid particles in which it is embedded. Capillary and adsorptive forces due to the soil matrix attract and bind water in the soil and lower its potential energy below that of bulk water.

When the water is located at an elevation different from that of the reference level, the gravitational potential, g has to be considered. Using units of length, the gravitational potential of soil water at any point will equal the elevation of that point above the chosen reference level. The sum of the gravitational potential and the water potential defines the total water potential (Ψ_t). This can be written mathematically as

$$\Psi_t = \Psi_w + \Psi_z \quad [2]$$

A knowledge of the total water potential at different levels in a soil water system is essential in determining the direction of water flow. Water will tend to move from positions of high to positions of low total water potential.

Assuming that osmotic effects and air pressure are negligible, the hydraulic potential, h can be written as

$$h = \Psi_h = \Psi_m + \Psi_z \quad [3]$$

Liquid water will flow as a result of a hydraulic potential gradient. For equilibrium conditions, the hydraulic potential will be everywhere constant; it will vary throughout the parts of the soil in which flow is occurring. In situations where the hydraulic gradient opposes and exceeds the gravitational gradient, the flux will be upwards. It will be downwards if the hydraulic gradient is acting in the same direction as the gravitational gradient.

3.3.1.2 The hydraulic conductivity function

Infiltrating water will move through the soil at a rate dependent upon the nature of the transmitting pores, their water content, and the potential gradients existing in the system. As the water content of a soil decreases, the largest pores are emptied first because the water potential in them is high. As the amount of water in the pores decrease, the resistance to water movement in the system increases rapidly. Thus as a soil becomes drier and the remaining water is held in relatively small pores at progressively lower potentials, the total conductive volume becomes less and the interconnecting pathways reduce and become more tortuous. The conducting capacity is usually referred to as the hydraulic conductivity (K). In a saturated soil, the hydraulic

conductivity is constant while in unsaturated systems, the conductivity will have different values for different moisture contents.

3.3.1.3 Mathematics of soil water flow

The basis of the physical theory of soil water flow is Darcy's Law, which can be written in the form

$$v = -K\nabla\psi_h \quad [4]$$

where v is the macroscopic flow velocity(L/T), K is the hydraulic conductivity, and ∇ is defined as performing the operation $[i(\partial/\partial x) + j(\partial/\partial y) + k(\partial/\partial z)]$ upon a scalar quantity. Here i, j, k , are, respectively the unit vectors in the x, y , and z directions of the cartesian co-ordinate system. The Darcy equation may be used to describe steady flow of liquids in unsaturated systems. In order to obtain a general flow equation that accounts for both transient and steady flow conditions, Darcy's Law must be considered together with the law of conservation of matter. The latter as applied to water flow into soil expresses the fact that water entering a volume element is either stored or flows out. The law may be stated mathematically as

$$\frac{\partial\theta}{\partial t} = -\nabla \cdot v + \text{sources} + \text{sinks} \quad [5]$$

If any change of state of the water in the system is neglected and an assumption that roots are absent is made, then sources and sinks must be zero. Assuming isothermal conditions, and substituting Equation 4 into the above equation, we obtain

$$\frac{\partial\theta}{\partial t} = \nabla K \cdot \nabla\psi_h \quad [6]$$

This equation may be rewritten as

$$\frac{\partial\theta}{\partial t} = \frac{\partial}{\partial x} \left(K \frac{\partial\psi}{\partial x} \right) + \frac{\partial}{\partial y} \left(K \frac{\partial\psi}{\partial y} \right) + \frac{\partial}{\partial z} \left(K \frac{\partial\psi}{\partial z} \right) + \frac{\partial K}{\partial z} \quad [7]$$

Equation 7, usually referred to as the Richards equation, is the governing non-linear differential equation for unsaturated zone flow, the solution of which depends on the initial and boundary conditions. For vertical flow, Equation 7 may be rewritten as

$$\frac{\partial\theta}{\partial t} = \frac{\partial}{\partial z} \left(K \frac{\partial\psi}{\partial z} \right) + \frac{\partial K}{\partial z} \quad [8]$$

Equation 8 is the differential equation which describes *vertical infiltration of water* in soil as a function of position and time. The equation contains two terms on the right hand side. The first term describes the contribution of the gradient of the matric potential to the transport process, while the second term expresses the contribution of gravity. The predominance of one term over the other will depend on the initial and boundary conditions and on the stage of the process considered. The first term predominates in cases where the influence of matric gradient is more important than that of gravity, for example at the beginning of infiltration. As infiltration continues and the water penetrates deeper, the average suction gradient decreases and the second term becomes progressively more important until it dominates the infiltration process.

A number of *analytical solutions* of the governing equation for water in unsaturated media have been described in literature. Milly (1988) gave numerous citations. Among the earliest proposed solutions was that by Green and Ampt (1911) sometimes referred to as the Delta function solution. The principal assumptions of this approach are that there exists a distinct and precisely definable wetting front, and that the matric suction at the wetting front remains effectively constant, regardless of time and position. The wetting front is viewed as a plane separating uniformly saturated soil behind it, of uniform hydraulic conductivity, K_s , from uniformly unsaturated and as yet uninfluenced soil beyond it. For vertical infiltration, the Delta function model can be written as

$$\frac{\partial I}{\partial t} = \Delta\theta \frac{\partial l_f}{\partial t} = K_s \frac{(h_0 - \Psi_f + l_f)}{l_f} \quad [9]$$

where h_0 is the ponding depth, Ψ_f is the effective pressure head at the wetting front and l_f is the length of the wetted zone, I is the cumulative infiltration, $\Delta\theta = \theta_t - \theta_i$ (where θ_t is the transmission-zone wetness during infiltration and θ_i is the initial moisture content prior to infiltration).

Another solution that is usually referred to is that of Philip (1957). Philip's approximate solution for vertical infiltration into a homogeneous soil with water ponded on the surface can be written as follows

$$I = St^{1/2} + At \quad [10]$$

where i is the cumulative infiltration; S is a physical property of the porous media called sorptivity which is essentially a measure of the medium to absorb or desorb liquid by capillarity. The parameter A is related to the saturated hydraulic conductivity. For saturated soil surfaces, $A = K_s$. For horizontal infiltration as $t \rightarrow 0$ ($i \sim st^{1/2}$) while the infiltration rate approaches A as $t \rightarrow \infty$.

Equation 8 can also be solved by using numerical techniques. The techniques used are similar to those used for groundwater flow in saturated systems. The main difference is that for the unsaturated flow problem, the hydraulic conductivity and water capacity are functions of the dependent variable (water content or potential). Thus, for the unsaturated flow problem, the governing equation is highly non-linear.

3.3.1.4 Infiltration in layered soils

The rate of infiltration will be affected if the wetting front encounters a material in which most of the pores are either larger or smaller than those through which it has been moving. A typical situation is that of a coarse layer of higher saturated hydraulic conductivity overlying a finer textured lesser conductive layer. When the infiltrating water comes to the interface, the finer pores in the bottom layer will begin to fill rapidly. At this stage, the infiltration will still be controlled by the top coarse layer. As the wetting front advances further into the bottom material, resistance to flow owing to the fineness of the pores may be so great that flow is markedly reduced. Thus, in the long run, the fine bottom layer is expected to control the infiltrating rate.

An opposite case to the one described above, is a situation where a fine-textured top layer overlies a coarse textured bottom layer. The infiltration rate in such a system is initially determined by the top layer. When water reaches the interface, the wetting front may pause temporarily. Before the wetting front can advance, the moisture tension in the sublayer must decrease until it is low enough to allow the pores to fill with water. Restriction of the advance of the wetting front lowers the infiltration rate. Flow, however, continues in response to a small potential gradient (Miller and Gardner, 1962). When water finally flows in the lower layer, the latter can not become saturated since the restricted rate of flow through the less permeable upper layer cannot sustain flow at the saturated hydraulic conductivity of the coarse lower layer.

3.3.1.5 Estimating the ability of the unsaturated zone to transmit water

The Delta function model (Equation 9) is often used to approximate flow of water in the unsaturated zone. For large t , the infiltration rate, v_i can be written as

$$v_i = K_s \frac{(h_0 - \psi_f + l_f)}{l_f} \quad [11]$$

Equation 10 shows that increasing the water depth h_0 will increase infiltration at the beginning of the infiltration process when l_f is small. In situations where l_f is already large, increasing h_0 will have little or no effect on the infiltration rate. It is usually assumed that the hydraulic gradient is equal to one and that the infiltration rate (v_i) = K_s .

For layered cases, the relation between v_i and l_f is calculated as a step-by-step process for the various layers and the average vertical K of the wetted zone, and hence, the final infiltration rate, is the harmonic mean of the K values of the various layers (Bouwer, 1995). If there is a definite perching layer, the height of water above such a layer can be calculated using Darcy's Law. If such a mound rises so high that the top of the capillary fringe gets close to the soil surface, a reduction in the infiltration rate will occur.

In situations where a clogging layer develops at the surface, K will decrease with depth. The 'clogging layer' will be saturated while the underlying material will be unsaturated. The infiltration rate can be calculated by applying Equation 10 to the clogging layer (Bouwer, 1995),

yielding

$$V_i = K_c \frac{(h_o - \psi_c + L_c)}{L_c} \quad [12]$$

where subscript c refers to the clogging layer.

3.3.1.6 Implications of the infiltration theory to recharge schemes

The main issues can be summarised in the following points:

- The porous media must be permeable enough to transmit water. The grain size of soils used for artificial recharge should typically be 0.2- 0.6 mm which is equivalent to fine sand to fine gravel (Dahlstrom and Pedersen, 1996).
- The drier the soil, the greater is the initial rate of water entry because the gradient of matric potential is then of greater magnitude.
- As the length of the wetted zone increases, the effects of matric potential and ponding depth become minimal. Flow occurs as a result of gravitational gradient. The infiltration rate will be equal to the hydraulic conductivity.
- The water table must be sufficiently deep. If the water table gets closer to the surface, the infiltration rate will reduce. Based on international experience, Dahlstrom and Pedersen (1996), suggested a minimum of 8 m of unsaturated zone.

3.3.2 The aquifer's hydraulic suitability to receive recharge water

There are two main physical characteristics which determine whether an aquifer is suitable for accepting, storing and allowing for the recovery of artificially recharged water. They are the aquifer's hydraulic conductivity and storage capacity. A third important factor is the aquifer's hydraulic gradient, which relates mostly to the recovery of the recharged water. The key questions are:

- Will the recharge water be able to flow into the aquifer (hydraulic conductivity)?
- Will the aquifer have sufficient space to accept the water (storage)?
- Will the water be recoverable?

This section discusses the first two questions, and the issue of recovery efficiency is discussed in section 3.6. A few case studies are presented in Chapter 6 and 7, where hydraulic conductivity, transmissivity, storativity and specific yield values for operational or planned artificial recharge schemes are given.

3.3.2.1 Hydraulic conductivity of aquifers

Hydraulic conductivity of a soil or rock is dependent on a variety of physical factors, including porosity, particle size and distribution, shape of particles, arrangement of particles, etc. (Todd, 1980). In fractured rock aquifers it is the density, apertures and roughness of the fractures affect the hydraulic conductivity value. Typical hydraulic conductivity values are given in the Table 3.3.

Table 3.3 Saturated hydraulic conductivity for different kinds of rock (Adapted from Bouwer, 1978)

Unconsolidated Materials	Hydraulic Conductivity (m /day)	Rocks	Hydraulic Conductivity (m /day)
Clay	$10^{-8} - 10^{-2}$	Sandstone	$10^{-3} - 1$
Fine sand	1 - 5	Carbonate rock	$10^{-2} - 1$
Coarse sand	20 - 100	Shale	10^{-7}
Gravel	100 - 1000	Dense solid rock	$<10^{-5}$
Sand and gravel	5 - 100	Fractured or weathered rock	0 - 300
Clay, sand and gravel	$10^{-2} - 10^{-1}$	Volcanic rock	0 - 300

The vertical and horizontal hydraulic conductivities of a soil or rock usually differ. If for example the horizontal K-value was greater than the vertical K-value water would be able to move relatively quickly away from the point of recharge. In the case of infiltration basins, both the vertical and horizontal hydraulic conductivities of the aquifer need to be determined in order to estimate the rate at which the aquifer will be able to receive and transmit the recharge water.

For artificial recharge purposes, aquifers with extremely low hydraulic conductivities are unsatisfactory. In some cases the infiltration rate can be improved by increasing the hydraulic gradient by pumping from abstraction boreholes, or by increasing the pumping pressure in the case of borehole injection schemes.

Conversely, extremely high hydraulic conductivities may result in rapid dispersal of the recharge water, either laterally or to a natural discharge point. As a result, only a limited quantity of water may be able to be stored in the aquifer before natural discharge leads to loss of the recharge water. High hydraulic conductivities also increases the rate of clogging of the aquifer when storm water or waste water with significant suspended solids is used for artificial recharge. Bouwer (1985) suggests that primary aquifers which are best for both storage and treatment of recharge water, are those with a high enough hydraulic conductivity to accept high water infiltration rates, and a sandy loam to fine sand particle size with sufficient clay for sorption of trace elements and filtration.

In fractured rock aquifers groundwater flows preferentially along the fractures. In these cases it is necessary to determine the orientation and significance of the principle fracture system with respect to groundwater flow. The fracture pattern will influence the location of injection and abstraction boreholes, the design and position of recharge basins and the recovery efficiency. Fracture orientations and the hydraulic significance of the fractures can be established by geological mapping, geophysical surveys, exploratory drilling and test pumping.

In many artificial recharge studies, particularly those which involve borehole injection, it may be convenient to express the ability of an aquifer to conduct water across its entire thickness. In this case hydraulic conductivity is expressed as a transmissivity, which is the product of hydraulic conductivity times aquifer thickness. Transmissivity (T) may be defined as the rate at which water of prevailing kinematic viscosity is transmitted through a unit width of aquifer under a unit hydraulic gradient (Todd, 1980). It follows that

$$T = K \cdot b \quad [(m/day)(m) = m^2/day] \quad [13]$$

where K is the hydraulic conductivity and b is the saturated thickness of the aquifer.

Transmissivity can be determined by pump testing and applying the appropriate groundwater flow equation (Kruseman and De Ridder, 1990). Transmissivity values can be obtained from the pumped borehole and observation boreholes.

3.3.2.2 Methods for determining hydraulic conductivity

Solution of the governing equation for water flow in the vadose zone requires estimates or measurements of the saturated hydraulic conductivity. A number of catalogues, which give unsaturated flow properties (including saturated K) for a wide range of soils, are available (Bouwer, 1995). By searching these catalogues for soils of similar textures, the saturated K for a given soil may be estimated.

For field methods, reference is made to ASTM Standard guide D 5126-90 (ASTM, 1990) in which a review of field methods for measuring K in the unsaturated zone is given. The methods reviewed include infiltrometers, double tube test method, permeameter methods, instantaneous profile and crust method. Section 5.1 discusses the use of infiltrometer tests in the design of basin recharge schemes.

A relatively accurate and easy way to measure saturated hydraulic conductivity is through pump testing, which involves the monitoring of water levels in observation boreholes near the pumping borehole. The advantage this method has over other methods, is that it gives an integrated hydraulic conductivity over a sizeable aquifer section. Other methods are based on flow measurements in a laboratory, tracer tests, slug tests and empirical equations based on porosity, grain diameter and shape factor.

Laboratory analysis requires measuring flow through a column of aquifer material under constant

or falling head conditions with a permeameter, and applying Darcy's Law. Obtaining an undisturbed sample which is not affected by the way it is packed into the permeameter can be a problem.

Hydraulic conductivity can also be obtained by measuring the time taken for a tracer to travel between two boreholes. The boreholes need to be in close proximity due to slow groundwater travel times, and the flow direction has to be known. If the aquifer is stratified or fractured, the K-value obtained may be much higher than the average, due to preferential flow paths along the transmissive sections of the aquifer. The decay in concentration of a tracer in a single hole can also be used to determine hydraulic conductivity. Since the value obtained would be related to the hydraulic conditions around the borehole, this value would not necessarily be representative of the entire aquifer. This problem also applies to slug tests, where the change in piezometric surface after the rapid removal of a volume of water or a 'slug' is monitored.

3.3.2.3 Aquifer storage capacity

The main concern with South African aquifers, and in particular, the secondary aquifers, is whether there is sufficient storage space to accept the artificially recharged water. Vegter (pers. comm.) commented that South African aquifers are typically full during or after the natural recharge periods (during or after the rainy seasons), and that it is only during the dry periods, that is, when there is no surplus water available, that the aquifers have space to store additional water. While this may be true for a number of South African aquifers, it may be possible to use dam water or municipal waste water for artificial recharge during the dry periods.

The volume of water which can be artificially recharged to an aquifer is often best determined by assessing the amount of water which has been abstracted from the aquifer, and relating this to water level data. If the hydraulic characteristics and areal extent of the aquifer are well understood, it will be possible to estimate this volume (V)

$$V = A \cdot b \cdot S \quad [14]$$

where A is the area of the aquifer (m²), b is the mean aquifer thickness (m) and S is the storativity or specific yield of the aquifer (dimensionless).

The accuracy of calculating storage, however, is normally only to within an order of magnitude. The area of an aquifer is difficult to determine without detailed geological and geophysical mapping, and an inventory of existing boreholes and their geological logs. Where such information is not available the area of the aquifer could be estimated from geological and topographical maps, and aerial photographs.

As with the area of an aquifer, it is difficult to determine the thickness of an aquifer without a detailed hydrogeological study of the area. Aquifer thickness can be determined from borehole lithological logs and by geophysical methods.

Storativity

Storativity (S) of a saturated confined aquifer is defined as the volume of water released from storage per unit surface area of the aquifer per unit decline in the component of hydraulic head normal to that surface (Kruseman and de Ridder, 1990). It is m^3 of water per m^2 of aquifer per m of head and is therefore dimensionless ($m^3/m^2 \cdot m$). Storativity is determined by test pumping the borehole and monitoring the drawdown in observation boreholes and then applying the appropriate hydraulic equation to solve for S. The appropriate equation for various hydraulic scenarios can be found in Kruseman and De Ridder (1991). Typical storativity values are given in Table 3.4.

Table 3.4: A Qualitative Classification of Hard Rock Storage (Vegter, 1995).

Main storage component of saturated zone	Qualitative indication of mean storage coefficient (order of)	Mean thickness of water-bearing zone
Fractured sedimentary hard rock; open fractures extend to depths of 100 m and more below the water level	<0.001	25 - 50
Fractured sedimentary hard rock within the zone of weathering	<0.001	<10
		10 - 15
		15 - 25
Fractured igneous/crystalline metamorphic rocks within the zone of weathering	<0.001	<10
		10 - 15
		15 - 25
Combination of fractured and decomposed to partially decomposed sedimentary hard rock	0.001 - 0.01	<10
		10 - 15
		15 - 25
Combination of fractured and decomposed to partly decomposed igneous/crystalline metamorphic rock	0.01	<10
		10 - 15
		15 - 25
Fractured dolomite and minor limestone; chert. Dissolution features not prominent	<0.01	10 - 15
Intensely karstified dolomite and minor limestone; chert.	>0.01	15 - 25

Bredenkamp, *et al.* (1995) caution against obtaining an S-value from a single observation borehole in secondary aquifers, as this value appears to decrease with distance from the pumped borehole, and at large distances the S-value may become unrealistically small.

Specific yield

Where the term storativity is used in confined aquifers, specific yield is used in unconfined aquifers. The specific yield of a rock or soil is the ratio of the volume of water that, after saturation, can be drained by gravity from its own volume (Todd, 1980). Like storativity calculations, specific yield is best determined by test pumping.

Values of specific yield depend on grain size, shape and distribution of pores and compaction of the stratum. Fine grained materials yield little water, whereas coarse-grained materials permit a substantial release of water - and therefore serve as better aquifers. In general, specific yields for thick unconsolidated formations tend to fall in the ranges of 0.07 - 0.15 percent (Todd, 1980).

3.3.2.4 Summary

Aquifers which have high hydraulic conductivities and which have high storage capacities are more suitable for receiving additional recharge water than those which have low hydraulic conductivities and low storage capacities (Figure 3.1). However, if the aquifer has a high hydraulic conductivity and a high hydraulic gradient, water will flow rapidly away from the point of recharge and may be difficult to recover. This problem is likely to be greatest in anisotropic, fractured rock aquifers. This issue is dealt with separately in the section on recovery efficiency.

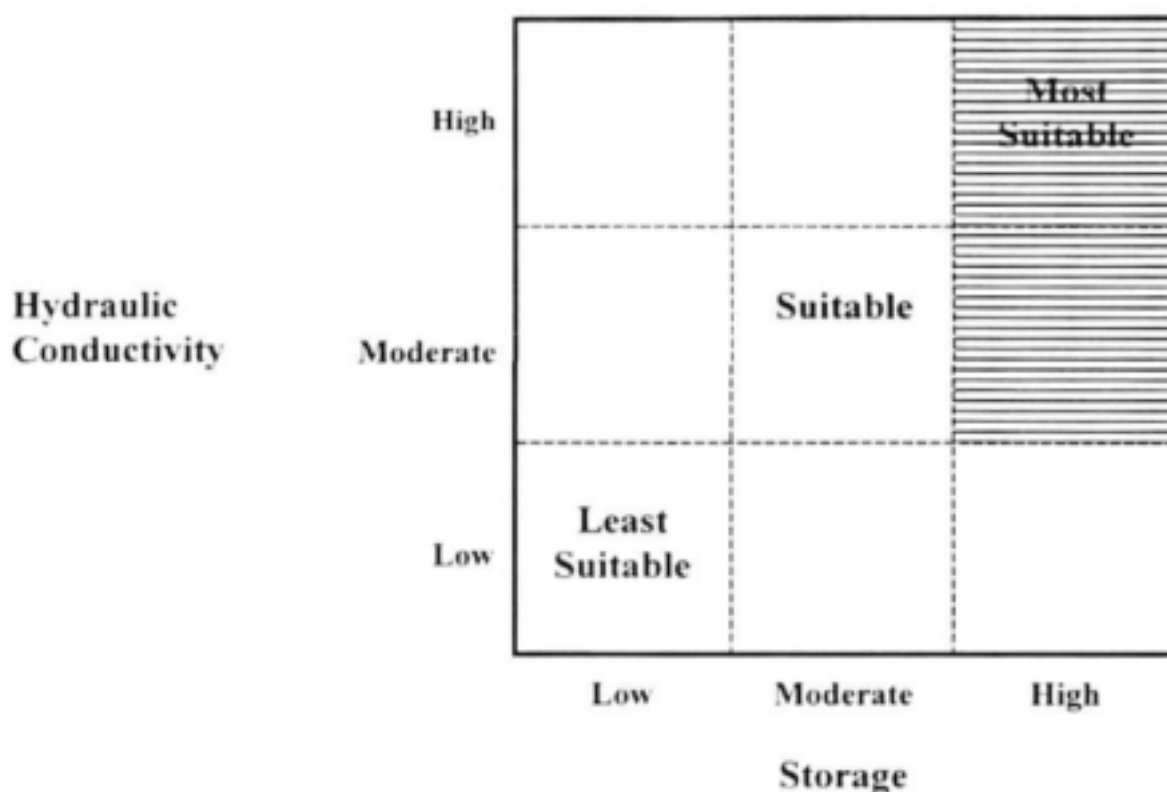


Figure 3.1 Suitability of an aquifer to receive artificially recharged water

3.4 CLOGGING POTENTIAL

Correctly dealing with the phenomenon of clogging of the artificial recharge system plays a decisive role in determining the success or failure of a scheme. Clogging of the system is due to mechanical, physical, chemical and biological processes, as well as a combination of these. It can take place at the infiltration surface, in the unsaturated zone, or in the aquifer itself. In the case of injection it could block the fractures leading away from the borehole. Dealing with the phenomenon needs a thorough understanding of the processes involved and the consequent reversibility or irreversibility of the situation. This section describes common forms of clogging and suggests preventative and remedial measures.

3.4.1 Borehole clogging

Artificial recharge of groundwater usually results in an increased resistance to flow near the well or borehole. This is referred to as clogging or plugging and is defined as a significant increase in injection head for a constant injection rate. Borehole clogging during artificial recharge operations results in a decreasing rate of recharge or the need to continually increase the recharge head to maintain a constant recharge rate (Pyne, 1995). Clogging also has a negative impact on recovery of artificially recharged water, since it increases drawdown during pumping.

Clogging Processes

The processes that are primarily responsible for clogging are:

- i. Air entrapment and gas binding.
- ii. Suspended solids in the recharge water.
- iii. Biological growth of bacteria in the gravel pack and surrounding formation.
- iv. Chemical reactions between recharge water, groundwater and the aquifer material.

Air entrapment

During injection, air bubbles may be entrained by the free fall of the recharge water when the injection pipe ends some distance above the water level. Huisman and Olsthoorn (1983) state that in water at rest, air bubbles with diameters between 0.1 to 10 mm rise at velocities of 0.3 to 0.4 m/sec. Consequently borehole injection with higher flow rates carries air bubbles downward, through the borehole screen openings into the gravel pack and into the surrounding formation where they clog the pore spaces between the grain particles. This results in an increased resistance to flow which in turn decreases the injection capacity.

Air entrapment is characterised by a rapid increase in the resistance to flow, which levels off in a matter of hours. The effects of air entrapment stabilize because the rate at which air bubbles redissolve into the flowing water equalizes with the rate of bubble formation. Figure 3.2 illustrates a typical response of resistance to flow caused by various clogging processes.

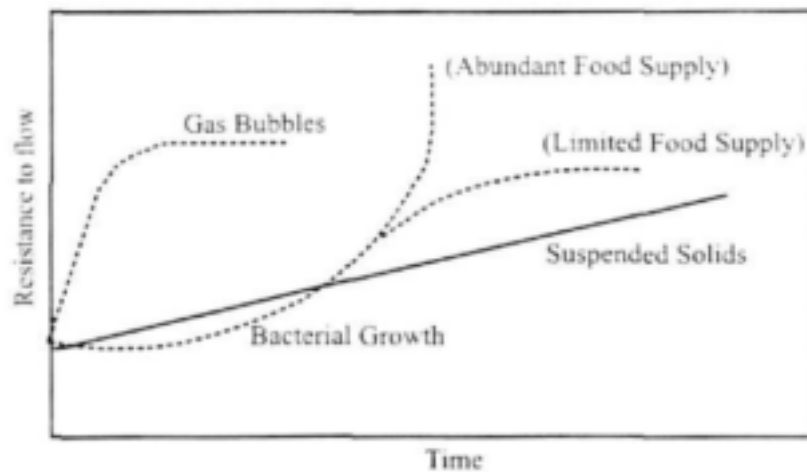


Figure 3.2 Typical clogging processes (after Pyne, 1995).

Air entrapment can be prevented by proper borehole design and operation. The most common method is to maintain positive pressures in the injection pipe and to place the injection pipe some distance below the dynamic water level in the borehole. A flow restricting valve or an orifice can be installed at the lower end of the injection pipe to ensure that positive pressures are maintained. Alternatively the borehole could be designed to be airtight.

Suspended solids in the recharge water

Clogging by suspended solids is a critical issue for virtually all injected recharge waters, unless they meet potable water quality standards or are treated to tertiary treatment standards including ultra-filtration with activated carbon treatment (NRC, 1994). Poorer quality recharge waters, including waters from most natural surface water sources, need a very high degree of pretreatment when being considered for subsurface injection, especially in terms of suspended solids (Bouwer, 1996; Vecchioli *et al.*, 1976). Such pretreatment is equivalent to being treated up to near potable-quality standards (Pyne, 1995).

Injection of water with suspended solids of less than 0.4 mg/l is associated with few problems (Stephens *et al.*, 1996). Clogging problems are normally experienced when recharge waters contain more than 1 mg/l of suspended solids, except maybe for cavernous limestone and fractured rocks (NRC, 1994). Issues relating to clogging using high quality water (chlorinated and treated to potable standards) for recharge are generally not critical (NRC, 1994; Bouwer, 1996).

Suspended solids may cause clogging by accumulating on the borehole wall and by accumulating within the aquifer. If the borehole is constructed with a filter pack, suspended solids will be removed from the recharge water as it flows through the filter pack into the aquifer formation. This filtration process results in the formation of a filter cake which increases resistance to flow near the borehole. This process is similar to the clogging of a membrane filter. A typical filtration curve is presented in Figure 3.3. There are three clogging phases of a membrane filter:

- i. Blocking filtration phase.
- ii. Cake of gel filtration phase.
- iii. Cake filtration with compression phase.

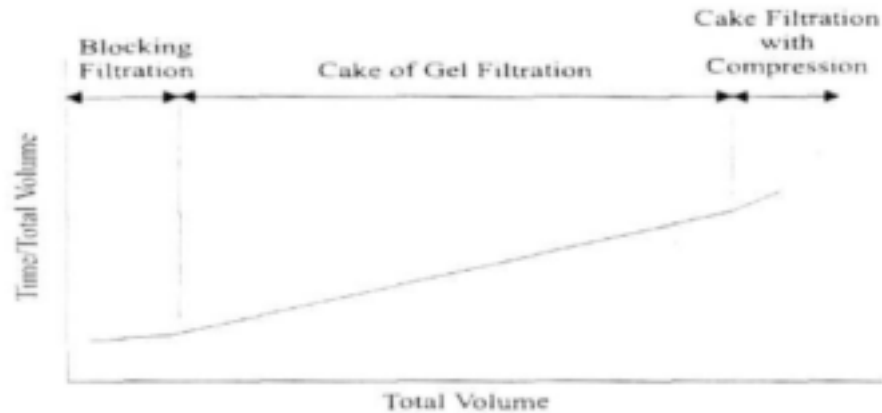


Figure 3.3 Stages of clogging on a membrane filter (after Pyne, 1995).

The physical blocking of pore spaces in the filter medium is characteristic of blocking filtration. The duration of this phase is short and the magnitude of clogging minor compared to the later stages of clogging of membrane filters. The filter pack surrounding the borehole screen may trap larger particles before they reach the borehole wall, thus reducing the long term clogging rate by acting as a coarse pre-filter. It is possible that blocking filtration in the filter pack continues while caking filtration progresses at the borehole wall.

The second phase of cake or gel filtration begins when the layer of filtrate on the filter begins to thicken. The increase in resistance is proportional to the thickness of the filter cake. The onset of cake filtration is indicated by the linear increase of injection head over time while maintaining a constant rate of injection. This phase continues until the filtrate thickness increases enough to allow compression of the filtrate, thus initiating the third phase of clogging.

Cake filtration with compression is characterised by a sharp increase of resistance to flow, which is dependent on the compressibility of the suspended solids. If this stage of clogging occurs at the injection borehole, then continued injection may not be practical due to the associated high clogging rate and the resultant increase in difficulty of redeveloping the borehole. Identifying the onset of this phase of clogging during recharge operations may provide a signal for the redevelopment of the borehole.

Borehole design plays an important part in reducing clogging potential. A screened borehole may be 10 -15 times less efficient than a well completed open hole (Hutchinson, 1993). A gravel packed borehole is more susceptible to clogging than a screened borehole, as the resistance of flow near the borehole increases as the filter cake of suspended material accumulates due to filtration (Gerges, 1996). Keeping nitrate and organic carbon concentrations low when injecting into anoxic aquifers will limit denitrification and the growth of a filter cake.

Biological Growth

Clogging mechanisms as a result of biological growth include an accumulation of impermeable slimes, the development of a mat of dead cells and byproducts, and the dispersion or alteration of colloidal particles in the soil aquifer matrix. The rate of growth of these films is dependent mainly on the quantity of nutrients, the C:N ratio in the recharge water and the temperature of the recharge water (Bouwer, 1985; Pyne, 1995). Although the concentration of nutrients in the recharge water may be low, the process of concentrating suspended solids near the borehole, due to filtration, often provides the substrate needed to foster biological growth.

Biological growth can be prevented by:

- Removing their nutrient supply prior to injection, for instance, by a preceding slow sand filtration.
- Chlorination of recharge waters so that a chlorine residual of between 1 and 5 mg /l always occurs at the bottom of the injection borehole. (Pyne, 1995; Bouwer, 1985).

A pause in recharge operations of more than two days can allow biological growth to form. Pyne (1995) suggests a continuous addition of chlorinated water at a trickle flow rate, between recharge and recovery operations be maintained to curb biological growth. The trickle flow rate should be sufficient to maintain a sufficient chlorine residual to control biological growth. Typical trickle flow rates for disinfection purposes range from 0.1 to 0.3 l/sec.

In some European countries artificial recharge is practised using water with little or no residual chlorine since water treatment includes chlorination followed by dechlorination.

Chemical additions usually rely on dissolving the microbial matter. However since they are more expensive than mechanical methods they tend to be used less often. Acids have been used successfully in carbonate aquifers (Dillon and Pavelic, 1996).

In some geohydrological environments, for example in the Cape Supergroup quartzitic aquifers in the Klein Karoo, clogging of casing and pumps due to iron bacteria is a problem. In order to minimise clogging during borehole injection, the system should be designed so that as little oxygen as possible enters the borehole.

Due to the difficulty of remediating aquifers clogged by biological growth, preventing biological growth from the start of artificial recharge operations is considered to be more effective than periodic removal.

Chemical reactions

Chemical reactions within the aquifer can adversely affect the aquifer's permeability or cause changes in the quality of the recovered water. These chemical and physical changes are a function of:

- Recharge water quality.
- Groundwater quality.
- Aquifer mineralogy.
- Changes in temperature and pressure that occur during recharge or recovery.

The most notable of the possible adverse chemical reactions are:

- The precipitation of calcium carbonate.
- The precipitation of iron and manganese oxy-hydroxides.
- The formation, swelling, or dispersion of clay particles.

The precipitation of *calcium carbonate*:

The precipitation of calcium carbonate within aquifer media is related to the saturation index of the recharge water (Ratray *et al.*, 1996; Pyne, 1995; Treweek, 1985); and the saturation index, is mainly dependent on alkalinity.

Recharge water should be chemically stabilised to help prevent precipitation reactions in an aquifer (DNDE, 1982; Pavelic, 1996). Telfer *et al.* (1996) suggest that saturation indices could be used to estimate the extent or likelihood of precipitation/ dissolution reactions occurring in an aquifer for minerals such as calcium carbonate. Pyne (1995) indicates that stabilisation of recharge water should preferably err on the corrosive side.

As a general guide Figure 3.4 can be used to estimate whether calcium carbonate will precipitate or not (Huisman & Oltshoorn, 1983). Deposition of calcium carbonate occurs when the point representing the compositions of the mixed water lies below the line indicating the chosen n value, where n is the mix ratio of 1 part of water type 1 with n parts of water type 2.

For example, assuming one part of recharge water (water type one) is mixed with ten parts of groundwater (water type two). The reference line is the $n = 10$ line. In order to establish whether the deposition of calcium carbonate will occur, plot the concentration ratios of HCO_3 and CO_2 of the recharge water and the groundwater. If the plot lies below the $n=10$ line, then deposition of calcium carbonate will occur. If however the plot lies above the $n=10$ line then deposition of calcium carbonate will not occur.

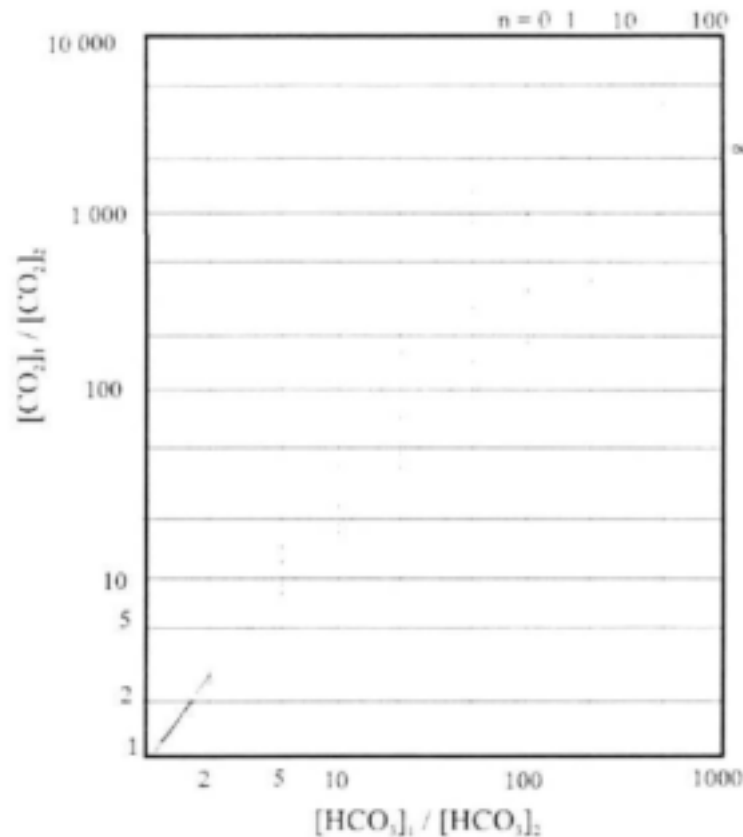


Figure 3.4 Calcium carbonate suitability for mixing 1 part of water type 1 with n parts of water type 2 (after Huisman & Oltshoorn, 1983)

The precipitation of *iron and manganese oxy-hydroxides*:

Recharge water containing dissolved oxygen can cause chemical clogging if mixed with aquifers containing dissolved iron. Recharge water usually contains more oxygen than the aquifer water. If siderite (ferrous carbonate) is present in an aquifer, iron dissolution can occur in high concentrations (up to 13 mg/l) (Pyne, 1995). If pyrite (ferrous sulphide) is present, iron dissolution can occur, but in lower concentrations (Pyne, 1995). Dissolved iron in the recharge water increases the potential for ferric hydroxide, iron oxy-hydroxide and manganese oxy-hydroxide precipitation, and consequent clogging problems (Pyne, 1995; Treweek, 1985; DNDE, 1982).

Chemical clogging issues relating to injection with waters containing *sulphates*:

Anoxic conditions stimulate the growth of sulphate-reducing bacteria which present potential clogging problems such as the precipitation of iron sulphide (Gerges, Sibenaler and Howles, 1996; Rattray *et al.*, 1996).

The formation, swelling, or dispersion of *clay particles*:

The adverse effects between the recharge water and the aquifer material mainly concern the swelling and dispersion of clay particles present as silt in the aquifer. Clay swelling and reduction of pore space occur when the ratio between $\text{Ca}^{2+} + \text{Mg}^{2+}$ and $\text{Na}^+ + \text{K}^+$ is reduced, or the ionic strength is lowered. This can be prevented by choosing another type of recharge water or by adding CaCl_2 to the recharge water. It may further be reduced by a pre-flush with water containing high amounts of CaCl_2 .

Chemical clogging can also occur when low Total Dissolved Solids (TDS) water containing calcium bicarbonate ions is injected into low TDS groundwater containing mainly sodium chloride ions. The introduced water tends to become sodium bicarbonate water, which stimulates the swelling and dispersion of clays in the aquifer media, with a resultant potential for clogging pores in the aquifer media (Rattray *et al.*, 1996).

Methods for measuring borehole clogging

Three methods have been developed for evaluating borehole clogging:

- i. Specific time of injection method
- ii. Difference in water level rise method
- iii. Observed versus theoretical water level rise method

i. Specific time of injection method

This method is useful particularly when an observation borehole is not available, since only the water level data of the injection borehole is used for analysis. The theory behind this method is that for a constant recharge rate, the rise in water level since the start of recharge would be the same, assuming no clogging has occurred. Therefore, a comparison of the water level measurements of two different recharge runs taken at the same recharge rate and time interval indicate whether clogging has occurred. A time interval of 2 to 4 hours since the start of recharge is usually used so that the water level measurement is taken when the rate of water level change is reduced.

A drawback of this method is the lack of control over the recharge rates. Rising water levels in the borehole can affect the hydraulics of the recharge system and thereby change the recharge rates. One way of overcoming this is to conduct a step injection test over a range of recharge rates prior to the production injection runs. A similar step injection test carried out at a later stage can be compared to the initial results to determine the magnitude of clogging.

ii. Difference in water level rise method

This method uses data from the injection borehole as well as one or more observation boreholes. The accuracy of this method is dependant on the assumption that the recharge rate is constant and

that the injection borehole and the observation borehole are screened over the same depth. When the flow regime in the aquifer has reached steady state, the difference in water levels between the injection borehole and the observation borehole, theoretically, will remain constant. An increasing difference in water levels is an indication of clogging.

iii. Observed versus theoretical water level rise method

The rise in water level in an injection borehole is a combination of aquifer response and borehole losses. It is assumed that for a constant recharge rate, borehole losses should remain constant, and therefore any water level rise in the borehole, without clogging, would be due solely to aquifer response. Therefore using estimates of aquifer parameters, transmissivity and storativity or specific yield, the water level response in the aquifer is calculated and compared to the observed change in water level in the borehole. The difference between the calculated water level and the observed water level is presumably due to clogging.

3.4.2 Basin Clogging

Clogging of the floor and sides of an artificial recharge basin can occur through one or more of the following processes:

- i. Clogging by suspended particles
- ii. Chemical clogging
- iii. Swelling of clays
- iv. Biological clogging
- v. Air entrapment

These have been discussed in the section on borehole clogging. This section highlights some of the important clogging issues with respect to infiltration basins.

There are three *physical factors* that control clogging by suspended particles in infiltration basins:

- The concentration of suspended particles in the recharge water or the turbidity of the recharge water.
- The particle size distribution of the soil in the recharge basin.
- The porosity of the soil.

Suspended particles in the recharge water are filtered by the soil in the recharge basin. During the initial stage of filtration up to 70% of the suspended particles may flow through the filter media (Blair, 1970). The larger particles are intercepted at the surface and with time this increases the filtration capacity of the porous media. This leads to the formation of a filter cake on the floor and sides of the basin which results either in a decrease of the rate of infiltration or no infiltration taking place at all.

Biological clogging co-exists with physical clogging. It occurs as a result of biological growth and the accumulation of by-products resulting from the decomposition of biological growth. The

development of a biological film on the soil surface is the result of microorganisms which establish themselves on soil grains using nutrients such as dissolved organic carbon (DOC) and ammonia to establish colonies which build up in layers on the soil and clog the soil pores (Bouwer, 1985). Biological growth reduces the size of the pore spaces and the by-products of biological degradation accumulate and reduce the size of the interstices eventually resulting in permanent clogging.

Algae suspended in recharge basin water promotes rapid clogging of the topsoil layers (Bouwer, 1985). Die-off of suspended algae and other vegetation promotes anaerobic conditions suitable for microbial growth. Consequently the rate of clogging increases rapidly (NRC, 1994; Okubo & Matsumoto, 1983; Bouwer, 1985).

Vigneswaran *et al.* (1987) conducted laboratory experiments to study the physical and biological clogging process during artificial recharge by considering factors such as effective filtration rate, media size and depth, and inundation period. The study revealed that clogging occurs in three stages:

The first stage is characterised by a rapid decrease in the infiltration rate due to the rapid decrease in porosity by suspended particle retention and microbial growth in the sand. This is pronounced in the top few centimetres of the filter bed. The removal of suspended solids is more efficient with a finer filter medium and a slower infiltration rate than with coarse grained filter media. However with coarse grained filter media, filtration takes place over a much greater depth. The onset of clogging will take much longer but will be virtually impossible to remove once it has set in. Thus coarse grained or fractured surfaces should always be topped with a layer of fine to medium grained sand.

The second stage is characterised by a slower decrease followed by a slight increase in the infiltration rate. This is due to the combined effects of inhibited microbial growth resulting from limited dissolved oxygen supply and continued suspended solid deposition.

The final stage is characterised by a gradual decrease in the infiltration rate until total clogging sets in. The study also found that the rate of decrease in the infiltration rate was faster when higher initial infiltration rates were applied.

Clogging can be prevented or reduced by:

- i. Treating the recharge water to remove turbidity.
- ii. Chlorinating the recharge water to prevent microbial growth.
- iii. Mechanical treatment of the soil by plowing or harrowing to increase the porosity.
- iv. Covering the spreading basin with a graded soil layer.
- v. The use of operational procedures, in particular a rotational system of spreading and subsequent drying. After drying the clogging matter falls apart and is blown away by the wind.

When clogging has taken place, cleaning of the basin is necessary. This is done simply by draining the recharge basin, allowing the filter cake to dry and mechanically scraping and removing the filtrate from the basin.

3.4.3 Summary of clogging prevention and redevelopment

Clogging can be problematic and expensive, but can be controlled by appropriate pretreatment of the recharge water and maintenance of the injection borehole or infiltration basin. Table 3.5 summarises the various forms of clogging, means of minimising or preventing clogging and methods of redevelopment.

Table 3.5 Summary of forms of clogging, means of minimisation, prevention and redevelopment (based on Dillon and Pavelic, 1996).

Form of clogging	Minimisation/prevention	Redevelopment
Suspended solids	Borehole injection: minimise suspended solids, install borehole screens, follow correct borehole completion procedures. Infiltration: minimise suspended solids, mechanical treatment of soil to increase porosity, graded soil layer on the basin, rotational system of spreading and drying.	Borehole injection: pumping, surging and jetting. Infiltration: drying and scraping the basin floor.
Microbial growth	Borehole injection: minimise organic matter, chlorination of recharge water, maintain chlorine residual. Infiltration: chlorination of recharge water.	Borehole injection: pumping, surging and jetting as well as disinfection and acids. Infiltration: drying and scraping the basin floor.
Chemical precipitation	Borehole injection and infiltration: recharge waters compatible with groundwater.	Change pH.
Clay swelling and dispersion	Borehole injection and infiltration: low clay aquifers (and unsaturated zones).	Add flocculants.
Air entrapment	Borehole injection: avoid cascading, positive pressure intake, high pressure feed, borehole designed to be airtight.	Pumping, surging and jetting.
Gas binding	Borehole injection: prevent denitrification in anoxic aquifers by disinfection, nitrate removal.	Pumping, surging and jetting.

3.5 MODELLING ARTIFICIAL RECHARGE TO AQUIFERS

Mathematical models can be used to quantify artificial recharge processes, thereby optimizing recharge rates. There are a wide variety of mathematical models which are designed to represent actual physical processes, including the flow mechanisms. Once these mechanisms have been defined, calculations can be carried out to estimate artificial recharge. Before identifying the probable flow mechanisms it is essential to examine the field evidence carefully. The ultimate goal of any such process is usually to convey the maximum volume of water to groundwater storage as efficiently as possible.

Mathematical models are used to determine best management options for artificial recharge for a wide variety of water supply projects. They are also used for numerous groundwater remediation projects, where contaminated groundwater is extracted from the aquifer, treated and then returned to the contaminated aquifer (Alaa and Peralta, 1994).

3.5.1 Modelling approaches

A standard groundwater flow model can be used for modelling *borehole injection*. Most of these models are based on the saturated groundwater flow equation,

$$S_0 D_t \phi(x,t) = \nabla \cdot [K \nabla \phi(x,t)] + f(x,t) \quad [15]$$

where S_0 is the specific storativity, ϕ the piezometric head, K the hydraulic conductivity and f is the source/sink term, which in this case is the borehole injection rate.

Modelling borehole injection usually involves changing the sign of the abstraction rate, that is, making it positive if the model considers abstraction as a negative amount or vice versa. However, it is important to note that in most groundwater flow modelling packages there is a mathematical discontinuity at the point of abstraction/injection. Thus a solution should not be sought at this point. The solution becomes more accurate the further one moves away from the borehole.

Spreading basin recharge, which is often used for unconfined aquifers, is slightly more complex. The principles and mathematics of infiltration into a porous medium are discussed in Section 3.3. Usually a three dimensional groundwater flow model has to be utilized. A surface water / river option combined with a leakage coefficient can be used to simulate the behaviour of a spreading basin. The combination of a leakage coefficient with a surface water / river option is the same as using a semi permeable boundary condition.

A problem does arise when the spreading basin is underlain by a thin clay layer. An additional layer will then have to be included in the model. To determine the hydraulic characteristics of this layer is a difficult task. It is, however, estimated that the hydraulic conductivity of this layer will be between 0.1 and 0.001 m/d (Van Tonder, 1997).

3.5.2 Data Requirements

In most cases all data required for normal groundwater flow models are also needed for an artificial recharge model including: aquifer dimensions, water level data, aquifer parameters (storage coefficients and permeability values), recharge values and boundary conditions. In addition to these, the volume of water being recharged and the period of recharging must be recorded. In the case where spreading basins are used leakage coefficients need to be calculated.

Artificial recharge usually takes place in cycles. In other words the aquifer is recharged for a period of time, followed by a period of no recharge allowing infiltration to take place. During the wetted period of the cycle, recharge rates and volumes can be influenced by a number of phenomena such as

- clogging which can be caused by silt and clay depositions and/or biological activities (normal groundwater flow models will not be able to account for this).
- mounding caused by hydrogeological parameters.

It is important to note that both factors can only contribute to the a long term decrease in recharge rates as the recharge cycle increases. However it is difficult to obtain numerical data for the phenomena mentioned above (Dickenson *et al.*, 1994).

3.5.3 Commercially Available Software

With the software MODFLOW, the user can either use the borehole injection method or the surface water/river option with a coefficient for leakage. This option is generally available in most three dimensional groundwater modelling packages for example SUTRA, a saturated unsaturated transport numerical modelling package.

There are numerous packages that calculate infiltration from seepage basins (Wilson *et al.*, 1995; Scientific Software Group, 1997). Examples of such packages are :

- FASTSEEP and SEEP/W which are finite element programmes developed specifically for simulating seepage from spreading basins.
- SWIM which is based on numerical techniques to solve the soil-water flow equation. It is one of the only packages where users can simulate infiltration, redistribution, deep drainage, evapotranspiration, transient surface-water storage and runoff.
- Software, such as CSUPAWE that calculate the solution of groundwater mounds forming beneath recharge basins are based on the linearized unconfined flow equations. Such solutions are very useful for quick mound calculations, but unsuitable for providing quantitative results (Akan, 1994).

SEEP/W is the most popular of the software packages mentioned above as it can simulate both infiltration due to spreading basins and recharge by means of injection boreholes for saturated

and unsaturated conditions.

3.5.4 Mass transport modelling

Hydrochemical changes related to artificial groundwater recharge can be simulated by mass transport models. Such models are based on the mass transport equation, which in a simplified form can be expressed as

$$\rho\theta D_c c + \rho_b D_s s + \rho q \cdot \nabla C = \nabla \cdot [\rho\theta \mathbf{D}_h \nabla C] - \lambda(\theta\rho C + \rho_b s) + \rho C(f_0 - f) \quad [16]$$

where ρ is the density of the solute, θ is the volumetric moisture content, c and C are the volumetric and mass concentrations respectively, ρ_b is the bulk density, s the fraction of dissolved solids taking place in the reaction, q is Darcy's velocity, \mathbf{D}_h the hydrodynamic dispersion coefficient and λ is the decay constant. The term $(f_0 - f)$ refers to the source or sink.

Mass transport models are linked to individual hydraulic models, for example MT3D is associated with MODFLOW. Most of these models are capable of modelling advection in complex steady-state and transient flow fields, anisotropic dispersion, first order decay and production reactions, and linear and nonlinear sorption.

Examples of commercially available software, are :

- MT3D, which is linked to MODFLOW
- CTRAN/W, which is linked to SEEP/W
- SWIM, which was mentioned in section 3.5.3 above, has a built-in mass transport module.

3.5.5 Discussion

Models are useful tools for the management of artificial recharge. However it is important to note that there is no ideal model. Therefore, the most suitable one has to be selected for the particular investigation. A brief literature review indicated that most of the artificial recharge modelling has been done using ordinary groundwater flow models and in particular MODFLOW. For mass transport modelling the associated package MT3D would be required. However, for basin recharge it is suggested that SEEP/W would be the best option as it is generally considered to be one of the better packages for the modelling of artificial recharge by means of infiltration basins. In this case CTRAN/W would be the recommended mass transport model.

3.6 RECOVERY EFFICIENCY

This section deals mainly with recovery efficiency from aquifer storage and recovery (ASR) systems, since it is in these systems that recovery efficiency is often quantified. The issue of recovery efficiency is usually only of concern when the quality of the recharge water and the

native groundwater are vastly different. Pyne (1995) states that "recovery efficiency usually has little significance where both stored water and native water are potable. In such situations the main concerns are usually aquifer plugging (clogging) and (borehole) development frequency". In this section, the main factors which affect recovery efficiency are introduced, and recommendations for maximising recovery efficiency are presented.

3.6.1 Recovery efficiency for ASR systems

ASR recovery efficiency is defined as follows: *the percentage of water volume stored that is subsequently recovered while meeting a target water quality criterion* (Pyne, 1994). The water quality criterion is typically total dissolved solids (TDS), electrical conductivity (EC) or chloride concentration.

For example: if the target established is 500 mg/L TDS and 1 000 m³ of water is injected (at 300 mg/L) into a brackish aquifer and 800 m³ is abstracted before the target is reached then the recovery efficiency is 80%.

The limit set is usually a higher salinity than the injected water. This reflects typical current ASR practice of injecting fresh waste water into saline or brackish coastal aquifers. This means that what is being abstracted is not only the injected water but a mixture of injected and native water until the proportion of native water becomes unacceptable.

The range of recovery efficiency is quoted by Martin (1996) as being up to 70%, however, Pyne (1994) states that most schemes can be developed to 100%. The exceptions being very transmissive, highly saline aquifers which reached 70 - 80%.

These figures should be considered in the context of the efficiency of surface storage facilities, like impoundments and dams, which lose significant volumes to unintentional seepage and evaporation. In arid or semi-arid areas surface storage efficiency may be significantly less than 50% (Pyne, 1994).

3.6.2 Factors controlling recovery efficiency for ASR schemes

There are many factors controlling recovery efficiency, as listed below. Several of these are interdependent. Recovery efficiency is mainly controlled by the aquifer characteristics of the receiving zone around the borehole, however, there are some operational factors that influence efficiency. The aquifer characteristics which affect recovery efficiency include:

- Aquifer configuration (dip)
- Hydrodynamic dispersion
- Permeability (degree of permeability and relationship between primary and secondary permeability)
- Hydraulic gradient
- Concentration gradient between the injected water and the native aquifer water
- Heterogeneity

- Anisotropy
- Aquifer thickness

When less dense, fresher water is injected into the upper section of a confined *dipping aquifer* with a very low hydraulic gradient it will tend to rise and displace up-dip along the permeable zone. This will result in greater mixing and a lower recovery efficiency.

Injected water enters the aquifer under a hydraulic head and displaces some of the native water which is already present in the aquifer. A zone of mixing between the two waters, also called a transition zone or a buffer zone, is formed around the advective front (the leading edge of the injected water). As injection continues the advective front and the transition zone are displaced away from the borehole (in the case of borehole injection) and further into the aquifer (Martin, 1996). The volume of water in the transition zone increases as the advective front advances due to *hydrodynamic dispersion*.

Hydrodynamic dispersion describes the changes in concentration of a solute in flowing groundwater and includes *molecular diffusion* and *mechanical dispersion* (Fetter, 1993). These are the processes that result in the mixing of the injected and native waters and result in a dilution effect. They are illustrated on a microscopic scale in Figure 3.5. It is necessary to understand how they affect the injected water because the degree of mixing and consequent size of the transition zone have a significant effect on recovery efficiency.

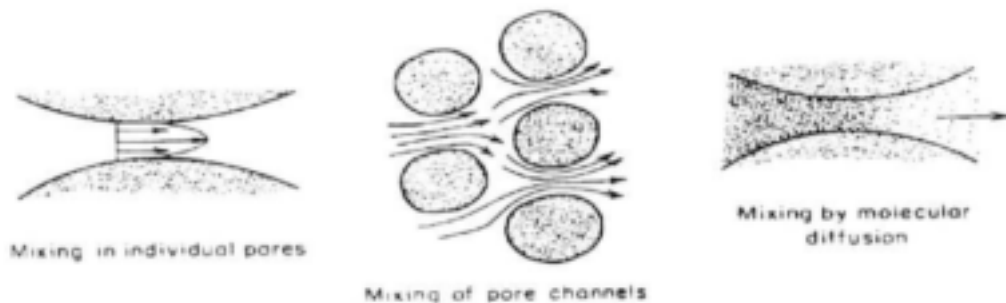
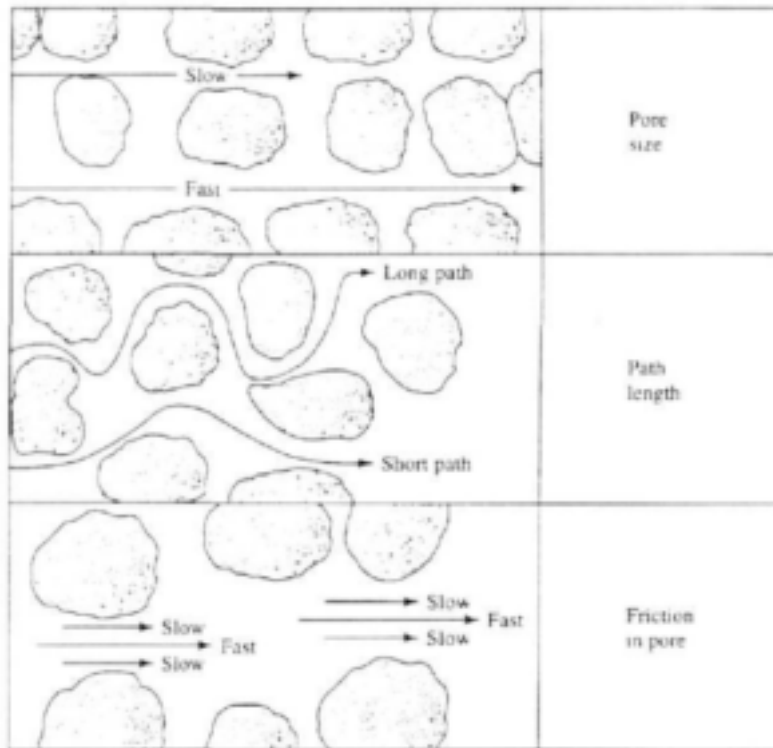


Figure 3.5 Processes of hydrodynamic dispersion on a microscopic scale (after Freeze and Cherry, 1979)

The process of *molecular diffusion* is controlled by the *concentration gradient* between the two waters, in addition to the permeability of the aquifer. Molecular diffusion will occur even when there is no flow in an aquifer. This will be the dominant process in a permeable saline aquifer with a low hydraulic gradient or no flow. Merritt (1993) suggests that recovery efficiency is typically lowest for highly permeable aquifers with saline native water.

Mechanical dispersion reflects the actual flow paths taken by the injected water. These have varying degrees of tortuosity and are dependent on the size of the individual pore throats, the interconnectedness of the pores, the velocity of flow and anisotropy (Figures 3.6 and 3.7). Coefficients of mechanical dispersion (length²/time) may be calculated for different aquifer settings.



NB. Tortuosity = Longpath / Shortpath

Figures 3.6 Factors causing longitudinal dispersion at the scale of individual pores (after Fetter, 1993)

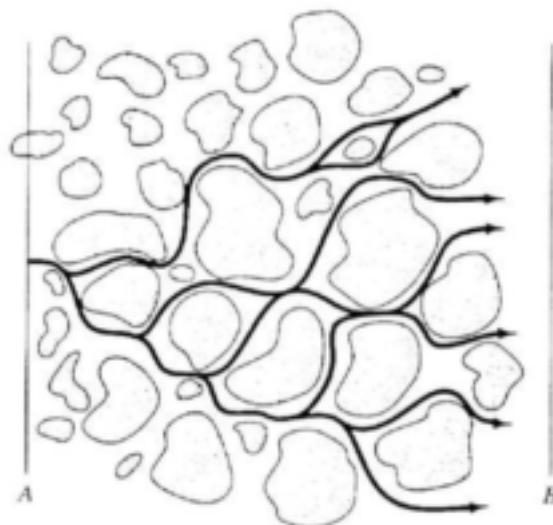


Figure 3.7 Flowpaths in a porous medium that cause lateral hydrodynamic dispersion (after Fetter, 1993)

Hydrodynamic dispersion is therefore controlled by the pore geometry (which affects permeability and tortuosity), the rate of flow and the rate of diffusion. Mass transfer flow modelling can help to indicate the degree of hydrodynamic dispersion expected at different sites if sufficient data are available on the nature of the controlling factors and their degree of variation. However, it is accepted to be very difficult to predict accurately. Usually only a good estimate can be made which should then be confirmed with field trials.

The degree and type of aquifer *permeability* influences the advancement of the advective front and the hydrodynamic dispersion. Dual permeability aquifers, like many of the fractured rock aquifers of South Africa, typically have a lower recovery efficiency (Figure 3.8). A comparison of the recovery efficiency of dual permeability fissured limestone and sandstone (primary permeability) is illustrated in the figure below.

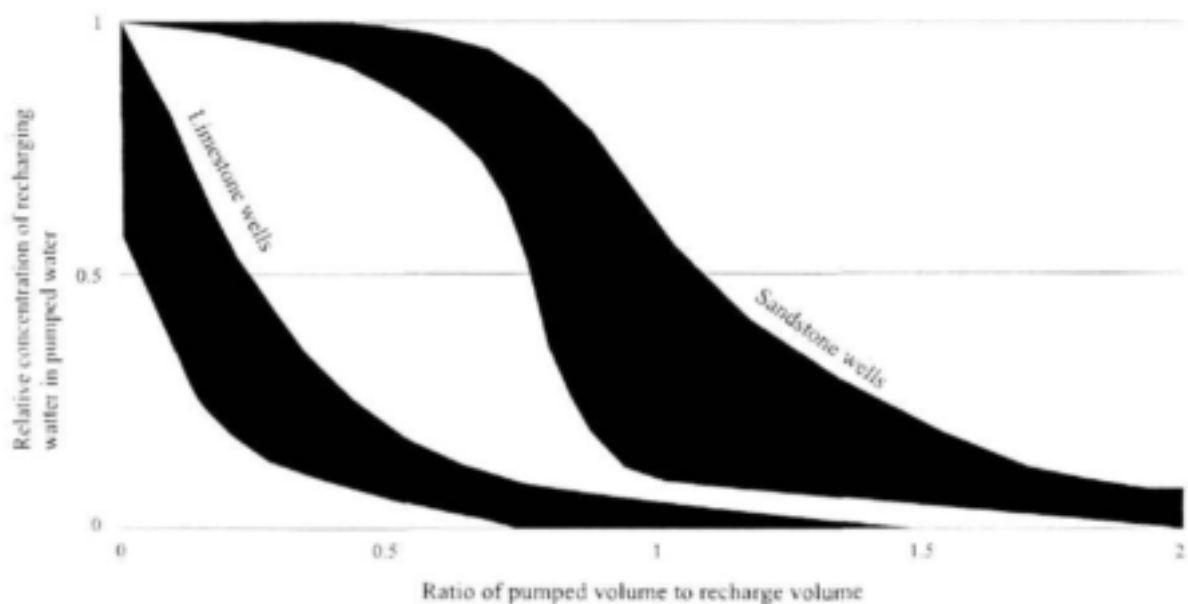


Figure 3.8 Typical dilution curves showing recovery efficiency in a dual permeability limestone and a primary permeability sandstone (after Harpaz, 1971)

The lower recovery efficiency in fractured hard rocks and limestones is mainly due to sustained diffusion from native water contained in primary permeability and immobile fluid zones around the edges of the fracture openings (Figure 3.9). This continuing diffusion extends the number of cycles required to flush out an injection zone.

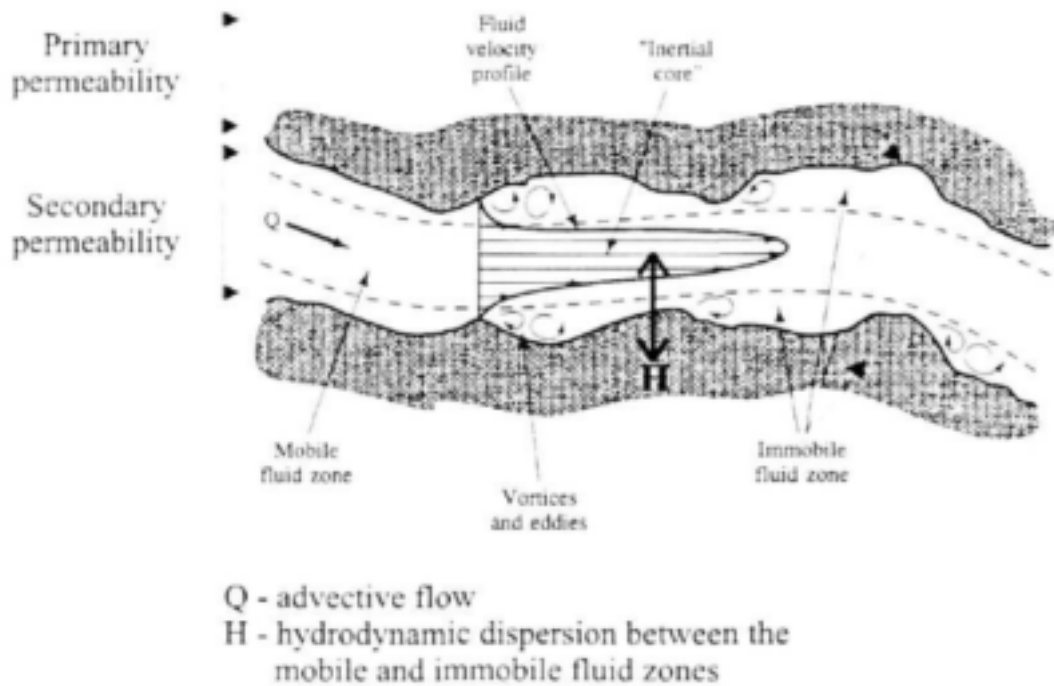
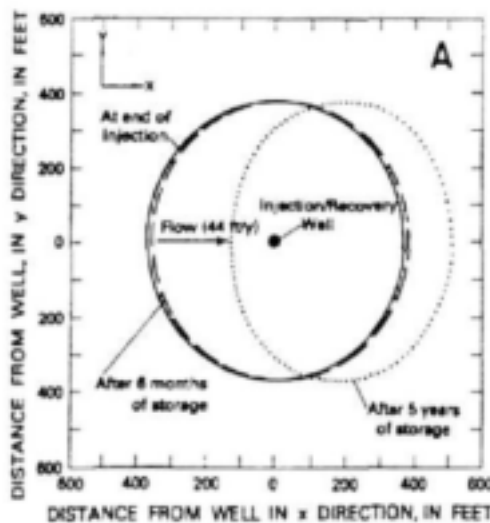


Figure 3.9 Zone of mobile and immobile water in a fracture (after Fetter, 1993)

A significant *hydraulic gradient* in the aquifer may displace the injected water away from the borehole and reduce recovery efficiency (Figure 3.10). This should be taken into account for longer periods of storage.



Recovery Efficiency

Immediate Recovery	- 61%
6 months storage	- 60%
5 year storage	- 22%

NB: 44 ft/yr = 13.4 m/yr

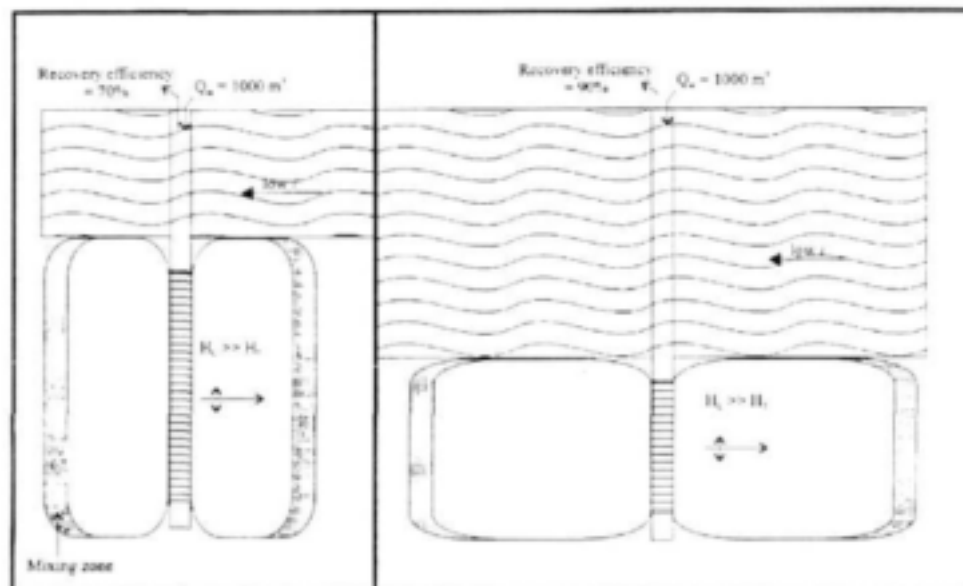
Figure 3.10 Diagrams showing the effect of regional flow upon the recoverability of potable water (after Merritt, 1986)

If there is a significant *salinity contrast* between the injected and native waters then there is a greater risk of buoyancy stratification. If the screens are not in the upper section of the aquifer, fresher, less dense water will rise up through the aquifer and this will result in lower recovery efficiency.

In *heterogenous aquifers* a greater degree of mixing can be expected. The flow away from the borehole can no longer be envisaged as a piston-like displacement because injected water will travel faster in the more permeable zones (Martin, 1996). This means the volume of water making up the leading edge of the advective front is greater and more injected water is lost to mixing.

Heterogeneity combined with *anisotropy* will increase this effect further. Anisotropy (different rates of flow in different directions) is problematic in borehole injection systems if vertical permeability is greater than horizontal permeability. Again, this will result in a larger volume of water in the transition zone.

The transition zone is comprised of a larger volume of injected water when water is injected into a greater *thickness of the aquifer* (Figure 3.11). This is because longitudinal dispersion (radially away from the borehole in the direction of flow) is usually greater than transverse dispersion.



- Q_m - Volume of injected water (the same in both cases).
- i - Regional groundwater gradient
- H_T - Transverse hydrodynamic dispersion
- H_L - Longitudinal hydrodynamic dispersion

Figure 3.11 Mixing zones in a thin and thick aquifer

3.6.3 Problems with predicting recovery efficiency

The complicated relationships of factors controlling recovery efficiency and their natural variation mean that, in many cases, recovery efficiency is difficult to predict and model with the data usually available following a site characterisation.

Hydrodynamic dispersion, for example, is very difficult to quantify. Field experiments show that dispersion in nature is often much greater than is shown in the laboratory (Peters, 1983). In addition it is scale dependent, so not only is predicting through numerical modelling difficult, but even pilot scale testing may give only an approximate indication.

Recovery efficiency is sensitive to changes in abstraction rates and times of storage. It will usually increase for the first 3 to 6 cycles before stabilising (if the cycles are the same duration, rate and water quality) (Pyne, 1994).

3.6.4 Design and management factors which affect recovery efficiency in ASR schemes

Design and management factors affect the quality of the water recovered in ASR schemes, and this (water quality), forms the basis on which recovery efficiency is defined.

The following operational characteristics influence ASR recovery efficiency:

- Borehole siting
- Borehole design - screen position and total screened thickness
- Rates of injection and abstraction
- Trickle flow injection
- Volume injected
- Storage time
- Target limit
- Successive cycles

The borehole should be *sited*, after exploration drilling and geophysics, in the part of the aquifer that appears to have the optimal hydrodynamic characteristics (as outlined above). [In terms of surface recharge schemes, such as basin infiltration incorporating Soil aquifer treatment, the borehole should be sited at a sufficient distance from the area of infiltration for natural attenuation of contaminants to occur.]

Borehole design is particularly important where there is a significant salinity (and therefore density) contrast between the injected and native water. Injected fresher water will tend to rise through the aquifer therefore screens should be constructed in the upper sections of the aquifer to minimise mixing.

The transition zone is comprised of a larger volume of injected water when a greater *thickness of the aquifer* is screened. Therefore the screened length should be minimised (within the constraints of borehole hydraulics for the optimal injection/abstraction rate) to improve recovery efficiency.

The impact of the *rates of injection and abstraction* is often site specific and is not always fully understood. The optimal regime for many ASR sites is to keep abstraction and injection rates approximately similar (Martin, 1996). In some instances, particularly highly saline aquifers, recovery efficiency is improved slightly with faster rates of injection (Merritt, 1986). Slower rates of abstraction minimise the risk of native saline water breaking through (Martin, 1994). More mixing will be caused by a greater drop in the pressure head at the borehole.

Increasing the *volume of water* injected results in a reasonable increase in recovery efficiency for smaller volumes of water (less than 1 500 m³/day) and a less significant increase for larger volumes of water (greater than 4 000 m³/day) (Merritt, 1986).

More mixing will occur if the injected water is *stored for a longer period* of time and recovery efficiency will be reduced. This will be exacerbated if the native water is significantly more saline, if the aquifer is highly permeable and if there is a hydraulic gradient displacing the 'bubble' of injected water away from the borehole. Losses due to diffusion during storage may be offset by maintaining a low rate of injection called *trickle flow* (Pyne, 1994).

Obviously the water quality *limit* set will determine the recovery efficiency. If the average salinity of the injected and native waters is more than double the target limit then recovery efficiency tends to be low (Martin, 1996).

Once a transition or buffer zone is established between the injected and native waters, less mixing occurs and the target limit is reached after abstracting greater volumes of water. Recovery efficiency in ASR schemes tends to increase with each *successive cycle*, but *only if abstraction is stopped when the target limit is reached*.

3.6.5 Methods for maximising recovery efficiency in ASR schemes

Recovery efficiency may be maximised for a given aquifer setting if the following steps are taken:

- Optimal siting of the ASR borehole(s) in relation to aquifer thickness, native water quality, etc. The operator should be confident that the receiving zone is not intercepted or influenced by other abstractors or points of discharge.
- In brackish to saline aquifers, ensure that the upper sections of the aquifer are screened to minimise losses due to buoyancy stratification.
- Maintain sufficient injection rate. Slower injection may result in lower efficiency due to greater mixing.
- If water is available, a higher recovery efficiency will be achieved more quickly if a large volume is injected in the first few cycles. This will establish a transition zone further from the borehole and protect subsequently injected water.

- Do not exceed the water quality target level during abstraction. This will ensure that the transition zone is maintained.
- Do not abstract at a rate that is high enough to cause saline water breakthrough.
- Minimise storage time to minimise mixing and losses due to advective flow away from the borehole(s).
- During the storage period in saline aquifers try and maintain a low (trickle) inflow to counter buoyancy stratification.
- If operating an ASR wellfield, rather than a single borehole, the most efficient regime in most scenarios is to make the abstraction regime the mirror image of the injection regime so that the flow patterns are the same but reversed (Merritt, 1986). This minimises mixing. In other words, abstract at the same rate (or slightly slower) from the same boreholes used to inject.

It is good management practice to keep a good record of the water quality of abstracted water, particularly during the first cycle. The first cycle cannot be repeated in the short to medium term once water of a different quality has been injected. These data will give a good indication of the dispersion characteristics of the aquifer which will be important for future management.

Chapter 4

SOCIO-ECONOMIC ISSUES

4.1 ECONOMIC ISSUES

The cost effectiveness of an artificial recharge scheme is usually a key consideration. Artificial recharge may be one of many options for augmenting current water supplies. It is important to evaluate all options on the same basis if the most cost effective option is to be identified. In certain areas where the cost of supplying potable water is high, it may be worth considering dual reticulation systems. Artificially recharged water of poorer quality than treated domestic water could be used for certain non consumptive uses such as irrigation. A reliable water source for artificial recharge in this case is treated municipal waste water.

The economic analysis of artificial recharge operations should include an estimate of:

- i. Annual operation and maintenance costs.
- ii. Loan repayment of capital investment.

Pyne (1995) states that aquifer storage and recovery (ASR) operations are invariably cost-effective when compared with conventional water supply alternatives involving development of new water sources. Vaux (1985) states that there are two cases when it may be advantageous to devote waste water for artificial recharge. Firstly, where percolation results in significant improvements in water quality, artificial recharge may represent a cost-effective way of upgrading water quality. Secondly, where the pattern of water demand is such that storage of water is required, utilization of aquifer storage capacity will generally be preferable to construction of expensive new surface storage facilities. Unused aquifer storage capacity can be developed at a significantly lower cost than surface storage facilities, and without the adverse environmental consequences frequently associated with surface storage.

Often the overall costs of artificial recharge operations are less than half the capital cost of conventional water supply alternatives, especially those involving development of new reservoirs, treatment facilities or extensive pipelines (National Research Council, 1994). A cost analysis of water supply alternatives for Kenhardt in the Northern Cape revealed that the capital cost of an artificial recharge scheme was only 25% of a bulk water supply scheme involving an extensive pipeline (Table 4.1). A study of six ASR facilities in the United States revealed that the capital cost is generally less than half the cost of other water supply and treatment alternatives and in some cases the cost savings are close to 90% (Pyne, 1995). In Saudi Arabia a case study showed that the cost of treating artificially recharged water to potable standards was 17% of the cost of desalinating water to similar standards (Ishaq, 1994).

Surface recharge is usually the most cost-effective option of getting water into the ground. If surface recharge is not feasible due to high land costs or hydrogeological constraints, then aquifer storage and recovery (ASR) can also achieve this. The unit costs associated with ASR are usually slightly higher than those associated with surface recharge. The higher costs are due to the need for advanced water treatment prior to recharge. In certain cases ASR facilities can be more cost-effective than surface recharge systems because no additional equipment is required to recover the stored water.

Table 4.1 Relative costs of augmenting Kenhardt's domestic water supply

Options	Capacity (m ³ /day)	Capital + Labour Costs	Major Capital Items
Treat Municipal Waste water to potable standards	200 to 300	- R4.5 million	Waste water treatment facility
Treat water from the Rooibergdam - 4 km from Kenhardt	1000	- R2.5 million	Raw water treatment facility and pipeline
Transfer Water from the Orange River via a - 70 km pipeline	1000	- R17 million	Bulk water supply pipeline
Develop a wellfield (shallow aquifer) - 15 km away from Kenhardt	1000	- R4 million	Pipeline and pump station
Develop a wellfield - 1 km from Kenhardt which contains saline water	1000	- R2.5 million	Desalination facility
Implement an Artificial Recharge scheme by recharging the current wellfield when water is available from the Rooibergdam	1000	- R1.5 million	Raw water treatment to reduce turbidity
Develop a wellfield (deep aquifer) using - 3 deep boreholes - 15 km from Kenhardt	>1000	- R4 million	Borehole drilling, pipeline and pump station

Note: Costs are based on 1997 estimates.

4.2 MANAGEMENT ISSUES

The successful operation of an artificial recharge facility depends largely on an effective management strategy and on sufficient human resources to carry out the necessary tasks. Maximum benefit from an artificial recharge scheme usually involves integrating the scheme into the planning and management of the entire groundwater basin. This includes optimising both surface and groundwater resources, and their storage capacities.

Artificial recharge schemes commonly involve surface or waste water capture, treatment, pumping, distribution and water quality monitoring. In order for these processes to be efficient, careful planning and management is needed. This requires competent personnel dedicated solely to the task of managing the scheme. The responsibility of this operation should not be viewed as another task of the water supply engineer, but rather as another water resource which requires a manager.

Knowledge of the following may be needed to successfully operate an artificial recharge scheme:

- i. Hydrogeology of the basin.
- ii. Integrated water cycle of the catchment.
- iii. Recharge and recovery technology.
- iv. Groundwater level monitoring.
- v. Water quality management.
- vi. Water supply engineering.

4.3 LEGAL ISSUES

Internationally, artificial recharge is one of the developments in water resource management that is challenging the legal fraternity. One of the reasons for this is that in many countries there is no ownership of groundwater, rather, there is only a right to use it. Some of the issues that need to be addressed within a legal framework include:

- i. The protection of public health, safety, property and ecological interests.
- ii. The right to use / ownership of water proposed for recharge.
- iii. How to ensure that the recharged water is not abstracted by competing users.
- iv. How to ensure that the water quality and storage capacity of an aquifer does not deteriorate as a result of artificial recharge.
- v. Who should monitor the management of the scheme. This may be particularly important if aquifer contamination is possible through recharge with water of impaired quality.

Laws for artificial recharge operations in Australia have been drafted along the rule that if the water utility proves ownership of the water prior to artificial recharge then it also has the right to recover that water. However in countries where groundwater law is not adequately defined it is possible for a competing water user to construct a borehole adjacent to the artificial recharge facility and abstract the stored water.

The permit requirement of the state of Arizona in the United States provide a good background around which laws can be drafted. In order to implement artificial recharge operations the applicant must show:

- i. The technical and financial capability of the project.

- ii. The right to use the water for recharge or replenishment.
- iii. The project is hydrologically possible.
- iv. The project will not cause unreasonable harm to land or other water users.
- v. The applicant is in possession of an aquifer protection permit which is a permit designed to protect groundwater resources by regulating point and non-point injections into aquifers.

In the state of California three conditions have to be fulfilled before a public entity can begin artificial recharge operations. They are:

- i. The right of the public entity to import water and store it underground when basin storage space is available, without paying overlying land owners.
- ii. The right of the public entity to prevent others from abstracting the stored water.
- iii. The right of the public entity to recapture the stored water.

Legislation should ensure that artificial recharge operations complement catchment management strategies. It is also important that legislation creates a suitable environment to ensure that initiatives to develop artificial recharge schemes are not suppressed due to inappropriate controls.

With the change in South African water law (Water Services Act No. 108, 1997 and the National Water Act No. 36, 1998), Catchment Management Agencies (CMA) will become the institutions responsible for surface and groundwater management. Catchment management involves managing the hydrological cycle of a catchment. Since groundwater forms an integral part of the hydrological cycle, all activities that have an impact on groundwater resources within a catchment should be managed as part of an overall catchment management strategy. As such artificial recharge will be managed as part of a catchment management strategy in term of the National Water Act, 1998.

4.4 SOCIAL ISSUES

Recovery of artificial recharge has the potential of becoming a significant source of water supply in areas with limited water resources. However public opinion concerning artificial recharge could be a controlling factor in the successful implementation of this technology.

Public concern centres primarily around the perceived quality of the water and whether it might serve to transmit pathogens, viruses or harmful trace chemicals. Public perception about reuse of stormwater or surface water is not likely to be a problem. However problems may arise when waste water is to be used for artificial recharge purposes. A study conducted by Lohman and Milliken (1985) found that in general, the public accepts the use of reclaimed water for a variety of purposes, but not for drinking or other high contact uses such as bathing and cooking. Bruvold (1976) suggests the following to overcome the problems associated with public perceptions:

- i. The public should be brought into the technical decision making process at an early stage.

- ii. Communication of potential risks should be presented in clear and simple language.
- iii. Information should be complete and accurate without distorting the risk or minimizing the existence of uncertainty.
- iv. Information should be oriented to the needs and concerns of the public.

This type of information can help develop the public's understanding of the issues involved in artificial recharge and educate the citizens so they act as informed decision makers (National Research Council, 1994).

Chapter 5

GUIDELINES FOR ESTABLISHING ARTIFICIAL RECHARGE SCHEMES

A phased and multi-disciplinary approach is needed to plan an artificial recharge scheme. Two examples of planning processes are described below. The first example applies to recharge basins schemes and the second example applies to borehole injection schemes. The level of effort and associated financial investment in obtaining data in each phase should be related to the degree of risk, both technical and non-technical. Pyne (1995) suggests that if land availability and hydrogeology are favourable, basin recharge as opposed to borehole injection is usually the most cost-effective recharge method.

5.1 PROCESS FOR PLANNING AN ARTIFICIAL RECHARGE BASIN

Nightingale and Bianchi (1981) describe two main categories of problems associated with artificial recharge basins. These are: 1) maintaining the hydraulic conductivity of surface and sub-surface soil layers; and 2) quality characteristics of the recharge water. The interaction between these two problem areas nearly always causes "soil clogging", whereby particles accumulate within the soil-pore spaces near the soil surface, and eventually seal these spaces. The process outlined below, which has been adapted from, and in places copied from Nightingale and Bianchi (1981), takes these problems into account.

Eight planning phases can be identified:

- i. Precursory site evaluation
- ii. Soil profile and geology determinations
- iii. Recharge water quality determinations
- iv. Biological considerations
- v. Artificial recharge modelling
- vi. Pilot test basin
- vii. Construction plans
- viii. Operation, maintenance and performance evaluation plans

Phase 1 Precursory site evaluation

The aim of this study is to establish the likelihood of the site being suitable for basin recharge without expensive fieldwork. The process involves defining or assessing the following:

Objectives

Define the objectives for implementing an artificial recharge scheme.

Aquifer acceptance potential

Step 1 Estimate the volume of water that can be stored in the aquifer. Examine seasonal groundwater level fluctuations, and establish whether the aquifer will provide sufficient storage for the period over which you want water to be stored. Note that in areas where the water table is close to the surface and natural runoff is to be used as recharge water, the aquifer to be recharged may not have sufficient surplus storage available.

Step 2 Gauge soil textures, typical infiltration rates, salinity, organic matter level, and soil structure stability in water.

Step 3 Estimate the rate at which the aquifer can receive water due to soil and aquifer permeabilities. Evaluate the hydraulic conductivity of geological formations and the significance of confining beds. Note that aquifer transmissivity is integrated over the thickness of the aquifer, and should not be used in establishing aquifer acceptance rates if the aquifer consists of layers with different hydraulic conductivities. Bianchi, Nightingale and McCormick (1978) caution that estimates of recharge rates which are based on soil surface texture (to a depth less than 140 cm), soil columns, or infiltrometer data can be at least ten times the operational recharge rate.

Cehrs, Soenke and Bianchi (1979) recommend that the first 15 to 18 m of the subsurface should have the best possible hydraulic parameters (permeabilities and transmissivities), and that the site should have adequate horizontal water transmission properties to negate any perching effects on the recharge rate.

During this step, the regional hydraulic gradient should be established so that the groundwater flow direction is known.

Recharge water quantity, reliability and quality

Step 1 Evaluate the quantity and reliability of the water available for recharge. If the water is supplied in a controlled manner, for example dam releases or municipal waste water, then obtain the quantity and frequency records. If the water is to come from precipitation, rainfall data and streamflow data should be obtained from local rain gauges and from Surface Water Resources of South Africa, 1990 (Midgely, Pitman and Middleton, 1994).

Step 2 Evaluate the quality of the water. Physical, chemical and biological characteristics need to be examined. The most important physical parameter is the sediment

load. It may be necessary to obtain turbidity measurements from different times of the year in order to establish the variation in sediment yields in relation to rainfall seasons. Estimates of sediment yield are presented in Surface Water Resources of South Africa, 1990 (Midgely, Pitman and Middleton, 1994).

Other important water quality aspects include biological load, temperature, salinity, toxic chemicals (eg. pesticides), level of plant nutrients, biological oxygen demands expected during the proposed recharge period. Aquatic plants, fish and insects can influence water quality, and should, therefore, be considered.

Establish whether there is potential for recharge water to dissolve salts from the unsaturated zone and affect the groundwater quality. This is likely to be of concern in areas where the groundwater is saline.

- Step 3 Evaluate water quality data in terms of:
- i. Its effect on recharge rate (eg. concentration of suspended solids);
 - ii. Its effect on groundwater quality (ie. what is the likely water quality after blending recharge and natural groundwater);
- Step 4 Examine methods (eg filtration, sedimentation) to reduce:
- i. The sediment load;
 - ii. The biological load.
- Step 5 Estimate the potential recharge rate. This initial estimation can be based on previous steps. The approximate volume of water put underground over the recharge period or potential recharge (R_p) can be estimated by :

$$R_p = (R_e \cdot A \cdot t) - E$$

Where:

R_e = initial estimated recharge rate (m/day)

A = estimated wetted basin area (m^2)

t = estimated time of recharge water available (days)

E = estimated volume of water lost by evaporation (m^3)

Evaporation estimates can be obtained from Surface Water Resources of South Africa, 1990 (Midgely, Pitman and Middleton, 1994).

Initial estimate of cost and benefits

- Step 1 Initial estimate of site construction costs. This should include estimates of water control structures, land clearing, soil surface preparation (eg levelling), road construction. During construction soil compaction should be minimised. There may be additional costs associated with developing construction strategies which minimise soil compaction.

- Step 2 Initial estimate of operation and maintenance (O&M) costs. This should include costs of energy, personnel, depreciation of infrastructure, water treatment and other O&M expenses.
- Step 3 Describe the benefits in terms of the primary and secondary objectives.

Information developed during Planning Phase 1 should produce a decision to proceed or not to proceed to the next stage, which involves the expensive collection of data.

Phase 2 Soil profile and geology determinations

The aim of this phase is to establish by field studies a more realistic projected recharge rate for cost and benefit analyses. The focus is on determining effective hydraulic conductivity, continuity and thickness of the soil layers.

Soil surface profile

- Step 1 Randomly select two to four study sites within each of the soil series that account for at least 90 percent of the site area. Collect soil samples at the study sites for each 20 cm increment to a depth of at least 2 m below the planned base of the recharge basin. Check for shallow impervious layers. If the soil deposits are highly variable and the results obtained from the two to four study sites are vastly different, then more sites will need to be studied.
- Step 2 Evaluate soil texture from the soil samples collected and measure the saturated hydraulic conductivity. Low permeability layers should be core-sampled and the conductivity measured by permeameter or other methods applicable to the vadose zone.
- Step 3 Analyse soil samples for salinity and exchangeable sodium percentage (ESP). These factors affect the hydraulic conductivity and stability of surface soil structure during recharge.

Subsurface soil layers

- Step 1 Reevaluate borehole logs for data on layers suspected of having low permeabilities. Aim to establish depth to, thickness and continuity of such layers.
- Step 2 Confirm subsurface soil layers with surface geophysics (e.g. resistivity soundings) or with drilling and core sampling.
- Step 3 Consider the horizontal hydraulic conductivity of the more sandy layers. These layers conduct recharged water away from the recharge site. Since collecting horizontal soil-core samples below the trenching depth is difficult, hydraulic conductivity may need to be measured on disturbed samples.

Characteristics of entire soil profile

- Step 1 Combine all estimates of the saturated hydraulic conductivity of soil layers above the water table. Make a reasonably valid areal distribution map for the facility's estimated vertical recharge rates. For flow that is mainly vertical, the hydraulic mean of the hydraulic conductivity of the layers should be used.
- Step 2 Re-evaluate the estimated recharge rate for the entire recharge basin. Estimate the total volume of water that could be recharged for the expected recharge period and estimated wetted area of the proposed basin.

Abandon further planning for this site if the projected recharge rate is too low or if the total volume of water that could be recharged is too low because of limited land.

Phase 3 Recharge water quality determinations

Field studies are involved to establish the physical and chemical quality of the recharge water. The aim is to project the effect of water quality on infiltration rates and on changes in groundwater quality. The three water quality concerns are:

- i. Physical parameters;
- ii. Inorganic constituents;
- iii. Organic constituents.

Physical parameters

- Step 1 Evaluate suspended solids (nonfilterable residue more than 0.45 μm on membrane filter, dried at 103-105°C). The suspended solids may be mineral or organic. Only mineral suspended solids, mostly silt and clay size particles should be evaluated in this step with respect to their magnitude and seasonal variability and their estimated effect on the infiltration rate of the specific soils series in the basins.

Plan to keep turbid storm runoff water out of the basins when the turbidity values are greater than 5 - 20 nephelometric turbidity units (NTU). In some areas, for example, the Cape Flats aquifer near Cape Town, recharge waters with values between 3 - 5 NTU caused a rapid decline in the infiltration rates and complete clogging of the surface within 14 days. In other cases, turbidities less than 5 NTU due to suspended silt and clay may not significantly reduce the infiltration rate of loamy sand to sandy loam surface soils (Nightingale and Bianchi, 1981).

If the probability of having an average daily turbidity value more than, say 5 NTU (or the desired maximum NTU value) is greater than 10% (e.g. more than 10 days out of a 100-day recharge period), then consider using flocculation and settling basins to protect the infiltration surface of the recharge basins, and/or regular

scraping of the basin surface.

- Step 2 Evaluate seasonal odour and colour variability of the recharge water. Odours can indicate organic waste waters, anaerobic conditions, or odour producing algae. Water colour can indicate dissolved chemicals, suspended material, or high levels of various algae.

Inorganic Constituents

- Step 1 Dissolved solids (filterable residue dried at 180°C and passing through a 0.45 µm membrane filter) can be correlated with the electrical conductivity of the water. First, evaluate the effect of salinity on the infiltration rate. The lower the salinity level for a given soil, the greater the tendency for dispersion of colloids (clays) and their movement downward, which may cause subsurface clogging or slightly turbid groundwater. Second, evaluate the significance of the initial impact of the recharge water's salinity plus soluble salts leached from the soil on the groundwater quality. These considerations provide projected short- and long-term effects on groundwater quality.
- Step 2 Evaluate the effects of the cationic constituents Ca^{2+} , Mg^{2+} , Na^+ and the sodium adsorption ration (SAR) on the infiltration rate at the salinity levels to be expected for the soil mineralogy of the basin soils.
- Step 3 Evaluate the magnitude and seasonal variability of the major plant nutrients (nitrogen and phosphorus) that promote excess growth of aquatic plants in the basins. High levels of N and P in the water can cause eutrophication, accumulation of organic detritus on the basin floor, and biological clogging of the soil. Water containing high nitrate concentrations requires careful management to control nitrate pollution of the groundwater. At the Leaky Acres Recharge Facility in the United States of America, $\text{NO}_3\text{-N}$ levels less than 0.45 mg/L and ortho- PO_4 levels less than 0.05 mg/L have not caused excess growth of aquatic plants.
- Step 4 Check recharge water for unacceptable levels of inorganic toxicants. These usually can be traced to industrial wastes, e.g., arsenic, barium, cadmium, chromium, lead, mercury, selenium, and silver in the recharge water.

Organic Constituents (Nonliving)

- Step 1 Check recharge water for organic chemical constituents and for the more persistent pesticides, including pollutants commonly associated with industry located in the area of the water source. Groundwater pollution is possible from a wide range of organic compounds under various geological and soil conditions. Medium-textured soil layers, however, are relatively effective in adsorbing most organic chemicals, and their movement into the groundwater is usually limited.

- Step 2 Evaluate the magnitude and seasonal variability of suspended organic debris in the water associated with the plant and animal life in the water sources. High levels of organic debris input to the basins cause biological clogging of the soil as soil microorganisms decompose this material.

Phase 4 Biological considerations

Biological clogging of the basin soils, where excess biomass is produced, can be more significant than sediment clogging. Plant-nutrient laden and high biochemical oxygen demand (BOD) recharge water can lead to massive aquatic plant populations that cannot be economically controlled.

Aquatic Biology

- Step 1 Determine the potential for biological clogging by evaluating factors that could generate excessive biomass production in the basins. The excess is that amount which fails to completely decompose and oxidize during the non-recharge or drying period. Therefore, the time of year selected for drying soil determines the soil temperature and moisture levels, which in turn influences the decomposition rate. The decomposition rate determines the maximum tolerable biomass production without causing biological clogging, when the soils are again submerged. It also determines what management practices can be used to prevent or even reduce the percentage of organic residue in the surface soils.
- Step 2 Gather the basic aquatic biology information needed to estimate the recharge basin aquatic biology by field observations at ponds, lakes, canals, and streams near the recharge facility. These observations should concentrate on common organisms and those that can move (float, swim, fly, or be carried) into the basins and, hence, into the surrounding area and thus become a nuisance or health hazard.
- Step 3 Initiate ideas and plans to control biomass production, flying aquatic insects and animals detrimental to the water control structures. Biological controls must be emphasized over chemical control methods or mechanical methods with heavy equipment that reduces surface soil porosity.

Terrestrial Biology

- Step 1 Establish whether there are any animals in the proposed recharge area which may have an effect on the recharge facility. A field study and discussions with local residents and civil engineering companies would give an idea of potential problems.
- Step 2 By field surveys, determine the species of the common terrestrial plants. Determine those species capable of growing in the soil levees and along

shorelines. Develop plans to control nuisance plants without use of herbicides.

Phase 5 Artificial recharge modelling

With the available geohydrologic information, the artificial recharge facility should be modelled with the following aims:

- i. Hydraulic simulation: To more accurately predict the rate at which recharge water can enter the aquifer; to establish the direction and rate of water movement through the aquifer; and to estimate the percentage of recovery.
- ii. Geochemical simulation: To evaluate mixing between the recharge water and the natural groundwater in the presence of aquifer minerals; and to establish constituent concentrations during recovery.
- iii. Wellfield design simulation: To optimise the hydraulic design of wellfield and operations.

The model would be useful in determining the location and design of the pilot recharge basin; the location of abstraction and monitoring boreholes; and in optimising the design of the production artificial recharge facility.

Phase 6 Pilot test basin study

In this planning phase, a pilot test basin option is considered worthwhile if the artificial recharge predictions obtained from the model appear acceptable, and if the recharge water quality and biological considerations are acceptable. But confirmation of the estimated projected recharge rate (Phase 2) is desirable because of the uncertain effect of subsurface layers on the facility's recharge rate. The aim of the pilot study is therefore to establish production recharge rates prior to the construction of the artificial recharge facility.

The size of the pilot basin will depend on the recharge rates which are established during the modelling phase and on the availability of recharge water. If a single large basin is planned, then the basin should probably not be smaller than 60 m x 60 m (± 0.4 -hectare), however if several small basins are planned then the size of the pilot basin should not be smaller than about 20 m x 20 m. Huisman and Oltshoorn (1983) present formulas to determine the size of a spreading basin under different hydrogeologic conditions.

The hydrogeologic factors which affect the design of the basin include: the depth to the water table, the aquifer's saturated thickness, the aquifer's permeability and the aquifer's porosity. Other factors to consider include: the capacity of the recharge basin, or the availability of recharge water (in m^3/day), the planned detention time of the recharge water in the aquifer, and the depth and shape of the basin. For more information on these factors, see Huisman and Oltshoorn (1983), Chapter 7 "Artificial Recharge by Spreading".

The following extract from Bouwer (1989) illustrates the complexities associated with designing the depth of the basin:

“Deep basins exert more hydraulic head on the bottom and, hence, produce higher infiltration rates. However, sediment or other clogging material that accumulates on the bottom will be more compressed in deep basins than in shallow basins because of the increased seepage force across the clogging layer due to the greater water depth. This can markedly increase the hydraulic impedance of the clogging layer. Thus, an increase in water depth may not necessarily produce an increase in infiltration rate. If infiltration rates do not increase in proportion to an increase in the water depth, the turnover rate of water in a deep basin is slower than in a shallow basin. This exposes suspended algae longer to sunlight and can result in significant increases in algal concentrations. Suspended algae are then filtered out on the bottom soil as the water infiltrates, and form a filter cake. This can seriously reduce infiltration. Moreover, a dense, actively photosynthesising algal population can lead to significant increases in the pH of the water due to uptake of dissolved carbon dioxide by the algae. This causes precipitation of the calcium carbonate, which aggravates clogging problems on the bottom. Thus, an increase in water depth in recharge basins may actually result in lower infiltration rates, a paradoxical phenomenon indeed!”

Piezometers should be placed laterally beyond all basin boundaries and down to at least the first suspected perching layer. If it is necessary to establish the affect of the basin on different hydrogeologic layers (for example, alluvium, weathered rock and fractured hard rock), then it would be necessary to insert the piezometers to different depths, with different layers cased off.

Piezometer data are used to estimate the area and height of the perched water mound. Using the area of the perched mound, the observed intake rate, and the ponded area, one could calculate the expected Darcy velocity through the sublayer. The results should be used to upgrade the model which was produced during the previous phase, so that an accurate estimate of the production recharge rate can be obtained.

Phase 7 Construction Plans

Construction plans depend on the site characteristics determined in the preceding five planning phases. Poor construction practices can largely destroy the porosity of the surface soil and prevent economical recharge. Improper construction techniques are:

- compacting basin soil using heavy construction equipment;
- improperly located inlet water control structures;
- allowing wave-caused soil erosion from the levees.

The following construction processes may be relevant for a single, fairly large recharge basin (from Nightingale and Bianchi, 1981):

Surveying

- Step 1 Adapt a surveying grid system appropriate to the site topography to obtain required surface elevation resolution.

- Step 2 Develop plans to obtain surface elevations and construct contour maps for locating levees and water control drop structures.

Planning for security and area cleanup

- Step 1 Perimeter fence and secure gates.
- Step 2 Secure equipment and maintenance yard.
- Step 3 Remove unwanted buildings, fences, trees and shrubs.
- Step 4 Provide for accommodation of existing utilities, sewer, water and roads.

Levee Construction

- Step 1 Develop levee construction plans that will prevent:
- i. soil erosion during basin filling;
 - ii. basin soil compaction.

This effort is best accomplished by using contour levees of appropriate height for the ponded water depth (\pm 40-80 cm) and for the desired freeboard. To minimize levee erosion due to wave action, keep the basin width less than about 150 m in the direction of prevailing strongest winds to minimize wave height. Confine all levee construction equipment and vehicles to the levee width. Where rodent activity is excessive, destroy burrows beneath levee alignment.

- Step 2 Plan for a soil ramp in an upwind corner of each basin leading from the top of the levee down into the basin.
- Step 3 Slope levee sides at about 1:1 to 1:1.5 and face sides with 3-6 cm crushed rock to control wave erosion. The basin walls should be built of permeable material to allow for lateral water movement. Note that the accumulation of silts and other suspended solids will be least on steep sided slopes. Another reason for wanting steep banks is that flatter banks are more suitable for vegetation, and subsequently encourage burrowing animals to build habitats. Flatter banks are also more suitable for harbouring mosquito populations.

Water Control Structures

The water control system must:

- prevent highly turbid storm runoff water from entering the basins;
- prevent rubbish from entering the basins;
- allow volume measurements of water diverted for recharge;
- provide security to prevent unauthorized change in flow rate into the basins;

- prevent overflowing of basins.

Ideally, water should enter a basin at its lowest point for maximum water erosion control. This arrangement can be accomplished by either correct positioning of the inlet structure or by grading a channel within each basin.

- Step 1 Plan site-specific structures to control the water flow into the basins and the water surface elevation within each basin. Each structure (e.g. an overflow pipe from an upper to lower basin with a drop-structure) must be designed to allow complete shut off and also to prevent soil erosion where water is discharged into a basin.
- Step 2 Plan to prevent rubbish in the delivery canal from entering the basins. The volume of water delivered to the basins should be determined by appropriate weirs (with recorders) in the main canal or laterals to the basins.
- Step 3 Plan for staff gauges in the basins to monitor ponded water elevation necessary for regulating inflow during basin operation.

Wells for Groundwater Quality Evaluation

Provide several groundwater quality observation boreholes around the perimeter levee. The casing can be PVC with longitudinal slots (± 3 mm by 30 cm) on the lower section of the casing, with a gravel pack around slots. The boreholes should have a surface cement grout seal.

Phase 8 Operation, maintenance and performance evaluation plans

By the time the recharge site planners have progressed through the preceding planning phases, they should easily develop ideas and plans to manage the recharge facility. The performance evaluation plans are prepared with respect to engineering criteria of performance, water quality changes, and the energy and economics of recharge and recovery - all within the framework of an environmentally acceptable recharge facility.

5.2 PROCESS FOR PLANNING AN AQUIFER STORAGE RECOVERY (ASR) SCHEME

The process outlined below is based on, and in places copied from Pyne's (1995) three phases in the ASR planning process:

- i. Feasibility assessment and conceptual design
- ii. Field investigations and test programme
- iii. Recharge facility expansion

Phase 1 Feasibility assessment and conceptual design

This study which is primarily a desk study, can be expected to take 2 - 3 weeks, and it will include:

- a review of available data;
- conceptual design of the scheme;
- preliminary modelling;
- cost estimate of the capital and operating costs.

Objectives

Define the objectives for implementing an artificial recharge scheme.

Recharge water quantity and reliability

Evaluate each potential source in terms of flow availability, monthly or other variability in flow rate, and any trends in flow. It is usually insufficient to know the average and peak rate of water availability - monthly variability is essential. Assess long term availability of each potential source, taking into consideration future possible competing users.

Recharge water quality

Seasonal or long-term trends in water quality should be assessed. This is particularly important if surface water is used, since periods of high flow can also be times of poorer quality water, which can cause problems with water treatment and borehole clogging. While data on the suspended solids content may not be available for Phase 1 investigations, plans should be developed on how to assess how these solids vary with time and flow, and what materials contribute to these solids. This information is necessary for an understanding of well clogging and the planning of well redevelopment. In order to provide a basis for understanding and resolving clogging issues, particle counting and a detailed analysis of particle size may be necessary.

Compare recharge water quality with applicable water quality standards (DWAF, 1993).

After recharge water quantity, reliability and quality have assessed, it is possible to determine the times of year when recharge water of suitable quantity and quality are available, and the annual recharge volume potentially available in the initial and subsequent years.

Water demand

In most situations it is important to evaluate average water demands, monthly variability and demand trends. In order to establish seasonal, monthly and daily variability, monthly records of maximum, minimum and average daily demand are useful. This information, combined with information on the rate at which an aquifer can accept water and the volume of water that can be

stored in the aquifer, can be useful in determining the number of recharge wells to be installed.

Hydrogeology

The hydrogeological evaluation during Phase 1 should consider the following issues to the extent possible with available data and resources. Where important information is not available, it will either have to be gathered during this phase or early in Phase 2.

Hydrogeological issues to be evaluated:

- stratigraphy, including geologic cross-sections;
- aquifers (areal extent, thickness, and depth);
- confining layers or aquitards (extent, thickness, and depth);
- lithology of aquifers and confining layers;
- potential availability of cores;
- hydraulic characteristics (transmissivity, storativity, leakage, hydraulic conductivity, porosity, etc.);
- recharge and discharge boundaries;
- water table levels or potentiometric surface;
- local gradient of the potentiometric surface;
- natural groundwater velocity and direction;
- typical well construction and production rates;
- mineralogy of clays, sands, and other soil components;
- geophysical logs;
- water quality of each aquifer;
- geochemical compatibility of recharge and natural groundwater with formation minerals;
- structure (unconsolidated, consolidated, fractures, bedding planes, solution features, fissures, etc.);
- well inventory within a reasonable radius;
- groundwater withdrawals within the surrounding area;
- proximity of potential sources of contamination;
- proximity of potential contamination plumes that may be affected by recharge operations.

In order to assess the risk of geochemical clogging, cores should be obtained in the potential storage zone for the following situations where:

- insufficient data is already available to perform such an assessment;
- clays, silts and other fine materials are possibly present in the potential storage zones;
- there is no local recharge experience to consider.

Site selection

Besides the hydrogeologic considerations, the test site should be located:

- close to the final water treatment plant, or at some point in the distribution system;

- close to qualified personnel who will be available to gather hydraulic and quality data, and who will be able to pick up problems quickly.

Conceptual design

The conceptual design should include:

- location of ASR test wells, pipelines, buildings, controls and other facilities;
- cost estimate based on the above facilities;
- cost estimate for the ultimate recharge facility, including capital and operational costs;
- whether existing or new wells are to be used. Note that existing wells are often unsuitable because of their inefficiency. Where storage water quality differs from recharge water and where geochemical issues are a concern, the ASR well is likely to require a different design to production wells, and therefore a new well should be used.

Hydrogeologic simulation modelling

If sufficient, quality data exists to model the ASR system, then the model should be developed at this stage. If sufficient data are only going to be available during the second phase, then the modelling should be delayed. The objectives of ASR modelling are:

- i. Hydraulic design of wellfield design and operations;
- ii. Geochemical simulation to evaluate mixing between recharge water and natural groundwater in the presence of aquifer minerals;
- iii. Determination of the direction and rate of water movement during aquifer storage, to establish constituent concentrations during recovery, and to estimate percentage recovery.

Outline test programme

Plan the testing programme for Phase 2. This should include: water levels; flows; pressures; water quality for cycles of operation.

Legal and institutional issues

A common concern is ownership of stored water. While the South African water law does not describe rights associated with artificially recharged water, it is reasonable to assume that rights to water are not lost due to underground storage. In the United States of America, state legislation is increasingly supporting the position that, if water is already available to a user, it is also available to that user through recovery from storage (Pyne, 1995). It may however, be difficult and costly to prove that another user is abstracting injected water, and it may also be difficult to prevent this from occurring.

All the institutional issues which could hamper the implementation of the recharge scheme should be identified and addressed. Since access to, and control of water resources commonly reflects political power, it may be necessary to develop strategies to overcome institutional

blockages.

Economic considerations

A preliminary cost estimate of the capital and operating costs should be made.

Final report

Phase 1 results should include a report presenting:

- the technical approach to the ASR programme;
- sufficient information on which to obtain the necessary funding, permits, institutional support and environmental support;
- sufficient information to decide whether to stop or continue with phase 2.

Phase 2 Field investigations and test programme

This phase involves designing and constructing the ASR test facilities. The aim should be to establish the following:

- well clogging;
- geochemical effects such as cation exchange, precipitation, or solution, and their effect upon well clogging;
- backflushing frequency required to maintain recharge capacity and control well clogging;
- mixing characteristics between injected and natural groundwater;
- water quality changes for selected non-conservative constituents of interest;
- improvement of water quality with successive ASR cycles;
- effect of storage time on water quality response;
- recovery efficiency;
- trickle injection flow rate during periods of no recharge or no recovery, required to maintain a disinfectant residual in the well (this may also be required to maintain a target recovery volume in a highly brackish or seawater aquifer subject to density stratification losses);
- regional and local response of water levels to ASR operations.

Baseline testing

Baseline hydraulic and water quality testing is needed to provide a reference point against which future results may be compared.

Hydraulic testing should include:

- multiple discharge or step tests in order to establish well and formation loss coefficients and well efficiency;
- a constant discharge and recovery test on pumped and observation wells in order to

estimate aquifer transmissivity and storativity;

- step-injection test in order to characterise water level response in the ASR well under reverse conditions. Each step should be 2 - 4 hours duration. Water level response to this test characterises the baseline water level of the well in the presumed absence of clogging. This test can be repeated in future to establish whether clogging has occurred;
- baseline water quality should be established. Water samples at the beginning, middle and end of the constant discharge test should be taken. The last sample should be comprehensively analysed (including physical, chemical and micro-organisms). The first two should be used to establish quality trends, and parameters such as chloride, conductivity and pH should be tested. If a trend is apparent, then the test should continue until equilibrium is reached.

ASR cycle testing

The appropriate number and duration of injection and abstraction cycles needs to be planned. The following points should be considered:

- A short initial cycle of 1 - 2 weeks is advisable to confirm ASR performance at small volumes, to provide a quick appraisal of clogging, and to assess geochemical reactions.
- Aim to recover 150 - 200 % of the injected water during the first cycle, or until recovered water quality approaches natural water quality. Recovery should be less than 100 % of the injected water volume in situations where geochemical reactions are a concern.
- A small number of long cycles is appropriate if water recharge and natural water quality is similar. The focus of the test is on clogging rates and backflushing frequency required to maintain recharge rates. At least three cycles should be carried out with the third cycle approximating an operational recharge cycle. Recovery of 100 % of the stored water volume in each cycle after the first is a reasonable target.
- A large number of cycles (4 - 10 cycles) will be required if there is a significant water quality difference between natural and recharged water. After the first cycle, the next three cycles should have the same recharge volumes and storage period in order to demonstrate improvement in recovery efficiency with successive identical cycles. Subsequent cycles can have larger quantities of recharge water. Recovery should occur to a target water quality concentration in each cycle. The water not recovered in each cycle forms a buffer zone to improve water quality during subsequent cycles (assuming that the injected water is of better quality than the natural groundwater).
- A larger number of smaller cycles (6 - 10 cycles) should be run where geochemical reactions are an issue. By doing so, it is possible to demonstrate control over geochemical issues near the well before moving to longer cycles.

It is advisable to analyse the test data during the test programme so that adjustments in the programme can be made if necessary.

The duration of the test programme can vary from about 3 months to 2 years, and is typically run for 6 to 12 months. Short programmes are appropriate where no geochemical or water quality issues are involved and where clogging does not appear to be significant.

Data collection

Hydraulic data collection includes the following parameters:

- flow rate during recharge and recovery;
- cumulative volume injected;
- water level or pressure in the well;
- wellhead injection pressure;
- water level response in observation wells;
- elevation of pressure/water level measurement points.

Due to mechanical problems that may arise during the test, it is advisable to have two flow meters (in series) and both a pressure gauge and hand operated water level meter.

An example of a cycle testing data collection form from Port Malabar, Florida is presented in Table 5.1. The form is set out in such a way that data is easily converted to specific capacity or specific injectivity.

Field data should be collected daily. The frequency of water quality analyses will depend on the water quality variability. Recovery sampling should be frequent enough to show hydraulic and water quality trends.

Table 5.1 Cycle testing data collection form, Port Malabar, Florida (From Pyne, 1995)

Date	Time	Elapsed time (hrs)	Injection meter	Volume injected (gal)	Well head pressure (psi)	Injection rate (gpm) (*calc.)	Specific injty (gpm/l)	Percent injected	Sample number	Chloride (mg/l)	Cond. (mmh/cm)	pH	pHs	Alkalinity "P" (mg/l)	Alkalinity "T" (mg/l)	Hardness Total (mg/l)	Hardness Ca (mg/l)	Turbidity (NTU)
08/01/88	10 6	0.00	6839960	0	5.27	0	0	0.00	--	--	--	--	--	--	--	--	--	--
BEGIN INJECTION New Meter Installed																		
08/01/88	12 15	0.00	0	0		405		0.00	--	--	--	--	--	--	--	--	--	--
08/01/88	17 20	5.08	57000	57,000	18.00	360	12.2	1.64	C31R1.1	186	664	8.3	8.08	0	32	180	160	0.60
08/02/88	11 22	23.12	437000	437,000	18.50	351	* 11.5	12.57	C31R2.1	178	653	8.2	8.14	0	32	168	140	0.32
08/03/88	7 55	43.67	866000	866,000	18.50	348	* 11.4	24.91	C31R3.1	170	614	7.8	8.26	0	24	160	140	0.36
08/04/88	8 11	67.93	1374000	1,374,000	18.80	349	* 11.2	39.53	C31R4.1	174	634	8.0	8.21	0	28	164	136	0.47
08/05/88	9 59	93.73	1872000	1,872,000	18.60	322	* 10.4	53.85	C31R5.1	178	642	8.1	8.21	0	28	164	136	0.46
08/06/88	8 5	115.83	2379000	2,379,999	18.70	382	* 12.3	68.44	C31R6.1	180	611	8.7	8.11	8	36	164	132	0.54
08/07/88	9 17	141.03	2908000	2,908,000	19.00	350	* 11.0	83.65	C31R7.1	183	634	8.2	8.22	4	28	164	132	0.87
08/08/88	8 6	163.85	3386000	3,386,000	18.90	349	* 11.1	97.40	CS1R8.1	178	635	8.5	8.16	4	32	164	132	0.49
08/08/88	12 0	167.75	3469000	3,468,000	18.50	355	* 11.6	99.79	CS1R8.2	176	592	8.5	8.38	0	20	168	128	0.34
08/08/88	12 21	168.10	3476250	3,476,250	--	345	* --	100.00	--	--	--	--	--	--	--	--	--	--

END INJECTION

Note:

*calc: Calculated injection rate

P alkalinity is alkalinity at pH > 8.3 (ie carbonate rather than bicarbonate)

Table 5.1 continued overleaf

Table 5.1 continued

Date	Time	Elapsed time (hrs)	Recovery meter	Depth to water (+/-abv MP) (-/-below MP)	Recovery rate (gpm)	Specific capacity (gpm/ft)	Volume recovered (gpm/ft)	Percent recovery	Sample number	Chloride (mg/l)	Cond. (mmh/cm)	pH	pHs	Alkalinity "P" (mg/l)	"T" (mg/l)	Hardness Total (mg/l)	Ca (mg/l)	Turbidity (NTU)	HS (mg/l)	Sulfate (mg/l)
08/08/88	14:30	--	0	+15.75	0	--	0	0.0	--	--	--	--	--	-	--	--	--	--	--	--
BEGIN RECOVERY																				
08/08/88	15:1	0:99	0	--	0	--	0	0.00	--	--	--	--	--	-	--	--	--	--	--	--
08/08/88	15:2	0:02	300	--	--	--	300	0.01	C3RR1.1	--	--	--	--	-	--	--	--	1.8	--	--
08/08/88	15:6	0:08	2500	--	550 *	--	2500	0.07	C3RR1.2	--	--	--	--	-	--	--	--	1.5	--	--
08/08/88	15:12	0:18	5500	--	700 *	--	5500	0.16	C3RR1.3	--	--	--	--	-	--	--	--	1.25	--	--
08/08/88	15:17	0:27	10000	--	900 *	--	10000	0.29	C3RR1.4	--	--	--	--	-	--	--	--	1.75	--	--
08/08/88	15:27	0:43	16000	--	600 *	--	16000	0.46	C3RR1.5	--	--	--	--	-	--	--	--	0.66	--	--
08/08/88	15:30	0:48	18500	--	--	--	18500	0.53	C3RR1.6	--	--	--	--	-	--	--	--	0.38	--	--
08/08/88	15:40	0:65	25500	--	700 *	--	25500	0.73	C3RR1.7	--	--	--	--	-	--	--	--	0.47	--	--
08/08/88	15:49	0:80	31500	-39.67	667 *	12.0	31500	0.91	C3RR1.8	--	--	--	--	-	--	--	--	0.82	--	--
08/08/88	16:50	1:07	43000	-45.81	700	11.4	43000	1.24	C3RR1.9	190	614	8.5	8.3	0	24	172	140	0.25	--	--
08/08/88	20:00	4:98	208500	--	710	--	208500	6.00	C3RR1.10	190	614	8.3	8.0	0	40	188	160	0.50	--	--
08/09/88	20:00	16:70	700500	-48.63	700	10.9	700500	20.15	C3RR2.1	185	676	8.3	8.0	0	40	190	148	0.22	--	51
08/09/88	14:20	23:32	975000	-48.15	695	10.9	975000	28.05	C3RR2.2	186	712	8.4	8.0	0	40	192	152	0.30	0.00	52
08/09/88	19:50	28:82	1160000	-46.50	695	11.2	1160000	33.37	C3RR2.3	180	712	8.5	8.0	0	40	188	156	0.34	0.00	50
08/10/88	8:20	41:32	1689500	-50-96	725	10.9	1689500	48.60	C3RR3.1	195	730	8.2	7.9	0	44	208	164	0.31	0.00	54
08/10/88	13:10	46:15	1870680	--	625 *	--	1870680	53.81	C3RR3.2	200										
08/10/88	15:20	48:32	1960000	--	680	--	1960000	56.38	C3RR3.3	198	734	8.3	7.9	0	44					
08/10/88	19:26	52:42	2123000	-43.50	660	11.1	2123000	61.07	C3RR3.4	216	767	8.2	8.0	0	44					
08/11/88	8:15	65:23	2672500	-51.43	730	10.9	2672500	76.88	C3RR4.1	250	846	8.2	7.8	0	60					

* Calculate Recovery Rate

Phase 3 Recharge facility expansion

Once the test phase indicates that a fully operational ASR facility is feasible, the wellfield is expanded. The factors to consider in the design of an ASR wellfield which are different from a conventional wellfield are:

Well spacing and arrangements

The most suitable well spacing and arrangements can be established using data from Phase 2 in a groundwater simulation model. The aim is to maximise recovery efficiency. In general, the poorer the quality of the aquifer's water, the closer the wells will need to be. Experience from the USA shows that for an ASR system that primarily meets seasonal water supply needs, the annual volume of water stored will create a bubble of injected water around the well, with a diameter in the range of 100 - 300 m. During seasonal storage it will move away from the borehole at a rate dependent on:

- the regional hydraulic gradient;
- the aquifer hydraulic conductivity;
- porosity.

The well spacing should be such that these bubbles of injected water coalesce, as this will lead to increased recovery efficiencies. Generally South African aquifers are not isotropic, and therefore the bubbles will tend to be ellipsoid or irregularly shaped.

Stacking

Where aquifers occur at different depths, separated by impermeable formations it may be possible to store water in more than one aquifer at one site. Although in South Africa the potential for this is small, there are areas where deep seated secondary aquifers exist below the near-surface, weathered rock aquifers. Commonly the deeper seated aquifers have poor quality water. These aquifers may have no value for water supply purposes, but could be useful for ASR purposes.

Regional hydraulic gradient

Where the regional hydraulic gradient is high and the stored water may move a significant distance from the ASR well between time of recharge and recovery, a linear arrangement of boreholes perpendicular to the hydraulic gradient may be more efficient.

Chapter 6

ARTIFICIAL RECHARGE AROUND THE WORLD AND IN SOUTH AFRICA

Artificial recharge to South African aquifers is not a new concept. Scattered throughout the country are small earth dams which farmers have built to augment their borehole supplies. In all but one case, so it seems, the impact of these artificial recharge schemes on groundwater has not been established. The one study where records were kept, was in the Soutpansberg District of the Northern Province, where DWAF recorded borehole yields and water levels prior to and after the construction of the earth dams (De Villiers, 1971). Unfortunately the information obtained is insufficient to establish the true effect of the recharge dams. Ideally, the data required should include water levels and rainfall, spanning a few years prior to, and after the construction of the dams. Monitoring the schemes for a few years after the construction of the dams is particularly important, since the rate of infiltration from the dams is likely to decrease dramatically as silt accumulates in the dams. The results of the De Villiers (1971) study are summarised in Table 6.2.

The one artificial recharge scheme where records exist which demonstrate the effectiveness of the scheme, is at the town of Atlantis in the South Western Cape. The two large recharge basins which feed the primary, dune-sand aquifer, are responsible for supplying in the region of 2 million cubic metres per annum of recharge water (Tredoux and Wright, 1996). The water source for the Atlantis recharge scheme is storm water runoff and treated domestic waste water. This scheme is briefly described in Table 6.2.

The two main hydrological factors which determine the potential for artificial recharge in South Africa are the availability of raw water and whether the aquifer can physically receive surplus water. In relation to the first factor, that is, the water source, of prime concern is the reliability and quality of the raw water. Possible water sources include ephemeral and perennial rivers, dams, municipal waste water and storm runoff. With South Africa's high evaporation rates it can be cost effective to store water underground rather than at the surface. If rivers or dams are to be used for artificial recharge, it will be necessary in virtually all cases to reduce the turbidity of the water in order to prevent clogging.

In relation to the second factor, namely the aquifer acceptance potential, the permeability of the aquifer (and the soil horizons above the aquifer if infiltration methods are applicable) and the storage potential of the aquifer are the key factors which will determine the suitability of an aquifer for artificial recharge. Although aquifers which have high storativity values and which are highly transmissive are most suitable for receiving additional water, aquifers with low storativities and low permeabilities like the hard rock aquifers commonly found in South Africa, can be artificially recharged.

Section 6.1 lists examples of successful artificial recharge schemes from around the world, and

it includes cases of artificial recharge in aquifers which are characterised by low storativities and low permeabilities. This is intended to provide examples which may guide local developments. Section 6.2 looks at existing artificial recharge schemes in Southern Africa; and Section 6.3 suggests South African geohydrological environments which may be suitable for artificial recharge. The following chapter looks at possible test sites in secondary aquifers which were identified by local geohydrologists.

6.1 ARTIFICIAL RECHARGE APPLICATIONS IN OTHER COUNTRIES

Artificial recharge is practised throughout the world, and in some countries, artificial recharge schemes were introduced over a century ago. A survey of artificial recharge practice in fourteen European countries (Connorton and McIntosh, 1994) concluded that:

- artificial recharge is important in ten of the fourteen countries surveyed;
- the use of artificial recharge is increasing within the countries in which it is operating;
- artificial recharge schemes are operating successfully;
- in addition to increasing water supplies, artificial recharge often has environmental and water quality aims and benefits.

Listed below are examples of artificial recharge schemes which could serve to encourage local applications. The first few examples show how artificial recharge forms part of bulk water supply schemes, and the latter examples show that it is possible to recharge relatively low permeability aquifers. Artificial recharge by borehole injection is carried out in secondary aquifers with transmissivities of 70 m²/day in Australia, 12 m²/day in Kuwait, and in India, boreholes with yields as low as 1 - 2 l/s are artificially recharged. Transmissivity values and borehole yields of this nature are common in South Africa's predominantly secondary aquifers.

The case study in Kuwait is particularly relevant, since the aquifer contains brackish water, like many South African aquifers. One of the aims of the project is to establish whether better quality water could be stored within a relatively saline aquifer.

Countries making up the former USSR (Usenko and Altshoul, 1987)

In the countries which make up the former USSR there are more than 50 operational artificial recharge schemes which provide 1.5 x 10⁹ m³/year; and a further 300 promising areas have been investigated which could yield an additional 47.3 x 10⁹ m³/year. Virtually all the schemes employ infiltration as opposed to injection methods.

Germany (Schöttler, 1996)

Fifteen percent of Germany's drinking water is supplied by artificial recharge. While a considerable proportion of this is by means of river bank filtration and recharge basins, there is increasing use of other methods. One method which is growing in popularity - six schemes were constructed between 1983 and 1992 - is the use of seepage trenches. The trenches are generally

1 m wide, up to 100 m long and several metres deep (generally about 5 m deep, with the deepest being 9 m). They are filled with coarse sand and a covering, and the cut through the impermeable soil layers. An example of such a trench system is at Hessian Reid, where water is taken from the Rhine River, treated to drinking water quality and then recharged into nine 100 m length trenches. All together they cover a length of 2 km. During test runs, the infiltration rate was $90 \text{ m}^3 \cdot \text{m}^{-2} \cdot \text{d}^{-1}$, but during production the rate decreased to between $10 - 60 \text{ m}^3 \cdot \text{m}^{-2} \cdot \text{d}^{-1}$. The main reason for this poor result is cited as being the lack of exploration drilling to find the most suitable site for the infiltration plant.

Table 6.1 summarises the use of artificial recharge in Germany

Table 6.1 Use of artificial recharge in Germany in 1992 (Source: Schöttler, 1996)

	Volume [million m^3/a]	Percentage [%]
Drinking water supply	520.1	52.9
Raising of groundwater level	196.4	20.0
Displacement of poor quality groundwater	105.2	10.7
Displacement of unwanted bank filtrate	106.1	10.8
Subterranean water storage	40.1	4.1
Preservation of wetlands	8.5	0.9
Raising of lake water level	3.4	0.3
Groundwater rehabilitation	3.1	0.3
Total amount	982.9	100.0

Finland (Hatva, 1996)

Groundwater usage in Finland by communities amounts to 56% of the total water used. Of this, 32% is artificially recharged - half by means of induced river bank infiltration and half by means of basin and pit infiltration. The depth from the bottom of the infiltration basins to the groundwater table varies from less than 2 m to 45 m, and the distance from the basins to the abstraction points varies from 50 m to 2.6 km. The detention time of the water in the ground varies from less than 7 days to 80 days, with the most common time being about 30 days. The infiltration rates at the basins vary from 0.72 m/day to 2.88 m/day. In most cases the water source is a lake.

The Netherlands (Stakelbeek, Roosma and Holzhaus, 1996; De Jonge, 1996)

Artificial recharge forms a major component of the Netherlands' drinking water supply schemes. In 1957 artificial recharge was introduced to enhance the groundwater resources in the dunes of North-Holland, and to counter the upconing of the fresh/salt-groundwater interface which resulted from over abstraction. The scheme initially consisted of open canals by which pre-treated surface water was infiltrated into the soil. In the early 1990's, deep injection boreholes were

added to the scheme. Besides having the advantage of penetrating the semi-confined aquifers at greater depths, the boreholes require very little land - which is a great advantage in the Netherlands.

In South-Holland, a similar scheme has been in operation since 1955. It is (continuously/regularly) being expanded to cater for an increasing water demand. Since 1990, the original infiltration basin scheme has been expanded to include 24 injection boreholes. The infiltration basins vary from 3 000 m² to 156 000 m² and the depths of the basins are generally in the region of 2 m. The infiltration rates of the more recent, deeper basins (≥ 2 m), are generally higher than the older, shallower basins. These rates range from 0.06 m/d to 1.15 m/d, with the average being 0.12 m/d. The infiltration rates at the boreholes, which have diameters of 0.315 m and 1.0 m, and which are screened for 35 m, range from 3.8 m/d to 5.6 m/d.

Israel (Guttman, 1986)

The Yarkon-Taninim Aquifer, a dolomite-limestone aquifer, is one of Israel's three principal sources of water. Artificial recharge is used to: increase the volume of water in the aquifer; increase the water available for production in the summer months and during periods of high demand; to prevent the intrusion of saline water into the aquifer; and to store surplus water from Lake Kinneret (another principal source of water for Israel). Recharge is carried out by means of boreholes and through quarries. The volume of water recharged depends on the availability of surplus water and the water level in the aquifer. The average over 23 years is about 26 million m³/year, or about 7.5% over and above the average volumes obtained from natural replenishment. Temporary clogging of the boreholes occurs during the injection periods, but the specific discharge returns to its previous value after several weeks of pumping.

United States of America

Many methods for artificial recharging aquifers are practised in the United States of America. A few examples are given below.

Peoria, Illinois (Pettyjohn, date unknown)

Two 10 m deep recharge pits (Figure 2.4) constructed adjacent to the Illinois River can recharge the underlying sand and gravel aquifer at a rate of 17 000 m³/day. The base of the pits are lined with gravel. Originally a 15 cm layer of sand was placed in the pit to serve as a filter media, but rapid clogging of the sand reduced infiltration rates, and the sand was replaced with gravel.

Minot, North Dakota (Pettyjohn, date unknown)

The recharge facility is contained within 3 ha piece of land about 300 m from the source of the recharge water, the Souris River. Raw water is fed to a settling basin, and from there into a canal system. Recharge takes place through thirty six 0.76 diameter wells and four 4 m diameter wells located at the base of a canal. The wells cut through a layer of silt and clay, and are filled with gravel. The maximum recharge rate is 15 000 m³/day.

Leaky Acres, California (Nightingale and Bianchi, 1981; Pettyjohn, date unknown)

This recharge facility which is situated on an alluvial fan, consists of ten large recharge basins covering an area of 47 ha. The water is obtained from the Kings River, and fed via an irrigation canal to the recharge basins. During numerous recharge runs between 1971 to 1975, more than $65 \times 10^6 \text{ m}^3$ of water infiltrated into the aquifer. Raw water with turbidities up to 5 NTU are well tolerated by this recharge facility. A significant reduction in the infiltration rates are experienced when the turbidity of the raw water is greater than 20 NTU.

Highlands Ranch, Colorado (Pyne, 1995)

This example is one of many borehole injection and recovery schemes in the USA. The transmissivity of this confined aquifer which is made up of poorly consolidated sands, is $106 \text{ m}^2/\text{day}$ - one of the lowest transmissivity aquifers where injection is practised in the USA. The storativity of the Aquifer is 0.0003 and the porosity is estimated to be 0.25. The scheme was implemented to meet seasonal peak water demands. Water is injected at 12 l/s to 16 l/s, and is recovered at 20 l/s to 26 l/s. Recommendations after the test recharge runs included: Backflushing every 4 weeks for a period of 4 to 8 hours to maintain the borehole's capacity; Maintaining the pH of the injected water within a range of 7.5 to 8.3 to minimise potential for precipitation of ferric hydroxide and calcium carbonate.

Australia (Gerges, Sibenaler and Howles, 1996)

Although artificial recharge by means of spreading basins is practised in Australia, there is an increase in the use of borehole injection methods - especially in confined limestone aquifers. The transmissivities of the recharged aquifers range from $70 \text{ m}^2/\text{day}$ to $1\,500 \text{ m}^2/\text{day}$. Two cases of relatively low permeability aquifers are described below.

Northfield

The scheme was developed to reduce storm water outflow and provide an irrigation water supply for parklands. Storm water is piped to a wetland detention basin, and after treatment it is gravity injected into a saline, fractured rock aquifer. The transmissivity of this confined aquifer, which consists of fractured slates and quartzites is $70 \text{ m}^2/\text{day}$. Water is gravitated into a borehole at 13 l/s, and it can be abstracted from the borehole at 24 l/s. The main fracture zone at this borehole is at 68 m and the water rises in the borehole to 14 mbgl. The year the scheme became operational (1994), $10\,000 \text{ m}^3$ was injected, followed by $40\,000 \text{ m}^3$ the following year. The native groundwater had a salinity of 2 700 mg/l, and the injected water, a salinity of 360 mg/l. After the initial injection run of $10\,000 \text{ m}^3$, the water quality on abstraction changed from 500 mg/l to 1100 mg/l with repeated pumping periods of six hours. The water quality is expected to improve when much larger volumes of water are injected.

The Paddocks

The aim of this scheme is to store storm water in a confined limestone aquifer. The injection borehole is 164 m deep (open from 134 m), gave a drilling "blow" yield of 10 l/s and the rest water level is 10 mbgl. The transmissivity of the aquifer is $75 \text{ m}^2/\text{day}$. After injection runs, the efficiency of this borehole decreases, and acid is used to break down the chemical precipitates which cause the clogging. A 30% increase in borehole efficiency was observed after 5 000 litres

of 15% hydrochloric acid was injected into the borehole.

India (Athavale, Muralidharan and Rangarajan, 1994)

Groundwater occurring in the weathered and jointed sections of the Deccan Trap basalts are the main source of drinking and supplementary irrigation water for numerous villages near Nagpur. The unconfined basalt aquifers consist of a weathered zone (a few metres to tens of metres thick), followed by jointed and fractured rock to depths of 30 - 40 m. The aquifers have limited lateral extent with spatial variation in thickness. Pre-treated surface water was injected under gravity at an average of 0.8 l/s for 53 days (3 600 m³) to a test borehole. Daily average intake declined from the first day of 1.0 l/s to 0.6 l/s at the end of the injection phase. After a time gap of 70 days, about 75% of this quantity was pumped out at a rate of 1.7 l/s for 18 days - 40% of this water was injected water.

Potassium iodide tracer was used to determine the velocity of subsurface flow in both injection and recovery phases and packers were used to determine the intake capacity (the rate of acceptance of water in litres per minute by the aquifer) at different depths in the aquifer. The results of the study indicated that:

- the intake capacity of different boreholes varied from 0.013 l/min - 2.5 l/min per metre of borehole depth;
- about 75% - 90% of the intake capacity is restricted to the weathered zone;
- hydrofracturing did not produce any significant change in the hydrological properties;
- by pumping (surging) the injection borehole for short durations (1 - 4 hours), the intake capacity was largely restored.

Kuwait (Pyne, 1995)

In the late 1980's research was carried out to assess the potential for storing large volumes of drinking water in brackish aquifers close to demand centres. Several areas were tested in a confined limestone aquifer and an aquifer which consists of sand layers between layers of cemented sandstone. One site, where the transmissivities are relatively low, is at Sulaibiya. The aquifer consists of two productive units: The upper unit has a transmissivity of 12 m²/day and a storativity of 2×10^{-5} , and the lower unit has a transmissivity of 24 m²/day and a storativity of 4×10^{-5} . The borehole consists of 124 m of 400 mm (16 inch) casing, below which is an open hole to 275 m. The rest water level at the time of testing was 37 mbgl. The injection run was carried out for 30 days. The injection rate decreased from 7.5 l/s to 4.5 l/s as a result of clogging with rust and sand in the distribution system. After backflushing a sustained injection flow of 6.7 l/s could be maintained. Following injection, the water was removed at an average rate of 11.3 l/s.

Background TDS in the aquifer was about 5 000 mg/l. Mixing resulted in over 45% of the recharge volume was recovered before the TDS exceeded 2 000 mg/l in the recovered water; and 90% was recovered before the TDS concentration reached 3 000 mg/l.

6.2 ARTIFICIAL RECHARGE SCHEMES IN SOUTHERN AFRICA

Artificial recharge schemes in South Africa range from small scale facilities involving, for example, an earth dam constructed by a farmer to raise the local water table, to large recharge basins like those in Atlantis, which cover an area of approximately 500 000 m².

Table 6.2 lists artificial recharge schemes which are currently operational or which were previously operational, and Table 6.3 lists pilot artificial recharge studies and planned schemes. The lists were compiled by liaising with local geohydrologists from DWAF, research institutions, groundwater consulting companies and a mining company. The lists may not be complete, however, they do show that there are a number of artificial recharge schemes in South Africa, and that the effect of artificial recharge is poorly understood.

6.3 SOUTH AFRICAN GEOHYDROLOGICAL ENVIRONMENTS SUITABLE FOR ARTIFICIAL RECHARGE

South African aquifers vary considerably from those with high a permeability and storativity like the primary aquifers at Atlantis and the Cape Flats in the South-western Cape and some of the dolomitic aquifers in the Northwest Province, to those with low permeability and storativity like most of the hard rock aquifers which are found throughout the country. While aquifers with high permeabilities and storativities are most suitable for receiving recharge water, aquifers with limited permeability and storativity can also be artificially recharged.

In many parts of the country where only hard rock aquifers exist, artificial recharge may be an appropriate method to enhance limited natural groundwater resources. In such areas, average borehole yields may only be in the region of 1 l/s. If these aquifers were artificially recharged at 1 l/s from a number of recharge points, it could make a considerable difference to the exploitable groundwater resource. In order to artificially recharge these aquifers, it may be necessary to consider innovative methods of getting recharge water to bypass impermeable formations or low conductivity soil horizons. In other secondary aquifers where permeabilities are relatively high, such as the dolomites, fractured Karoo and Cape Super Group aquifers, it may be possible to inject substantial volumes of water underground. In these areas, artificial recharge could become one of the key factors in water resource management. For example, groundwater which is usually held in reserve could be used if a reliable water source is available for artificial recharge. In the dolomites, artificial recharge could also be used to prevent the formation of sink holes.

Spreading basins should be considered as the first technology choice so long as the permeability of the soil horizons and aquifer material is sufficient to allow rapid infiltration and percolation. If not, trenches which cut through the impermeable layers, or boreholes which penetrate the most permeable parts of the aquifer, should be considered as the more appropriate artificial recharge methods.

Table 6.4 presents geohydrological environments which may be suitable for artificial recharge and it lists the main factors which will determine the areas' suitability.

Table 6.2 Artificial recharge schemes in Southern Africa

Locality	Aquifer	AR ¹ method	Water source	Description
Atlantis ²	primary dune sands	spreading	treated municipal waste & storm runoff	Two large basins covering an area of approximately 500 000 m ² when full are situated in the dunes and provide artificial recharge to the aquifer some 500 m up-gradient of the wellfield. The recharged quantities are in the region of 2 million cubic metres per annum. The scheme also consists of a series of infiltration basins near the coast which are used for discharge of poorer quality wastewater from the town. This includes saline effluent originating from the regeneration of the softening plant ion exchange resins, treated industrial wastewater and the runoff from the noxious trade area stormwater collection system. This system both provides an environmentally acceptable way of disposing of poorer quality water and also forms a barrier between the wellfield and possible saline intrusion from the sea. The scheme has been in operation for over 15 years. The K-value obtained from double-ring infiltrometer tests which were carried out inside a dry basin which had been in operation for eight years, was 9.5 m/d.
Swakopmund & Central Namib Water Supply Scheme	primary riverine sands	spreading	natural runoff / the Omdel Dam	Recharge into the Omdel aquifer, located in the Omaruru delta, only occurs during flood events. As with many other non-perennial African rivers, the flood waters are heavily laden with silt which clogs the river bed and prevents free infiltration into the aquifer. Nawrowski & Tordiffe ¹ calculated that as a result only 1×10^6 m ³ /y of surface water recharges the aquifer and the remaining $14,6 \times 10^6$ m ³ /y escapes to sea. In order to counter this water loss the Department of Water Affairs constructed the Omdel Dam (40×10^6 m ³) at the head of the delta. The flood waters are stored in this dam long enough for the sediments to settle, after which the water is released down the river to recharge basins located 6 km down stream. The recharge basins appear as earth dams within the river bed, and they overly a minor subterranean channel in the aquifer. This channel feeds into the main aquifer channel on which the wellfield is located. Only one major flood event has occurred since completion of the scheme and it is still too early to gauge how successful the scheme is. It is however interesting to note that after only one flood event the entire floor of the dam already has a 1 to 1,5 m thick silt layer ² .
Pietersburg ³	riverine sands & weathered/fractured gneisses	spreading	treated municipal waste	The Pietersburg municipality pumps about 5×10^6 m ³ /y of treated waste water into the usually dry Sand River. No recharge basin exists, rather, the water runs for a few hundred metres before it all disappears (infiltrates and evaporates). The alluvium in the river bed is about 25 m thick. The maximum the municipality has abstracted from the aquifer is in the order of 3×10^6 m ³ /y. The aquifer is only used in times of need, ie when the surface water supplies are inadequate.

Table 6.2 continued overleaf

Locality	Aquifer	AR ¹ method	Water source	Description
Dendron ⁵	weathered gneisses	spreading	earth dams	Farmers in the Dendron area have built numerous shallow earth dams with the aim of enhancing groundwater recharge. The effectiveness of these dams is not known, however, DWAF is currently setting up systems to monitor their effectiveness.
Springbok flats ⁶	basalts	injection	earth dam	A borehole on a farm south of Roedtan is fed by an earth dam via a pipeline with a control valve. Two water supply boreholes are located on either side of the injection borehole. This scheme was described in 1986, and it is not known whether the scheme is still operational. If the turbidity of the injected water was not reduced prior to injection, it is likely that the scheme is either very inefficient, or that the borehole has clogged up completely.
Soutpansberg	weathered gneisses?	spreading	earth dams	Farmers in the Gordon area of the Soutpansberg District built earth dams in the late 1960's to enhance recharge. De Villiers ⁷ describes water level rises, and in some cases, borehole yield increases after the construction of the earth dams. For example, Borehole No 62 on the farm Cliffdale was reportedly dry when drilled in 1958, but after an earth dam up-slope of the borehole had been enlarged, a yield of 1.2 l/s was recorded, and a water level reading in 1971 was 23 m higher than the original water level. Borehole No 78 on the farm Freyburg had an initial yield of 0.9 l/s. The hole dried up during the dry years of 1965 and 1966. After an earth dam was built up-slope of the borehole, a yield of 1.3 l/s was recorded, and when the water level in the borehole was measured in 1971, it was found to be 21 m above the original water level. Without more information on rainfall, water levels and abstraction, these examples (and the others like these) do not prove the effect of the recharge dams on the aquifers - they do however indicate that there is a likelihood that the dams did have some positive effects.
Rustenburg ⁸	igneous rocks, base of the Bushveld Igneous Complex	injection	spring	A farmer in the Rustenburg area gravitates clear spring water during the summer rainfall period into 4 or 5 boreholes. During the winter, the water is abstracted from the boreholes for irrigation purposes. The boreholes yield (and receive from the springs) on average about 1 l/s, and the farmer says that he can abstract the same amount of water that he injects. The scheme has been operational for 3 years.
Karoo farms	Karoo sedimentary rocks and dolerite intrusions	injection and spreading	earth dams	Farmers in the Karoo have built recharge dams along dykes ⁹ and recharge dams with injection boreholes ¹⁰ . Although the farmers say that the schemes enhance their groundwater resources, none of them are monitored, and therefore it is not possible to say how efficient they are.

Table 6.2 continued overleaf

Locality	Aquifer	AR ¹ method	Water source	Description
Khomas Hochland, Namibia ¹¹	Damara Sequence	spreading	earth dams	Farmers in the upper Kuiseb River catchment have built dams on geological structures for the purpose of recharging the aquifers on which livestock depend. The effect of these dams have not been properly documented.
Karkams	granite	injection	surface runoff	As part of a groundwater supply scheme for this Namaqualand village, DWAF ¹² recommended that one of the boreholes be used for injection and supply. Water from a small stream is fed through a sand filter up-slope of the borehole and then gravitated into the borehole. The scheme has been monitored since it came into operation in 1995 ¹¹ . Two problems have emerged: Firstly, the sand filter was not constructed (or designed?) to maximise filtration and flow efficiency, and is therefore not working properly. The borehole is currently not being recharged because sand and organic matter pass through the filter. Secondly, the rate at which water enters the borehole during artificial recharge is too high. The borehole gave a blow yield of 7.2 l/s during drilling, and was test pumped for 50 hours at 3.2 l/s. Due to the poor recovery after the constant discharge test (a residual drawdown of 3 m), a production yield of 0.3 l/s for 20 hours per day was recommended. The ± 50 mm diameter pipeline from the sand filter allows water to enter the borehole at a greater rate than which the aquifer can receive water, and as a result, the recharge water overflows from the borehole. From the monitored data, which includes abstraction, rainfall, water levels and electrical conductivity, it is difficult to establish to what extent the aquifer receives this recharge water. An observation borehole would need to be drilled near the injection borehole to establish this.
Farmers adjacent to the Mogol River, near Ellisras ¹⁴	alluvium in the Lower Mogol River	spreading	Hans Strydom Dam	Releases from the Hans Strydom Dam recharge the Mogol River alluvial aquifer. Farmers take water directly from the river and from numerous well points located adjacent to the river. The alluvium is estimated to be between 2 m and 10 m deep, with an average depth of 5 m. Yields from the well points vary from 12 l/s to 38 l/s, however these figures may in reality refer to the capacity of the vacuum pumps. Artificial recharge to this aquifer has not been accurately quantified, however, water releases from the dam do not appear to reach the Limpopo River confluence until the sand aquifer throughout the 118 km length of the Mogol channel has been fully recharged. Observations from a dam release in 1987 indicate that the volume of water required to recharge the aquifer was in the region of $10 \times 10^6 \text{ m}^3$.

Key to Table 6.2 is overleaf.

Key to Table 6.2:

- 1 Artificial Recharge.
- 2 Tredoux, G. and Wright, A. H. 1996. Artificial Groundwater Recharge in Southern Africa. Paper presented at the workshop entitled: Groundwater-surface water issues in arid and semi-arid areas. Mabalingwe 16-17 October, 1996;
CSIR. 1989. Atlantis groundwater management programme. Double ring infiltrometer tests of the artificial recharge basin. Internal report. Cape Water Programme. Stellenbosch.
- 3 Nawrowski and Tordiffe. In: Tredoux, G and Wright, AH. 1996. Artificial Groundwater Recharge in Southern Africa. Paper presented at the workshop entitled: Groundwater-surface water issues in arid and semi-arid areas. Mabalingwe 16-17 October, 1996.
- 4 C. Haupt (pers. comm.), Water Systems Management, Pietersburg.
- 5 H. Roux (pers. comm.), DWAF.
- 6 Fayazi, M. 1984. Regional Groundwater Investigation of the Northern Springbok Flats. DWAF Report No. GH 3684.
- 7 De Villiers, S. B. 1971. Verslag oor boorgatresultate en aanvulling van grondwater deur gronddamme in die omgewing van Gordon 310 MS, Distrik Soutpansberg. DWAF Report No. GH 1704.
- 8 S. Van Schalkwyk, (pers. comm.).
- 9 A. Woodford (pers. comm.), DWAF.
- 10 G. Van Tonder (pers. comm.), Institute for Groundwater Studies.
- 11 E. Braune (pers. comm.), DWAF.
- 12 Esterhuyse, C. J. 1990. Namakwaland-Sentraal: Ondersoek na die Grondwaterbronne te Leliefontein-Wes as Aanvullende aan Karkams. DWAF Report No. GH 3685.
- 13 Toens and Partners. 1997. Grondwater monitering en bestuur van waterstelsels in Namakwaland. Vorderingsverslag No. 19 (April 1997 - June 1997).
- 14 Vipond, S. H. 1987. Groundwater potential of the Lower Mogol Channel, Review of existing data. DWAF Report No. GH 3519.

Table 6.3 Pilot artificial recharge studies and planned schemes

Locality	Aquifer	AR ¹ method	Water source	Description
Cape Flats ²	primary dune sands	spreading and injection	treated effluents	<p>Five recharge basins were constructed on the perimeter of a 200 m diameter circle with two abstraction boreholes (one 20 l/s and the other 15 l/s) situated five metres apart near the centre of the circle. One 200 mm diameter injection borehole was drilled next to one of the recharge basins. A network of 30 observation boreholes were installed at the site. The floor area of the recharge basins varied from 156 to 400 m² while the thickness of the unsaturated zone below them ranged from 3 to 13 m. The total saturated thickness of the aquifer varied from 28 to 37 m.</p> <p>In order to obtain reference values for the recharge capacity of the basins, groundwater with a turbidity of 0.6 NTU was used as a recharge source. Recharge rates varying between 5 and 9 m/d were recorded for runs lasting up to six weeks. The recharge rate for a given basin remained practically constant during these tests, even though algae species belonging to the genus <i>Mougeotia</i> established themselves about two weeks after commencement of the recharge. Infiltration runs using partially renovated oxidation pond effluent with a turbidity below 2 NTU was successful and proceeded at rates similar to those observed with groundwater. Turbidity values between 3 and 5 NTU, however, caused rapidly declining rates and complete clogging of the surface within 14 days. Manual removal of the top layer of sand (50 to 80 mm thick) restored the original recharge rates. Allowing the basins to dry out for approximately a week had practically the same effect but subsequent clogging occurred much sooner. On the whole it was concluded that under the conditions at the experimental site, turbidity of the feedstock was the most important factor affecting recharge rates.</p> <p>Chloride, total nitrogen and potassium concentrations in the recharged water differed significantly from those in the indigenous groundwater. Therefore, these constituents were used as tracers to monitor the subsurface movement of the recharged water. Chloride was the only conservative tracer. Potassium interacted with the geological material and its concentration decreased after infiltration. In the case of ammonia, fairly high concentrations were retained in the unsaturated zone, where it was partially or completely converted to nitrate by nitrifying bacteria after cessation of recharge. Porous cup lysimeters monitoring the unsaturated zone indicated a higher nitrate concentration for a short while after recharge was resumed. Bacterial indicator organisms were also monitored and only at very high recharge rates of 9 m/d were <i>E. Coli</i> found at distances of up to 27 m from the recharge basins while recharge was in progress.</p> <p>The conclusion was that artificial recharge of treated effluents through basin recharge was demonstrated to be feasible. Borehole injection was carried out continuously for a period of 8 days at a rate of approximately 8 l/s. Although this seemed to be successful, the turbidity of the water varied too much to produce any reliable results on borehole injection and the experiments were discontinued.</p>

Table 6.3 continued overleaf

Locality	Aquifer	AR ¹ method	Water source	Description
Maun, Botswana ³	primary riverine sands	spreading	Groundwater was used in the pilot study, but natural runoff will be used for production.	<p>The upper, unconfined aquifer in the Shashe River, dewatered by abstraction for Maun water supply, is considered to be replenished by artificial recharge in those years that natural floods do not reach the area. A 20 m by 20 m pilot-scale basin was constructed in the Shashe River and the system was tested using groundwater from the deeper aquifer, as it was the only locally available water at the time.</p> <p>The pilot test indicated an infiltration rate of 2.1 m/d measured over a seven-day period. Hydraulic simulation modelling enabled extrapolation of the experimental data and a production AR site consisting of several basins recharging a total of 2×10^6 m³/a (500 000 m³/month for the four months water is available) has been recommended. The water source will be surface water from the Thamalakane and/or the Boro River. The recharged water will also seep into the semi-confined aquifer below.</p>
Rössing Uranium Mine, Namibia ⁴	alluvial and unconsolidated river bed sediments	spreading	Flood water in the Khan River stored temporary in an earth dam	<p>A dam with a capacity of approximately 9 million m³ is being planned for the ephemeral Khan River on the West Coast of Namibia by Rössing Uranium Limited. The sole purpose of this dam will be to capture flood water and act as a temporary storage reservoir. Once the high silt load of the flood waters has settled, the water will be released to a system of cross-bunds. The cross-bunds will be constructed in the alluvial river channel below the dam wall, and they will detain the water and enhance infiltration.</p> <p>This project is expected to allow abstraction to increase from the current approximately 0.24 Mm³/year to 1.28 Mm³/year from the alluvial Khan River aquifer.</p>

Table 6.3 continued overleaf

Locality	Aquifer	AR ¹ method	Water source	Description
Windhoek, Namibia ^{1,4,6}	fractured quartzites	injection	Chlorinated groundwater from the same aquifer was used for the pilot study. The aim is to use treated dam water for production purposes.	<p>Windhoek depends on three sources of water, namely surface reservoirs built in ephemeral rivers, groundwater and water reclamation from domestic waste. During 1997 the evaporation from the surface reservoirs was $37 \times 10^6 \text{ m}^3$, while the consumption was $15.7 \times 10^6 \text{ m}^3$. The aim of artificial recharge to the Windhoek secondary aquifer is to minimise the losses associated with evaporation. The volume of water stored in the aquifer, and thus the artificial recharge potential, is believed to be in the region of $0.6 \times 10^6 \text{ m}^3$ per metre (vertical) of the aquifer.</p> <p>The first pilot artificial recharge study was carried out in 1996/7, and a second study is planned to commence in 1998. The second study will involve injecting treated water from the surface reservoirs for a duration of several months. The results of the first pilot study are described below.</p> <p>A relatively low yielding, currently unused borehole was used for the first pilot study so that the water supply from the high yielding production boreholes would not be disturbed. The injection run was carried out for 3 weeks at an average of 2.8 l/s, followed by a three week recovery period. The water level in the borehole rose by 6.7 m during the test, and at the end of the recovery period, the residual drawdown was 0.9 m above the original rest water level. The transmissivity of the aquifer, as obtained from the injection borehole and observation boreholes is $\pm 13 \text{ m}^2/\text{day}$; and the storativity, ± 0.033. The injection rate under gravity flow (2.8 l/s) was higher than the blow yield of this borehole (1.3 l/s) and the maximum production yield of the borehole (2.2 l/s).</p>
The Nyl River valley, near Potgietersrus	alluvium and weathered basalt	injection	floodwater	<p>One of the aims of this study was to assess the potential for artificial recharge using floodwater from the upper Nyl River and the Klein Nyl River. The aquifer consists of heterogeneous alluvial deposits underlain by weathered and fractured basalts and sandstones. The alluvium consists of sand, gravel, silt and clay; and their thickness varies from 5 m on periphery to a maximum of 35 m in the centre of the basin.</p> <p>The pilot artificial recharge study was not successful because: the aquifer was full at the start and could not accept more water, and the rate of inflow (into the boreholes) was low as a result of the fine grained sediments⁸.</p>

The key to Table 6.3 is overleaf

Key to Table 6.3:

- 1 Artificial Recharge
- 2 Tredoux, G. and Wright, A. 1996. Artificial groundwater recharge in South Africa. Paper presented to the Department of water Affairs and Forestry workshop: Workshop on Groundwater-Surface water issues in arid and semi-arid areas. Warmbaths, October 1996.
- 3 Eastend Investments. 1997. Maun Groundwater Development Project. Appendix E: Artificial Recharge Feasibility. Report to Department of Water Affairs, Contract TB 10/3/26/94-95, Gaborone, Botswana.
- 4 CSIR. 1997. An assessment of the Potential Environmental Impacts of the proposed aquifer Recharge Scheme on the Khan River, Namibia. Final Report by the Division of Water, Environment and Forestry Technology to Rössing Uranium Limited, Swakopmund, Namibia. Report No. ENV/P/C-97104, xxxvi + 172 pp. + 7 Appendices.
- 5 Carr Barbour & Associates. 1997. Results of a borehole injection test carried out to investigate the technical feasibility of artificial recharge by means of borehole injection (Draft). Report to the City of Windhoek.
- 6 B. Van der Merwe (pers. comm.), City Engineer (Water Services), Windhoek.
- 7 Porszasz, K. and Bredenkamp, D. B. 1973. An investigation of the groundwater potential of the Nyl River Valley. Department of Water Affairs, Division of Hydrology, Hydrological Research Institute.
- 8 Z.M. Dziembowski (pers. comm.), DWAF.

Table 6.4 Geohydrological environments suitable for artificial recharge

Possible South African environments	Key prerequisites for application
Induced recharge schemes (eg. river bank filtration)	
Perennial rivers flowing over sandy flood plains. Fresh water lakes with surrounding sandy soils.	River water with little suspended matter; Permeable river beds/banks; Permeable riverine sands or gravels extending away from the river; Regular & sufficient flows; Resources available for river bed/bank scraping.
Infiltration schemes (eg. infiltration basins, land flooding and subsurface trenches)	
Thick sand deposits, for example in ephemeral river beds, buried valleys and coastal dunes. Weathered and fractured rock aquifers overlain by sandy soils or gravels (eg. weathered granites).	The quantity and reliability of the raw water must be sufficient; The raw water must be of adequate quality - the turbidity of the water must be low so that the clogging rate is acceptable; and in the case of land flooding, the salinity of the water must be relatively low so that the rate of salt accumulation at the soil surface is acceptable; Resources must be available to clean basins of clogging material; The soils must be sufficiently permeable with no continuous impermeable layers; The aquifer must have sufficient storage available, and have sufficient permeability to accept the recharge water; Land must be available.
Borehole injection	
Permeable secondary aquifers such as those associated with: dolomites; fractured or brecciated fault zones; linear or circular intrusive rocks; folded rocks with intense jointing; and aquifers with permeable bedding planes. Primary aquifers, for example basal conglomerates. The aquifers can be confined and deep seated.	The quantity and reliability of the raw water must be sufficient; The turbidity of the injected water must be very low in order to minimise clogging; The chemistry of the injected water should be such that it does not lead to excessive mineral precipitation within the aquifer; The aquifer must be sufficiently transmissive and have available storage in order to receive the water; Injection schemes require relatively advanced management.
Artificial aquifers (sand storage dams)	
Rugged, arid areas with high run-off and ephemeral sandy river beds.	Hilly or mountainous topography with well defined valleys. Arid area with high runoff resulting in ephemeral river flow. The predominant parent rock in the area should weather to produce a coarse sandy sediment (eg. granites, sandstones, quartzites) which will act as the aquifer material. The bedrock in the valley should be able to provide a solid rock foundation for the wall. The dam should be underlain either by a low permeability bedrock to prevent seepage losses, or bedrock which hosts a secondary aquifer to be recharged by the sand dam. Building material for the wall should be readily available (eg. hand sized rocks for stone masonry) as construction of the wall is an ongoing process (raised after each flood).

Chapter 7

POSSIBLE PILOT ARTIFICIAL RECHARGE SCHEMES

An objective of this study was to conceptually design one or more pilot artificial recharge schemes which could be used to demonstrate the applicability of artificial recharge in South Africa. In order to identify possible sites, a number of people (mostly geohydrologists) were contacted. These people are listed in the acknowledgements at the beginning of this report.

7.1 SUGGESTED PILOT ARTIFICIAL RECHARGE SITES

The potential artificial recharge sites which were considered for further study are shown in Figure 7.1 (excluding Windhoek in Namibia) and listed in Table 7.1. Although a number of other towns were suggested by the geohydrologists that were contacted, they have not been listed because insufficient information on these sites was obtained. The towns listed in Table 7.1 are areas where artificial recharge may be possible. In all cases, however, further investigations, including field studies are required in order to establish whether they are indeed suitable for artificial recharge.

Out of the ten suggested sites, six have been identified for possible further study. These are Kenhardt, Calvinia and Williston which are described in greater detail in sections 7.2 - 7.4; Karkams and a farm near Rustenburg, which are described in Table 6.2; and Windhoek which is described in Table 6.3.

The six sites selected for further study are all associated with secondary aquifers, and they have potential for success in terms of the following factors:

- The availability of water for recharging the aquifer;
- The hydraulic characteristics of the aquifer;
- The quality of the recharge water;
- Minimal clogging of the recharge basins, trenches or boreholes;
- Groundwater recovery;
- Economic factors;
- Management requirements.

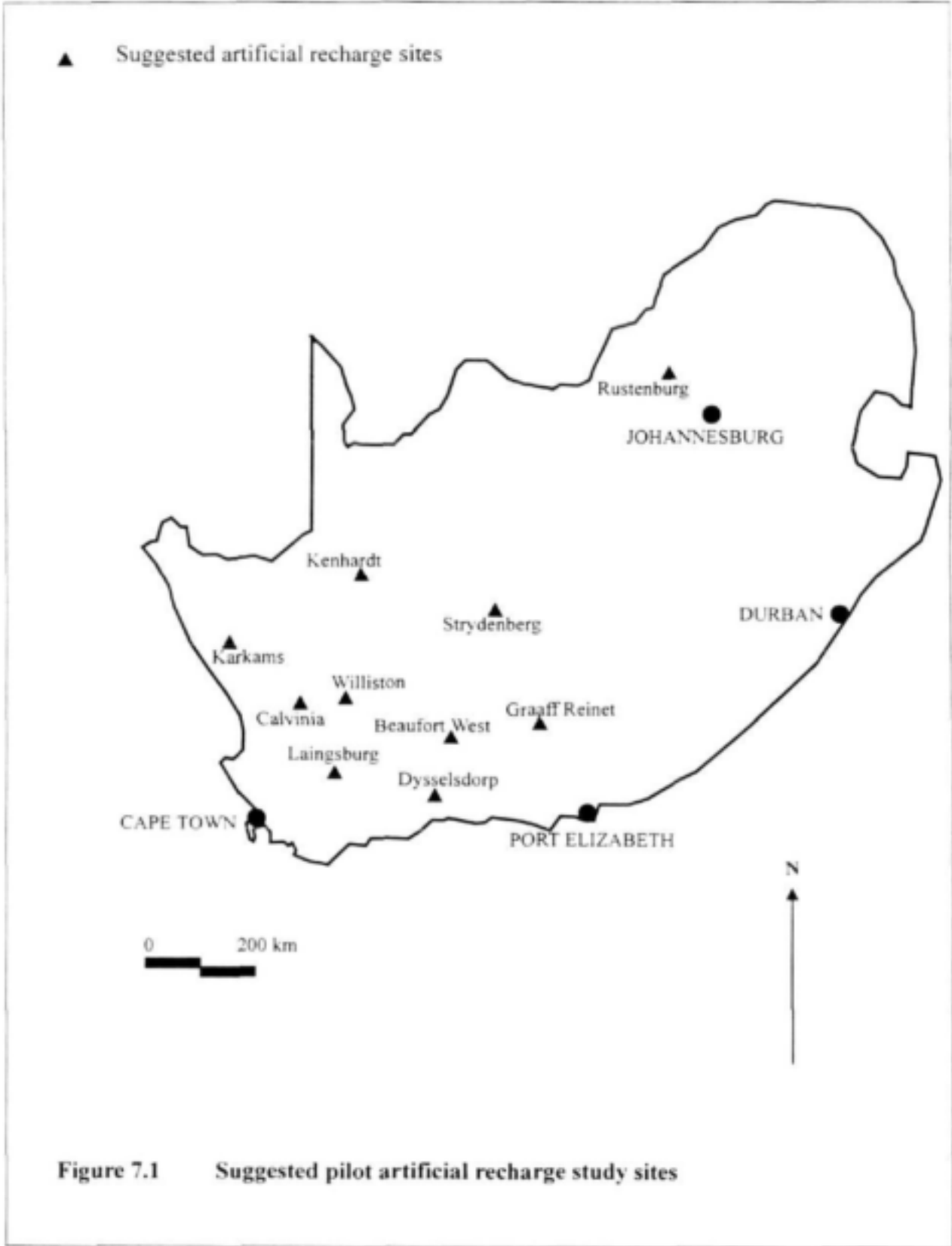


Table 7.1 Suggested artificial recharge pilot study sites

Town	Aquifer	AR ¹ method	Water source	Comments
Kenhardt	Alluvium and weathered/fractured gneiss	spreading	Rooibergdam which usually fills during winter and is often dry during summer	<i>This case study would test the use of dam water for AR in typical weathered and fractured gneisses overlain by alluvium.</i> Kenhardt depends on a wellfield for all their water requirements. The wellfield cannot yield the seasonal peak demand, and it will not be able to meet future demands. AR, which was recommended by DWAF ² , is probably the cheapest way to solve Kenhardt's water resource problems. The more expensive alternatives include: develop a known aquifer 18 km from town; develop a known, but saline aquifer within 2 km from town - desalination would be required; develop a known, deep-seated aquifer 15 km from town; and pipe water 75 km from the Orange River. An advantage of using Kenhardt as a pilot AR site is that an established monitoring system is in place and historical water level and abstraction data exists. This information would be useful in assessing the impact of an AR scheme. The conceptual design of this scheme is described in section 7.2.
Calvinia	Breccia plug in Karoo Sequence	injection	groundwater (existing boreholes) & the Karee dam	<i>This study would test the use of a transmissive groundwater compartment for surplus water storage.</i> A new wellfield has recently been developed to augment the town's water supplies. With little additional expense, a large groundwater storage system could be developed. The aim would be to transfer both groundwater from an aquifer which is annually recharged, and treated water from the Karee dam, when available, to a groundwater compartment which is not regularly recharged. This water would then be available for use in times of high demand. Boreholes and monitoring systems exist. The conceptual design of this scheme is described in section 7.3.
Williston	Horizontal fracture in Karoo Sequence	injection	groundwater (existing borehole)	<i>This study would test the concept of AR to a confined aquifer with a horizontal fracture pattern.</i> Williston will soon need more water to meet their domestic water needs. An aquifer adjacent to the currently used aquifer could serve as an ideal recharge source. The possible AR system is conceptually extremely simple and may not require pumping. Williston's alternative is to pump groundwater about 8 km, which is an expensive alternative to this proposed AR scheme (which is described in section 7.4). Monitoring has taken place for 14 years.
Karkams	Granite	injection	surface runoff	<i>This study would establish the effect of injecting filtered surface water into a low yielding borehole.</i> The existing scheme is described in Table 6.1. Because the water is gravitated at far too high a rate into the borehole (so that the water flows out the top of the borehole), and because there are no observation boreholes, it is difficult to establish the effect that this scheme is having on aquifer recharge. If this scheme were to be chosen as one of the pilot studies, it would be necessary to re-design it so that its effect on the aquifer can be established. The design is described in section 7.5.

Table 7.1 continued overleaf

Town	Aquifer	AR ¹ method	Water source	Comments
Rustenburg farm	Igneous rocks, base of the Bushveld Igneous Complex	injection	spring	<i>This study would establish the effect of injecting spring water into low yielding boreholes.</i> The existing scheme is described in Table 6.1. The farmer who operates this scheme says that he recovers what he injects. It would be interesting and of use to geohydrologists to quantify the effect of this scheme on aquifer recharge. This study, if accepted as one of the pilot projects, would involve establishing a scientific monitoring system so that the effectiveness of this scheme can be established. This is discussed in section 7.6.
Strydenburg	Calcrete with solution cavities	spreading	natural runoff	<i>The study would test the use of natural runoff in a semi-arid environment to recharge a shallow secondary aquifer.</i> This site was initially thought to be suitable to test whether a relatively small rainfall event could be turned into a recharge event. The high permeability and storativity of the aquifer is suitable for such a study, and a number of existing boreholes make up a good monitoring system. After studying the rainfall and water level data however, it became apparent that the natural recharge systems are operating very efficiently, and that there is no need for artificial recharge.
Laingsburg	Alluvium	spreading	natural runoff	<i>The study would test the use of natural runoff in a semi-arid environment to recharge shallow primary aquifers.</i> The CSIR ³ recommends AR, and this site would be suitable to test whether a relatively small rainfall event could be turned into a recharge event. A monitoring system exists. This site was not pursued further because the researchers felt that the pilot study should take place in a secondary aquifer.
Dysseldorp	Cape Supergroup quartzites	injection	natural runoff	<i>This study would test AR from natural runoff to a deep seated fractured rock aquifer.</i> It was initially suggested that the sustainable yield of the existing wellfield could be substantially increased if runoff during rainfall periods could be stored in the aquifer. From discussions with staff from Overberg Water, it became apparent that there is negligible runoff in the stream next to which the boreholes are located. The runoff rapidly enters base flow of the stream, and due to a vertical fracture pattern, it also recharges the aquifer fairly efficiently. The concept of artificially recharging this aquifer has not been rejected, rather, more information is required in order to establish whether it remains a possibility.

Table 7.1 continued overleaf

Town	Aquifer	AR ¹ method	Water source	Comments
Beaufort West	Fractured Karoo	injection	treated municipal waste & surplus dam water, when available	<i>This study would test the potential for borehole injection in a fractured Karoo aquifer with waste water or treated dam water.</i> Municipal waste water is currently treated to irrigation standards, and may not require extensive further treatment before injecting it into the aquifer. Although the town engineer favours this idea, the aquifer into which water would be pumped is currently under utilised, and therefore it is not feasible to artificially recharge it. An alternative would be to inject treated surplus surface water, when available, into the domestic water supply aquifer. This aquifer is often utilised to its capacity, and therefore frequently has storage space available. The town engineer is considering this as a future water augmentation option.
Graaff Reinet	Fractured Karoo	spreading	natural runoff	<i>The study would test the use of natural runoff in a semi-arid environment to recharge fairly shallow fractured Karoo aquifers.</i> The capacity of the Mimosadale wellfield is limited by natural recharge. Recharge dams in and above the wellfield would need to be constructed in order to test the AR concept. A monitoring system (since 1991) exists.
Windhoek	Fractured quartzites	injection	treated dam water	<i>The study would test injection of treated surface water into a highly fractured, secondary aquifer.</i> Artificial recharge to the Windhoek aquifer is described in Table 6.3. Data on groundwater levels and abstraction have been recorded over more than 30 years.

Aquifers not mentioned in the table, but which may be suitable for artificial recharge include dolomitic aquifers, and thick alluvial deposits in the Limpopo River.

KEY

- ¹ Artificial recharge;
- ² Nonner, J. C.1979. Groundwater resources investigation for Kenhardt Municipality, Cape Province. DWAF, Report No. GH 3102.
- ³ CSIR, 1997. Laingsburg Water Supply: Soutkloof and Buffels River.

7.2 KENHARDT: INFILTRATION BASIN PLAN

Kenhardt is likely to face severe water shortages in the near future. Domestic water is currently obtained from the Driekop wellfield which has sufficient capacity to supply the current average winter demand of 300 m³/day, but not the peak summer demand of 1 000 m³/day. The severity of this problem is clearly shown by the dramatic drop in the water table since 1991 (Van Dyk, 1994). There are a number of water sources which could be used to meet the town's additional water needs. Probably the cheapest way would be by implementing an artificial recharge scheme using an existing dam which is generally not used because it is unreliable as a perennial water source. Water from this dam, when available, would be treated and piped to spreading basins in the existing production wellfield. This section describes the possible artificial recharge scheme; and a summary of the liaison between the relevant role-players is provided in Appendix 1.

7.2.1 Kenhardt's projected domestic water demand

Kenhardt's population in 1982 was 3 100 and in 1990 it was 3 600 (Van Dyk, 1994), giving a population growth of 1.9%. The volume of water pumped in 1993 was 232 870 m³, or 638 m³/d (Van Dyk, 1994). Using this daily average consumption, the water demand for Kenhardt in fifteen years from 1993, that is in 2013, will be in the region of 1 300 m³/d or 475 000 m³/a; and the peak demand in summer will likely be closer to 2 000 m³/d. This is possibly a worst case scenario, considering that tariff restructuring and water conservation education is increasingly forming a part of South Africa's water management plans.

If by 2013, 1 300 m³/d is needed, then an additional 260 000 m³/a would be required, assuming that natural recharge to the current production wellfield is in the region of 215 000 m³/a (Van Dyk, 1994).

7.2.2 Water source options

Kenhardt's domestic water supplies can be augmented in a number of ways. These are summarised in Table 7.2. These options, together with the following water conservation measures have been presented to the Kenhardt municipality:

- a) Run an education programme to encourage residents to minimise summer consumption.
- b) Increase the cost of water during summer months.
While some form of tariff restructuring may be inevitable, it may not be necessary to dramatically raise the cost of water if long-term solutions to increasing the water resources can be found.

Table 7.2 Options for augmenting Kenhardt's domestic water supplies

Options	Capacity m ³ /day	Capital and Labour Costs	Comments
Treat Municipal Waste water to potable standards	200 to 300	~ R 4.5 million	The amount of municipal waste water is insufficient to meet peak and future domestic water needs.
Treat water from the Rooibergdam, ~ 4 km from Kenhardt.	1000	~ R 2.5 million	The Rooibergdam is not reliable as a daily source of water.
Transfer water from the Orange River via a ~ 70 km pipeline	1000	~ R17 million	The cost of this scheme is very high.
Develop a wellfield in the Rietfontein River basin ~ 15 km from Kenhardt	1000	~ R 4 million	This option is expensive to develop, since it will require borehole siting, drilling and test pumping. The running costs of pumping water this distance will be high.
Develop a wellfield in the Hartebees River basin ~ 1 km from Kenhardt.	1000	~ R 2.5 million	The groundwater in this aquifer has a high salinity (EC values from 300 - 1 500 mS/m), and would need to be desalinated prior to domestic consumption. The construction, and operation and maintenance costs for the necessary water treatment works would be expensive.
Develop a wellfield (deep aquifer) using ~ 3 deep boreholes ~ 15 km from Kenhardt	1000	~ R 4 million	The reliability of this aquifer needs to be established. The running costs of pumping water this distance will be high.
Artificial recharge to the Driekop wellfield - when water is available from the Rooibergdam	1000	~ R 1.5 million	This option has been recommended previously (Nonner, 1979), and it still appears to be the most cost effective option - both in relation to implementation (construction) costs and running costs. Because the Driekop wellfield is currently being used for production purposes, and because current abstraction is limited by recharge and not aquifer permeability, no new production boreholes are required.

7.2.2 The water source for artificial recharge - the Rooibergdam

The Rooibergdam, which was constructed in 1900, is also referred to as the Rooidam and the Kenhardt Dam. It has the following characteristics:

	Surface area (ha)	Net capacity (x 10 ⁶ m ³)
1935	490	7.1
1983	298	3.7

This shallow dam covers an area of approximately 3 km by 1 km when full, and is usually dry for several months of the year. When water is available in the dam, it could be used for artificially recharging the Driekop aquifer. The only other option would be to use municipal waste water. This water, however, is not available in sufficient quantities to meet the required water demand, and therefore it does not warrant the additional cost of further treating it to acceptable standards for groundwater recharge.

Water from the Rooibergdam, when available, would be gravitated down the existing open canal to Kenhardt; it would be filtered, and pumped to the Driekop aquifer (Figure 7.2).

Reliability of the Rooibergdam water

The reliability of the Rooibergdam is currently being investigated by DWAF (Van Dyk, pers comm). Water level data has been recorded from 1935 to 1974. Figure 7.3, supplied by Mr Van Dyk, gives an indication of the reliability of the dam.

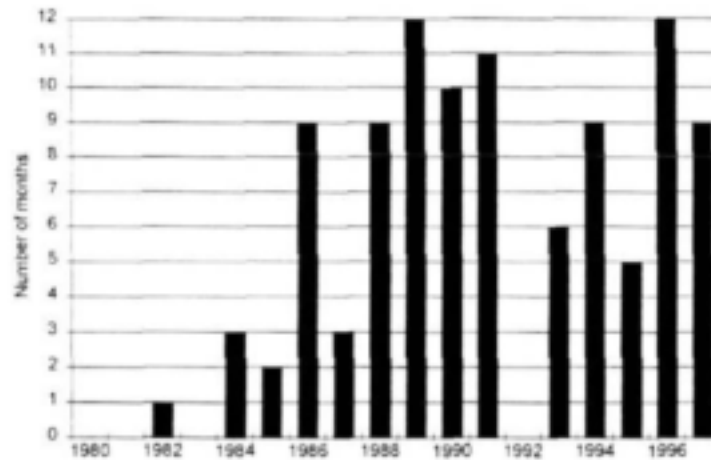


Figure 7.3 The number of months per year, since 1980, that the Rooibergdam has had water

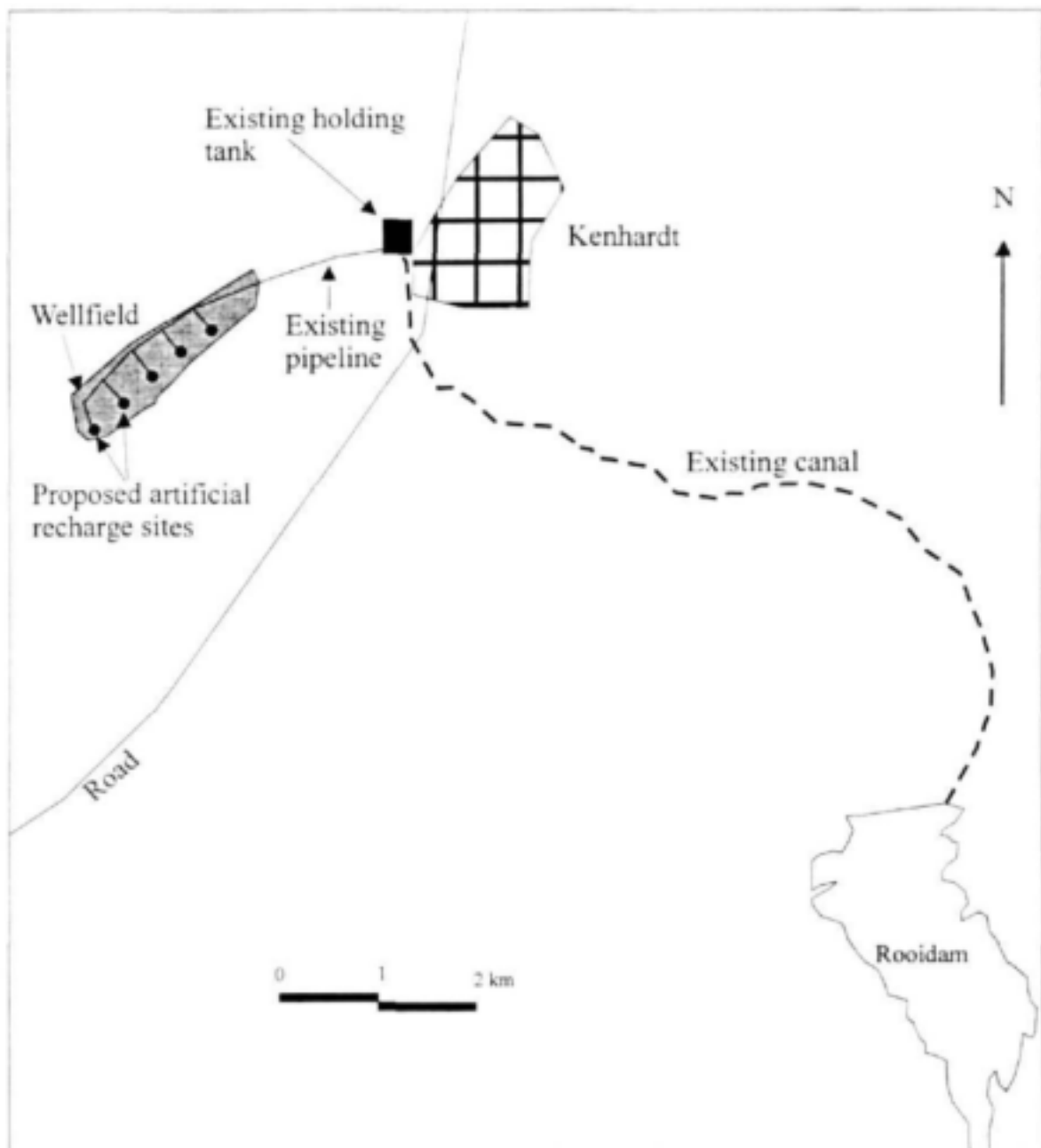


Figure 7.2: Driekop wellfield showing possible artificial recharge source

Quality of the Rooibergdam water

The three main water quality concerns relate to the amount of suspended solids, the salinity and the bacteriological quality of the water. Since the proposed method requires infiltration through 5 - 10 m of alluvium, and the abstraction points will be located at least a hundred metres from the infiltration basins, it is likely that most micro-organisms will die off prior to reaching the abstraction boreholes. The water from the aquifer is also chlorinated before it is distributed through the reticulation system.

Suspended solids, either measured in mg/l or represented by the turbidity of the water, has not been analysed on a regular basis. In June 1997, a water sample was collected from the surface of the dam, and the turbidity of this sample was 68 NTU. The turbidity of the water is likely to fluctuate substantially in relation to its standing time in the dam. It is estimated, that the turbidity will have to be reduced to below 2 NTU before it can be pumped to the infiltration basins - this, however, will need to be determined by conducting a pilot artificial recharge study.

The major anions and cations have been analysed on a regular basis by DWAF since 1989. Figure 7.4 shows how electrical conductivity (EC) has fluctuated over the past eight years. The EC generally falls between 25 - 100 mS/m, with the average being 68 mS/m. Like turbidity, EC of the dam water would have to be monitored so that higher salinity water is not allowed to enter the recharge facility.

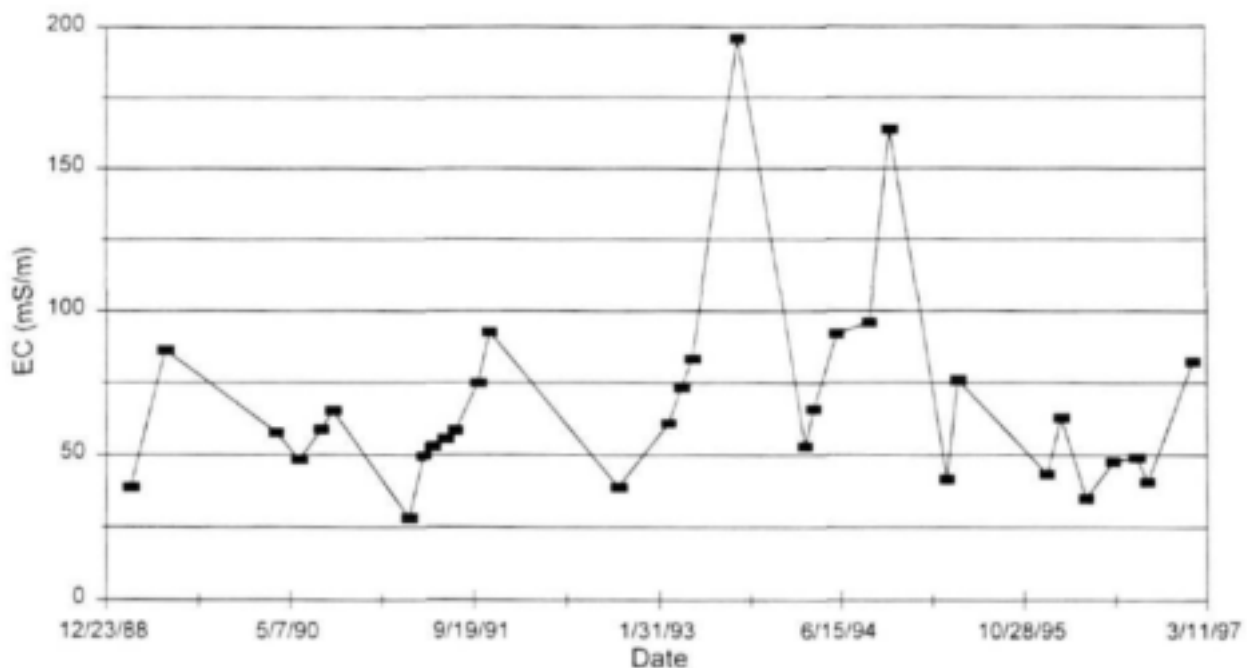


Figure 7.4 Variation of electrical conductivity in the Rooibergdam with time (Div. Geohydrology, DWAF)

7.2.3 The Driekop aquifer

The Driekop aquifer has been described in detail by Nonner (1979) and Van Dyk (1994). Extensive geophysical surveys including resistivity, electromagnetics and magnetics have been done in the Lower Driekop River Basin, and more than 40 boreholes have been drilled. The aquifer consists of alluvium which range up to 11 m in thickness; weathered gneiss of the Namaqua Metamorphic Complex, which range up to a thickness of 30 m; and fractured gneiss which have water bearing fractures up to a depth of about 50 m.

Based on drilling yields, Nonner (1979) describes the alluvium as either being dry or low yielding (5 - 50 m³/day); the weathered gneiss as being the main water yielding formation - especially where water strikes are located near the top and bottom contacts of the weathered rock; and the unweathered gneiss as giving high yields if the boreholes penetrate fractures.

The water level in the aquifer prior to development in the late 1970's / early 1980's lay between 3 - 6 metres below ground level (mbgl), which was within the alluvium or at the contact between the alluvium and the weathered rock. Currently, the water levels lie between about 10 - 14 mbgl.

Aquifer storage

Nonner (1979) used drilling logs, geophysical data and test pump data to estimated the volume of water held in storage. The specific yield (or "unconfined storativity") for the alluvium was taken to be 0.081; and the specific yield of the weathered rock was taken to be 0.025. The specific yield of the weathered rock is a rough approximation based on the average specific yield of the upper strata (0.042) and on resistivity data. The resistivity data shows that specific yields decreases with depth within the weathered rock, hence the lower assumed "averaged" value.

Nonner (1979) estimated the area covered by alluvium to be 658 000 m² and the area covered by weathered rock to be 2 700 000 m². The volume of water held in storage was estimated to be just under 1 000 000 m³. This is equivalent to four times the volume pumped in 1993, and twice the projected 2013 demand.

Based on these estimates, it is clear that this aquifer has the potential to hold sufficient water to meet Kenhardt's current and future water demands.

Natural recharge

Nonner (1979) obtained the following recharge values:

Water level-abstraction method:	> 22 000 m ³ /a
Throughflow method (Darcy's Law):	110 000 - 208 000 m ³ /a
Water level rise method:	290 000 m ³ /a

Van Dyk (1994) describes a rainfall recharge relationship which was developed after studying rainfall, water level and abstraction data:

$$\text{Recharge} = 0.064 (\text{Rainfall} + 21) \text{ [mm]}$$

Using this relationship, Van Dyk estimates the annual recharge, based on a mean annual precipitation of 147.8 mm/a, to be in the region of 215 000 m³/a. This is approximately 15 000 m³/a short of 1983's abstraction from the Driekop wellfield, and 260 000 m³/a short of the projected demand in 2013.

Aquifer permeability

In order for additional water to enter and spread within the aquifer, the aquifer must be sufficiently permeable. An indication of the horizontal permeability has been established from borehole drilling yields and from test pumping, but permeability in the vertical direction still needs to be determined in order to establish whether infiltration basins would be suitable for artificial recharge.

Drilling results from Nonner's study (1979) show that all boreholes struck water, and that nine out of the fifteen boreholes drilled in the Driekop aquifer gave drilling yields in excess of 1 l/s. The potential production boreholes were test pumped (constant discharge tests) at yields ranging from 0.7 l/s to 3.6 l/s; and the transmissivity values obtained varied from 2 - 150 m²/day.

Van Dyk (1994) describes the test pump results for additional boreholes which were drilled in order to increase delivery to Kenhardt. The blow yields of these boreholes ranged from 1.9 l/s to 13 l/s, and the yields used during the constant discharge tests ranged up to 10 l/s. The transmissivity values obtained ranged from 5 m²/day to 390 m²/day.

Vertical permeability needs to be determined by field tests, including infiltration tests, and possibly laboratory tests. Nonner (1979) took soil samples at three sites, and during the course of this study a additional soil samples were collected at one site (Figure 7.5). Particle size analyses were performed on the samples, and the results have been plotted on a trilinear diagram (Figure 7.6). A relative assessment of how these particle sizes relate to hydraulic conductivity can be obtained from Figure 7.7.

Figure 7.6 shows that grain size varies throughout the wellfield, and with depth. It also shows that certain areas will be better suited for an infiltration basin than others. For example the up-slope area around boreholes G42300 and G27974 are better suited for infiltration than the down-slope areas around boreholes G27966 and G27973.

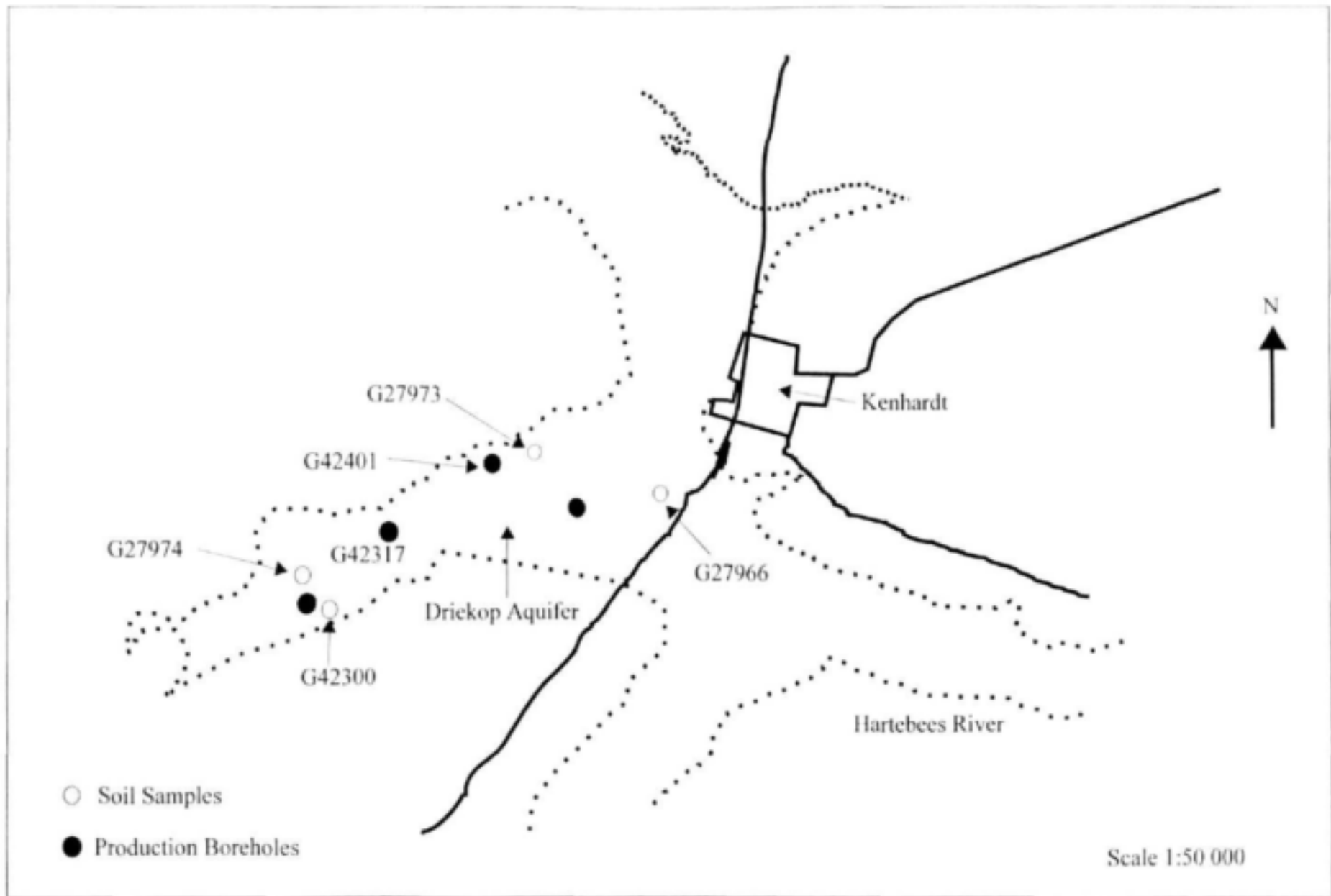
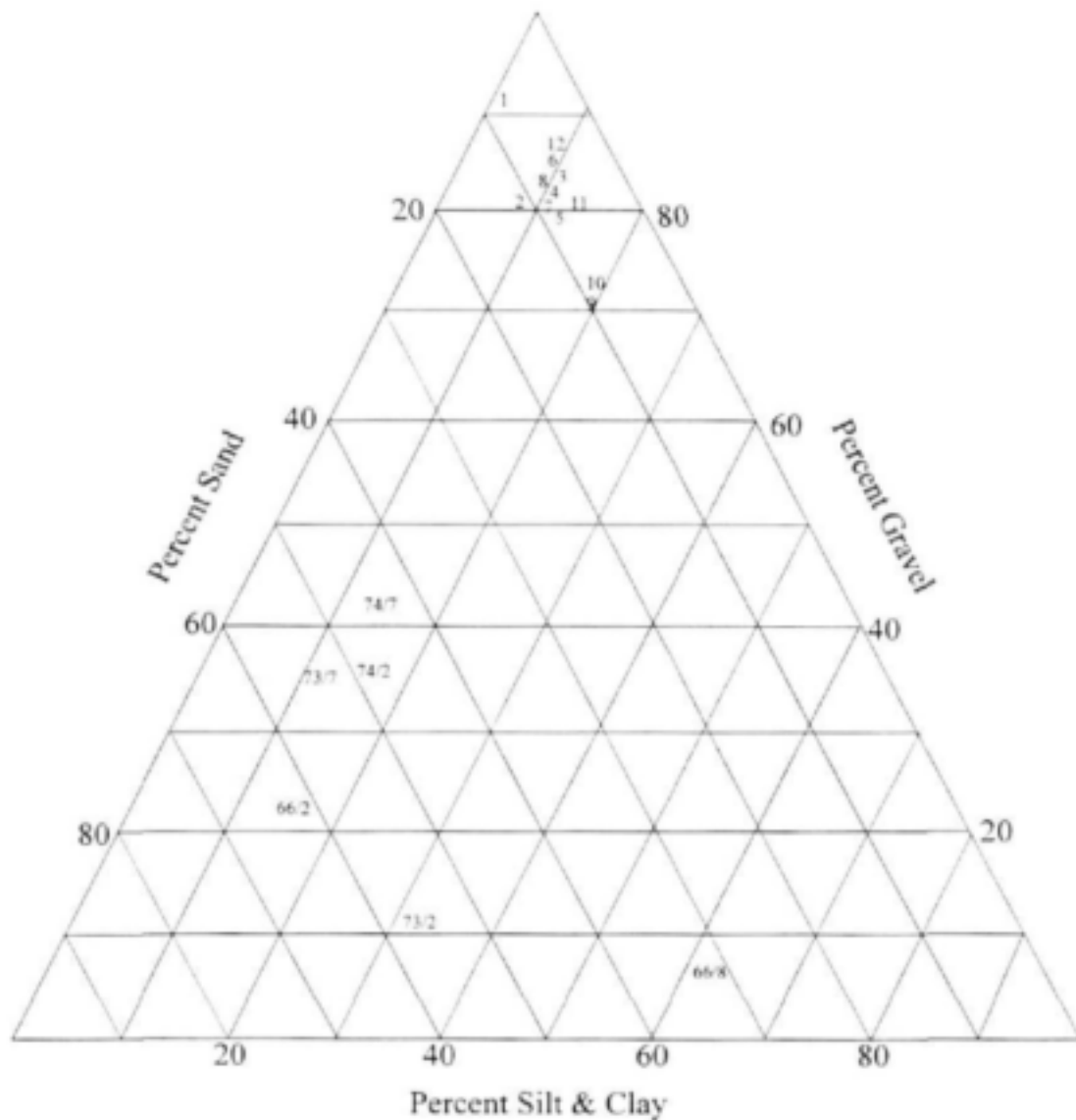


Figure 7.5 Location of soil samples



Near Borehole G42300

No. Depth (cm)

1	0 - 40
2	50 - 65
3	50 - 100
4	180 - 190
5	110 - 130
6	130 - 150
7	190 - 220
8	220 - 250
9	250 - 295
10	290 - 325
11	325 - 390
12	370 - 425

66 / 2: G27966 at 2m

66 / 8: G27966 at 8m (below water table)

73 / 2: G27973 at 2m

73 / 7: G27973 at 7m (below water table)

74 / 2: G27974 at 2m

74 / 7: G27974 at 7m (below water table)

Figure 7.6 Particle size distribution at four locations within the Drickop wellfield

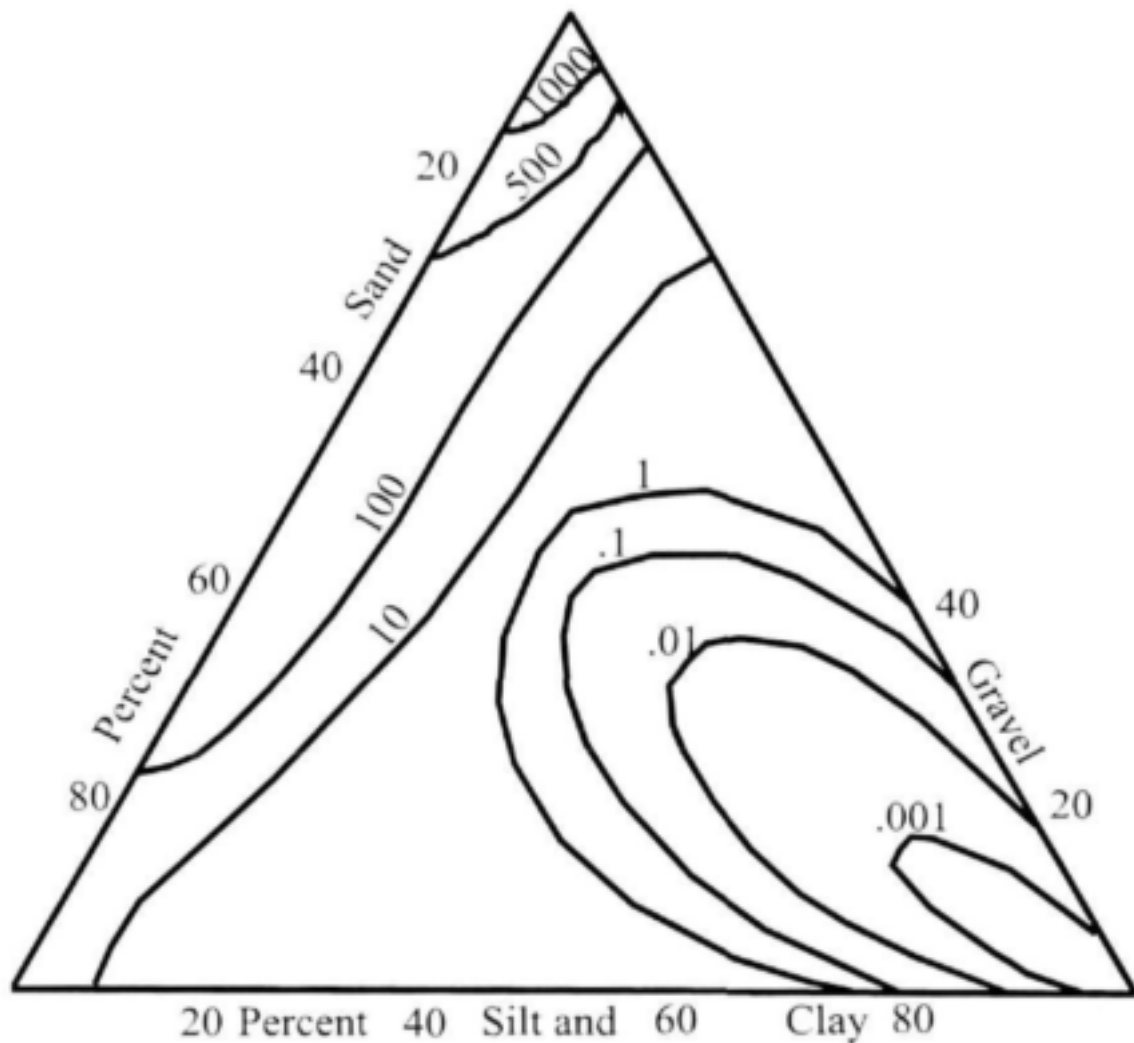


Figure 7.7 Ternary diagram showing isopleths of maximum value of hydraulic conductivity (ft/d) that unconsolidated clastic masses may achieve (after Summer and Weber, 1984)

Groundwater quality

Both Nonner (1979) and Van Dyk (1994) report comprehensively on the groundwater quality. The main concern with respect to groundwater quality in this region is its high salinity. The salinity of the water in the Driekop aquifer is substantially better than the water in the adjacent Hartebees aquifer. Production boreholes G42300, G42317 and G42399, in the Driekop aquifer, have electrical conductivity (EC) values of 194 mS/m, 140 mS/m and 142 mS/m respectively, while boreholes G42316 and G42400 which are located in the Hartebees aquifer (within 3 km of borehole G42399), have EC values of 798 mS/m and 1148 mS/m respectively (Van Dyk, 1994).

Fluoride also tends to be on the high side: production boreholes G42300, G42317 and G42399 boreholes have fluoride values of 3.4 mg/l, 2.3 mg/l and 2 mg/l respectively (Van Dyk, 1994). These values are however relatively low in comparison to the fluoride concentrations found in boreholes G42316 and G42400 (Hartebees aquifer), which are 4.5 mg/l and 9 mg/l respectively (Van Dyk, 1994).

By selectively introducing low salinity water from the Rooibergdam, the groundwater quality in the Driekop aquifer should get better.

7.2.4 Design of the artificial recharge facility

If the Kenhardt's Development Forum and the DWAF agree to pursuing the artificial recharge option, then a thorough study will need to be done in order to establish whether the Driekop aquifer is capable of receiving recharge water at relatively high rates. The process of this investigation will follow the stages set out in Chapter 5.

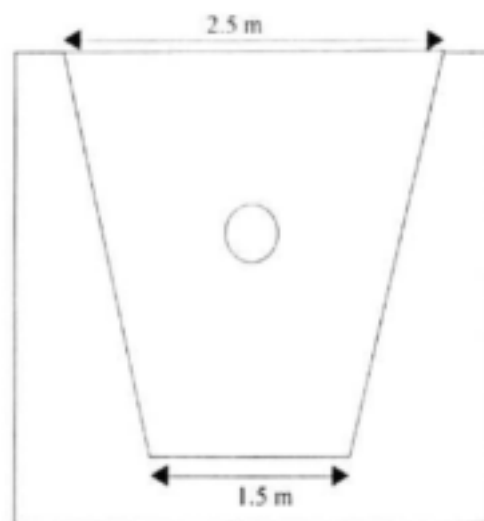
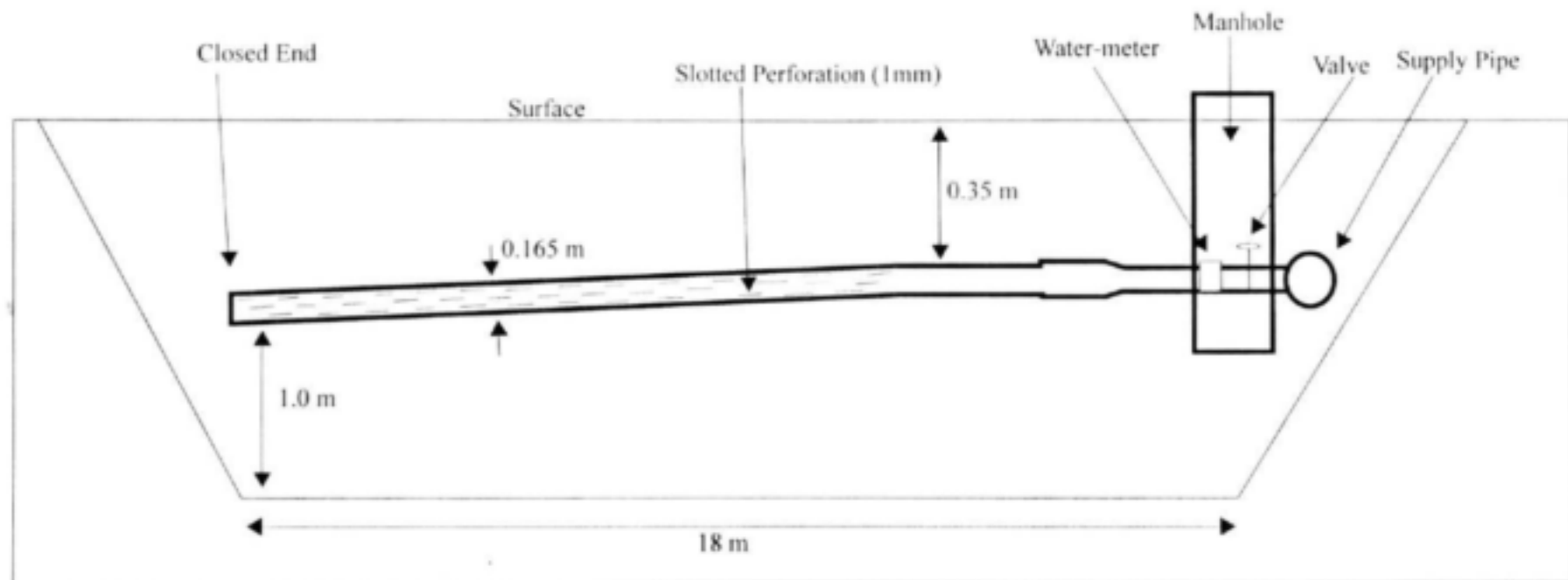
Considering that artificial recharge may be prioritised as a viable water supply option, it seems as if infiltration basins may be the appropriate artificial recharge method - both in terms of recharge efficiency and in terms of management requirements. However, more field work is needed in order to establish this. Other options include borehole injection and a trench system which was proposed by Nonner (1979). Nonner's design consists of an 18 m long, 1 m deep trench filled with gravel. The recharge water would be fed into gently sloping, slotted casing which is located in the middle of the trench (Figure 7.8). The recharge trenches would be placed about 100 m up slope of the main production boreholes.

Besides clogging considerations, the design of the recharge facility will need to be based on an assessment of the hydraulic relationships between the various soil and rock layers. These will need to be established by the following field tests:

In order to establish the relationship between the three main hydrogeologic layers, namely the unconsolidated alluvium, the weathered rock and the fractured rock, three boreholes will be drilled within a few metres of each other, and the water level response to rainfall will be monitored (Figure 7.9).

In order to determine the vertical conductivity (K_z), infiltrometer tests will be done at the surface and at about 1-2 m below the surface. If necessary, cores will be taken to determine K_z using laboratory techniques.

In order to determine to what extent recharged water will flow along the contact between the alluvium and the weathered rock (ie horizontal conductivity or K_x), the borehole which penetrates the alluvium only will be pump tested (if this zone is saturated), or a slug test or injection test will be performed on the borehole (if this zone is dry). An attempt to estimate K_x using historical base flow data at a the sub-surface weir which is located about 2 km upstream from the top of the wellfield, will also be made.



Cross Section



Natural Unsorted Sand



Gravel Bed (grainsize > 1.0mm)

Figure 7.8 Subsurface slotted casing (after Nonner, 1979)

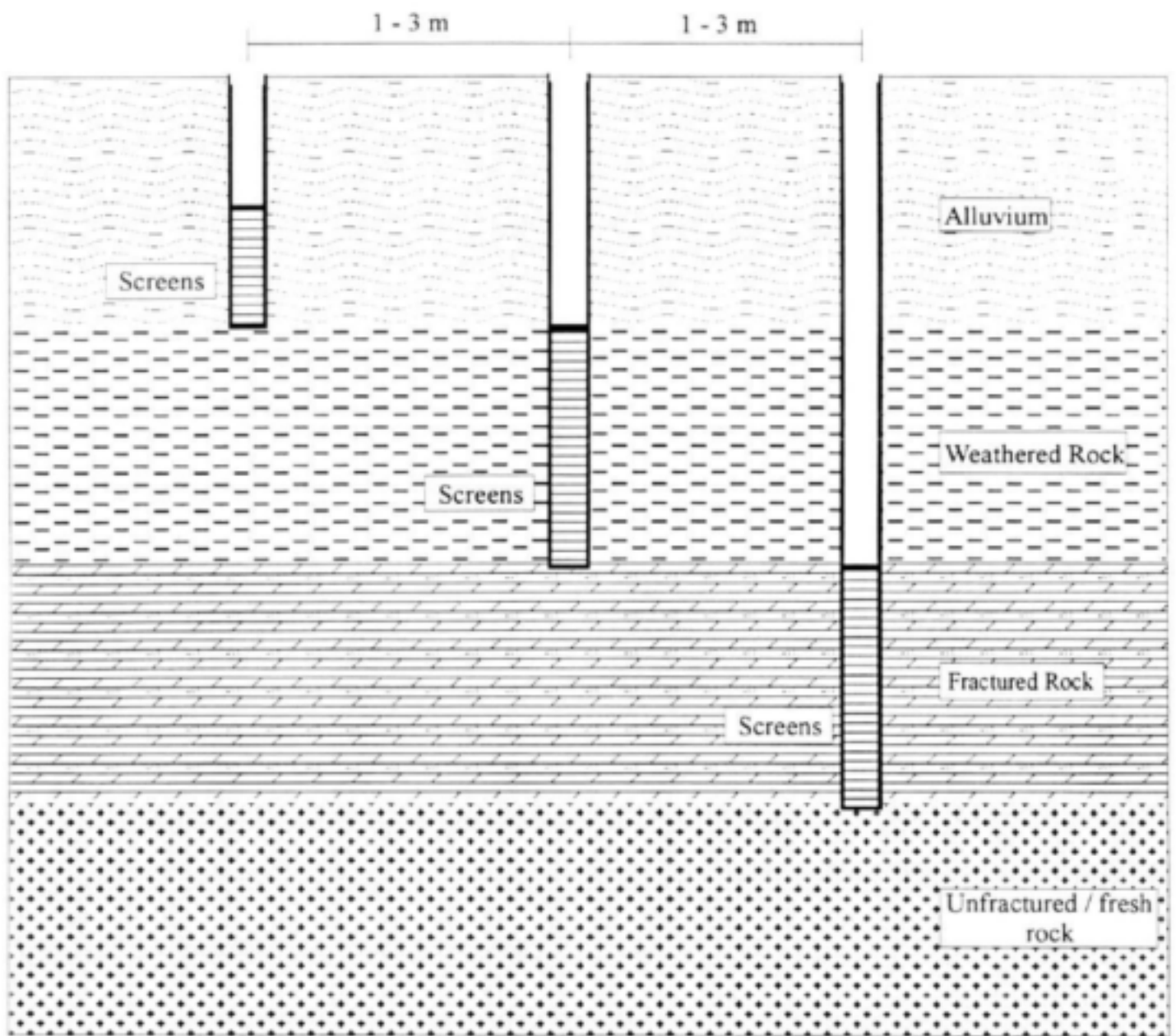


Figure 7.9 Water level monitoring boreholes to establish the relationship between the three main hydrogeologic layers.

7.3 CALVINIA BOREHOLE INJECTION PLAN

Calvinia has experienced severe water shortages in the past. The town is dependent on both surface water from the Karee Dam, which is the town's main source of domestic water, and groundwater. Recently, a new wellfield was developed in order to alleviate the water reliability problems. Although the new wellfield will improve the reliability of water for the town, with little extra cost, a reserve sub-surface storage compartment could be added to the water supply system. This would help provide a solution to the water supply problems.

In 1995 the volume of water used was 443 700 m³, which equates to a daily usage of 1 216 m³; and the projected demand for 2015 is 896 400 m³, or 2 456 m³/day (Wouter Engelbrecht Ing, 1995). The volume of water stored in the sub-surface reservoir is estimated to be between 57 000 and 74 000 m³. This water could serve as a valuable emergency water supply which could be used to meet peak demand requirements. Based on these estimates, the sub-surface reservoir could provide water for 47 to 60 days at 1995's water consumption rates, and 23 to 30 days at the projected daily requirements in 2015.

7.3.1 The sub-surface reservoir

The sub-surface reservoir, which consists of a highly porous and permeable cylindrical brecciated plug, is located about 12 km east of Calvinia, near the newly developed wellfield (Figure 7.10). At least six boreholes have been drilled into and on the outside of the plug (DWAf, 1994). The borehole information is summarised in Table 7.3.

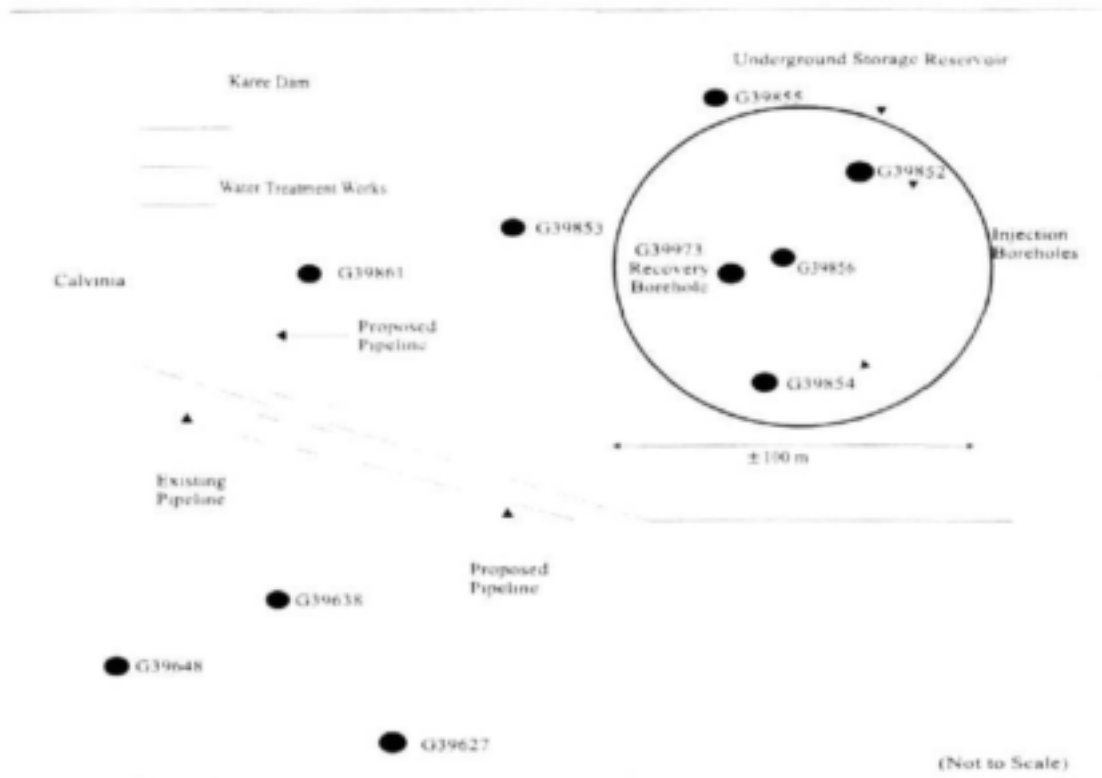


Figure 7.10 Location of Calvinia's sub-surface reservoir

Table 7.3 Breccia plug - borehole information (Woodford, 1994 & Woodford, pers comm)

Borehole Number	Depth (m)	Steel casing			Maximum blow yield (l/s)	Water level (m) & Date
		Plain (m)	Slotted (m)	Diameter ID (mm)		
Boreholes within the plug						
G39973	360	0 - 10.5	10.5 - 182 182 - 255	203 165	80.1	16.1 16/5/94
G39852	150	0 - 12.0	-	165	16.0	26.0 9/7/92
G39854	150	0 - 13.0	-	165	8.1	26.6 21/8/92
G39851A	backfilled	-	-	-	-	-
G39851	backfilled	-	-	-	-	-
G39856*	± 1000	0 - > 400	?	± 50	-	-
Boreholes outside the plug						
G39853	130	0 - 4.5	-	165		
G39855	162	0 - 12.0		165		

* This borehole was a geological exploration borehole

The summarised geological logs (obtained from DWAF) for boreholes G39973, G39852 and G39854 are presented in Figure 7.11.

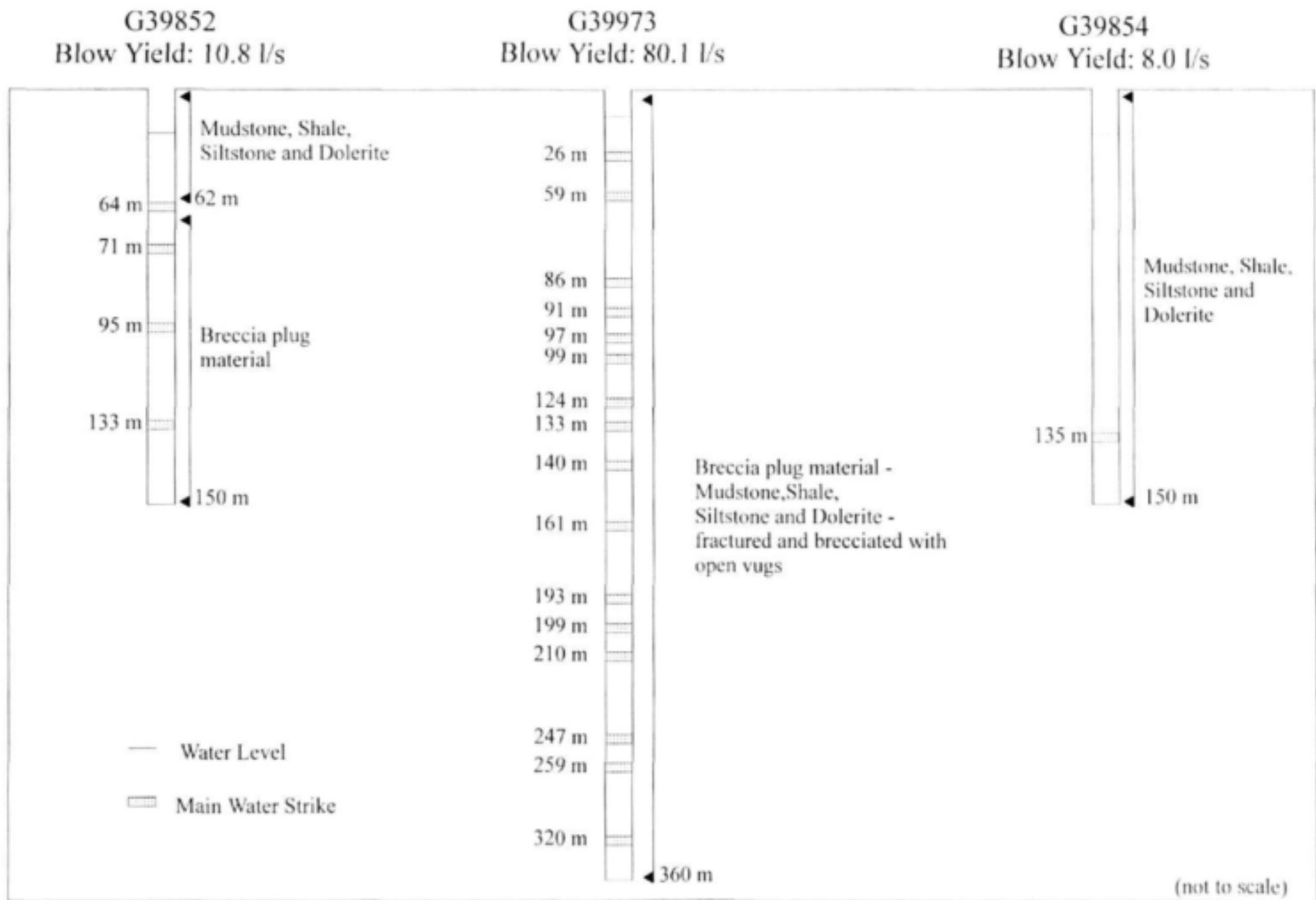


Figure 7.11 Summarised geological logs of boreholes G39973, G39852 and G39854

Volume of the reservoir

In 1994, DWAF carried out a step and constant discharge test on borehole G39973. In total, the volume of water pumped from the borehole was 22 550 m³, and the water level in the boreholes within the plug were drawn down by just over 120 mbgl. The plug is believed to be highly brecciated with open vugs, to about 240 mbgl. A summary of the results of these tests are presented in Tables 7.4 and 7.5.

Table 7.4 Step Test Results of Borehole G39973

Step	Time (min)	Yield (l/s)	Drawdown (m)	Water Level (mbgl)
				RWL: 66.18 m
1	100	1.0	0.103	66.28
2	100	5.8	0.869	67.05
3	100	9.5	1.693	67.87
4	100	12.6	2.486	68.67
5	100	14.6	3.427	69.61
6	100	18.4	4.504	70.68
7	100	21.6	5.631	71.81
8	100	26.0	6.563	72.74
9	100	27.2	7.656	73.84
10	100	29.0	8.296	74.48
Volume pumped: 993 m ³				

Note: The rest water level of 66.18 m was the water level after a substantial volume of water had been removed from the aquifer during drilling. The water level prior to abstraction is generally in the region of 16 m.

Table 7.5 Constant Discharge Test Results: Pumped Borehole G39973 - 26.1 l/s

Borehole Number	Water level at start of CD (m)	Constant Discharge Test - after 10 days pumping		Constant Discharge Test - Recovery		
		Drawdown (m)	Water level (mbgl)	Time since pump shutdown (days)	Residual drawdown (m)	Water level (mbgl)
<i>Boreholes within the plug</i>						
G39973	73.89	52.39	126.28	7.8 19.1	47.47 46.75	121.36 120.64
G39852	71.00	51.48	122.48	7.8 19.2	49.81 48.46	120.81 119.46
G39854	71.16	52.25	123.41	7.8 19.2	50.50 49.12	121.66 120.28
<i>Boreholes outside the plug</i>						
G39853	31.46	0.07	31.53	7.8 19.2	0.24 0.45	31.7 31.91
G39855	54.95	2.16	57.11	7.8 19.1	2.62 2.73	57.57 57.68
Volume pumped: 22 550 m ³						

From these tests, it is apparent that boreholes G39973, G39852 and G39854 are hydraulically well connected, and that boreholes G39853 and G39855 are located outside the plug.

In order to estimate the volume of water stored within the plug from the Constant Discharge Tests, the following should be considered:

Total drawdown: 52.39 m

Total volume pumped: 22 550 m³

Assumed m³/m of drawdown*: 438 m³/m

* Assuming a linear relationship between drawdown and volume pumped

From the geological logs, it is apparent that the main storage zone within the plug is at least between 70 m and 200 m, and possibly between 70 m and 240 m. There is also some storage between the rest water level (\pm 16 m) and 70 m. Assuming that the relationship between volume of water pumped and drawdown is linear, the volume of water stored within the plug is estimated to be between 57 000 m³ and 74 000 m³.

7.3.2 The water sources

Surplus treated water from the Karee Dam, when available, and groundwater will be used for storage within the sub-surface reservoir. During the course of 1997, two new production boreholes were added to the existing surface and groundwater supply scheme. These boreholes, G39638 and G39648, and an existing production borehole, G39861, will not be used on a daily basis. Rather, they will be brought into production when the need arises, which could vary between two to six months of the year.

Quantity of raw water

The aquifers which boreholes G39638, G39648 and G39861 penetrate can supply water on a daily basis throughout the year. The recommended production yields of these boreholes are given in Table 7.6.

Table 7.6 Recommended production yields of supply boreholes (Woodford, 1994)

Borehole No.	Pump rate (l/s)	Maximum Abstraction (m ³ /month)	Maximum Abstraction (m ³ /year)
G39638	4.0 - 8.0	2 500	30 000
G39648	3.0 - 7.0	1 500	18 000
G39861	5.0 - 8.0	3 300	39 600
Total	12.0 - 23.0	7 300	87 600

Although the maximum recommended yields range in total from 12 - 23 l/s, the pump and pipeline design for these boreholes caters for a flow of about 10 l/s (Graeme McGill, Wouter Engelbrecht Ing/Inc, pers comm). Thus the injection flow rate will not exceed about 10 l/s.

Assuming these boreholes are used for 6 months, 4 months and 2 months per year, the volume of surplus groundwater available on an annual basis for storage in the sub-surface reservoir, will range from 43 500 - 73 000 m³ (Table 7.7).

Table 7.7 Likely surplus groundwater available for storage

Usage of boreholes G39638, G39648 and 39861	Volume of non-utilised water (m ³)
6 months/year	43 500
4 months/year	58 400
2 months/year	73 000

Water quality

Water quality has been sampled on a number of occasions. Table 7.8 provides the salinity and fluoride concentration during drilling and the constant discharge test (CD), which was done in 1995.

Table 7.8 Salinity and fluoride concentration in 1995

Borehole No.	EC (mS/m)				F (mg/l)			
	During drilling		During CD (1995)		During drilling		During CD (1995)	
	Depth (m)	EC (mS/m)	Time since start of CD (min)	EC (mS/m)	Depth (m)	F (mg/l)	Time since start of CD (min)	F (mg/l)
G39638	10.5	268	4300	232	10.5	0.6	4300	0.6
G39648	15	370			15	0.9		
	30	491			30	1.2		
	65	486			65	1.2		
G39861*	?	70	120	72	?	3.6	120	2.8
G39973	26	95			26	12.8		
	360	65			360	15.4		

* Borehole G39861 was reported to have a high H₂S concentration.

The EC and fluoride values can be related to the following drinking-water quality guidelines (Kempster and Smith, 1985):

	Recommended limit	Maximum permissible limit	Crisis limit
EC (mS/m)	70	300	400
F (mg/l)	1.0	1.5	3.0

In order to model the quality of the water which would be stored in the sub-surface reservoir (that is, after mixing), various water samples were collected in December 1997 for detailed analysis (Table 7.9). The average chemistry of the Karee Dam is also presented in Table 7.9 (Woodford, 1994).

Table 7.9 Borehole and Karee Dam water quality

SAMPLE ID:	G39638	G39648	G39854	G39852	Karee Dam
SAMPLE DATE:	97/12/11	97/12/11	97/12/09	97/12/10	
Potassium as K mg/L	2.0	3.6	0.6	1.2	1
Sodium as Na mg/L	612	577	160	205	12
Calcium as Ca mg/L	278	145	1.5	10.1	18
Magnesium as Mg mg/L	218	121	0.1	3.2	10
Ammonia as N mg/L	<0.1	0.7	1.4	0.5	
Sulphate as SO ₄ mg/L	413	106	45	166	9
Chloride as Cl mg/L	1515	1272	93	97	13
Alkalinity as CaCO ₃ mg/L	354	261	157	150	
Nitrate plus nitrite as N mg/L	0.26	<0.1	<0.1	<0.1	0
Ortho phosphate as P mg/L	<0.1	0.2	<0.1	<0.1	
Iron as Fe mg/L	0.22	0.13	0.22	0.07	
Manganese as Mn mg/L	<0.05	0.36	<0.05	<0.05	
Zinc as Zn mg/L	<0.05	<0.05	<0.05	<0.05	
Dissolved Organic Carbon mg/L	1.94	<1	<1	<1	
Conductivity mS/m @25deg C	560	443	78	105	21
Conductivity mS/m (field)	494	390	72	93	
Field temp. deg. C	18.6	22	24.5	23.0	
pH (Lab)	7.7	8.1	10.0	8.9	
pH (field)	7.5	7.5	10.0	8.5	
Saturation pH (pHs) (20deg C)	6.7	7.1	9.2	8.4	
Total Dissolved Solids (Calc) mg/L	3584	2835	499	672	135
Hardness as CaCO ₃ mg/L	1592	860	4	38	
Silica as Si mg/L	11.09	7.58	17.21	8.83	
Fluoride as F mg/L	0.4	1.4	12.0	8.9	0
Turbidity NTU	6.8	0.30	4.0	0.55	

Note: Boreholes G39852, G39854, G39638 and G39648 were all reported to have a H₂S smell. Boreholes G39638 and G39648 were reported to contain turbid water for the first 10 - 15 minutes of pumping (this water will need to be diverted so that it does not enter the injection boreholes).

Chemical equilibrium modelling assessment of precipitation potential

Chemical equilibrium modelling was carried out, using the modelling package MINTEQA2 (Allison *et al.*, 1991), for determining whether mixing of aquifer and recharge water may lead to chemical precipitation, which, in turn, may cause blockages in the aquifer. The approach was to assess the precipitation potential for each water individually, as well as for mixtures of water from the source boreholes and surface water (from the Karee Dam) with the injection well water. Analyses were available for the source boreholes G39638 and G39648 but not for the injection well (G39873) itself. As a result, data for borehole G39854 were used to represent the water quality in the injection borehole. Other assumptions were that thermodynamic equilibrium was reached, that oxidation reduction equilibria could be disregarded, and that the traces of dissolved organic carbon could be ignored in the equilibrium simulations. Finally, it was assumed that the partial pressure of carbon dioxide in the aquifer exceeds that of the normal atmosphere and was taken as $10^{-1.5}$ atmospheres based on literature information (Schoeller, 1959; Stumm & Morgan, 1996). Good agreement between field pH measurements and calculated values using the higher carbon dioxide partial pressure served to support the assumption of the elevated values for water in the aquifer.

The equilibrium modelling showed that in all mixing scenarios the solutions would be in equilibrium with the potential solid phases. Only ferric solids would show a degree of super saturation and thus have a possible precipitation potential. It depends, however, on the oxidation state of the iron in solution. If it is assumed that iron occurs exclusively as Fe^{+2} , i.e. in the reduced form, no precipitation potential exists. The mooted presence of traces of hydrogen sulphide will support the existence of reducing conditions. The possible precipitation of the traces of iron as ferrous sulphide is not considered to present any problem.

7.3.3 Design of the sub-surface reservoir scheme

The plan involves pumping raw groundwater and treated water from the Karee Dam (when available), to borehole G39854 and borehole G39852. In order to test whether these boreholes could receive an injection flow of up to about 10 l/s, they were test pumped in December 1997. The test pump curves are shown in Figures 7.12 and 7.13. These tests indicate that the proposed injection boreholes should be able to receive the designed maximum injection flow of 10 l/s.

Unfortunately, boreholes G39973, G39852 and G39854 are currently blocked above their recorded depths. Once this problem has been addressed, and the boreholes have been restored to their original depths, the plan is to place the inlet pipes in both boreholes at 140 mbgl. When needed, the water will be pumped from borehole G39973, and the pump intake will be at 250 mbgl, which will also be the maximum depth to which the water level will be drawn down (ie maximum pumping head is 250 m). The rate at which G39973 can deliver water is up to about 80 l/s, but the production yield will be restricted by the pipeline design, and it is only expected to be in the region of 20 l/s.

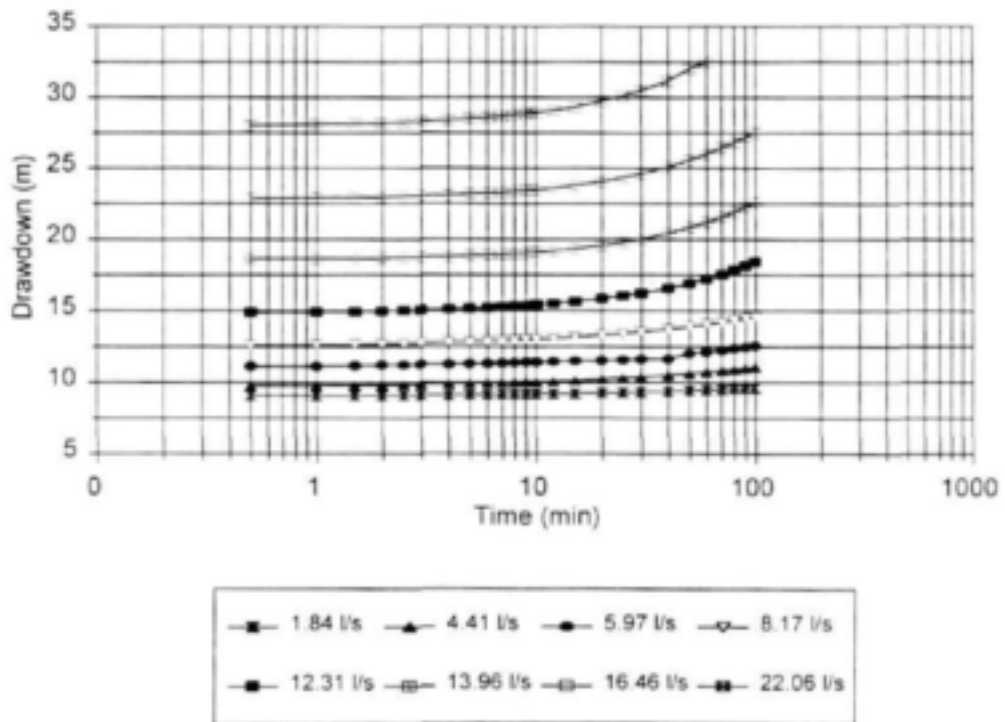


Figure 7.12 Step test on borehole G39852

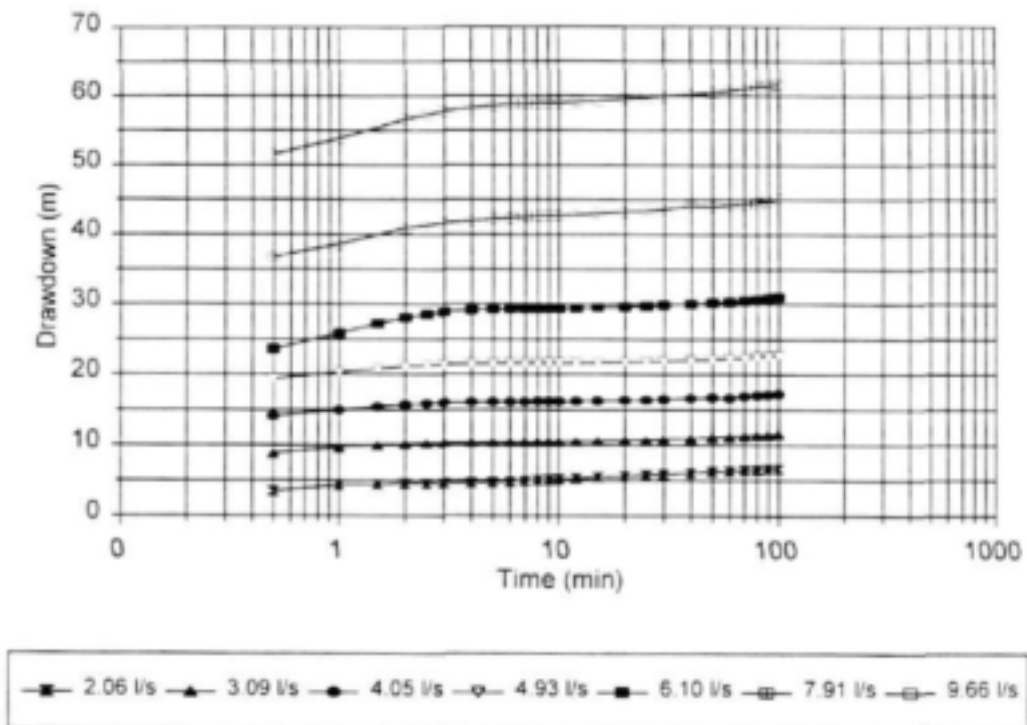


Figure 7.13 Step test on borehole G39854

7.3.4 Proposed implementation plan

The stages needed to develop the sub-surface reservoir are:

- i. Restore boreholes G39973, G39854 and G39852 to their original depths.
- ii. Complete the pipeline to these boreholes.
- iii. Install the injection pipes and pump in borehole G39973.
- iv. Install the monitoring system, which will include flow meters and water level meters on boreholes within and outside the plug. A meter outside the plug is necessary in order to establish the maximum amount of water that can be stored in the sub-surface reservoir before water is lost to the near surface aquifer.
- v. Using borehole G39973, pump a substantial volume of water out the sub-surface reservoir in order to create space for the first injection run.

The current status of this project is described in Appendix 1.

7.4 WILLISTON: GROUNDWATER TRANSFER SCHEME

Williston depends heavily on borehole G33076 for their domestic water. The water level in this borehole, however, has declined steadily over the years since it has been used as a production borehole. The aim of this proposed injection scheme is to transfer groundwater from an adjacent aquifer to the aquifer which supports this borehole.

The Division of Geohydrology, DWAF, has monitored since 1983, the aquifer which supports Williston's main production borehole, G33076. From the borehole water level data, it is apparent that boreholes G33076 and G39975 are hydraulically connected - the water levels match one another, even though they are located about 4 km apart (Figure 7.14). This hydraulic interconnectiveness is a result of a horizontal fracture pattern which resulted from doleritic sill intrusions into Karoo sedimentary rocks. Both boreholes intersect the main horizontal fracture at ± 70 mbgl. Thus, if water was injected into borehole G39975 it could be abstracted from borehole G33076.

The water would come from borehole G 39976, which is located within 50 m of borehole G39975. Borehole G 39976 penetrates a different groundwater compartment to borehole G39975. The water levels in these boreholes differ by about 20 m, indicating that they are separated by a seemingly impermeable boundary (Figure 7.15). Borehole G39976 would, however, need to be test pumped in order to clarify the hydraulic relationship between this borehole and borehole G 39975. The rate at which water could be injected to borehole G 39975 is estimated to be in the region of 6 l/s or 500 m³/day.

One question that arises is: Would it be possible to siphon water from borehole G39976 to borehole G39975, considering their different water levels? If this is possible (with the necessary bleeding system), substantial savings on pumping costs could be made.

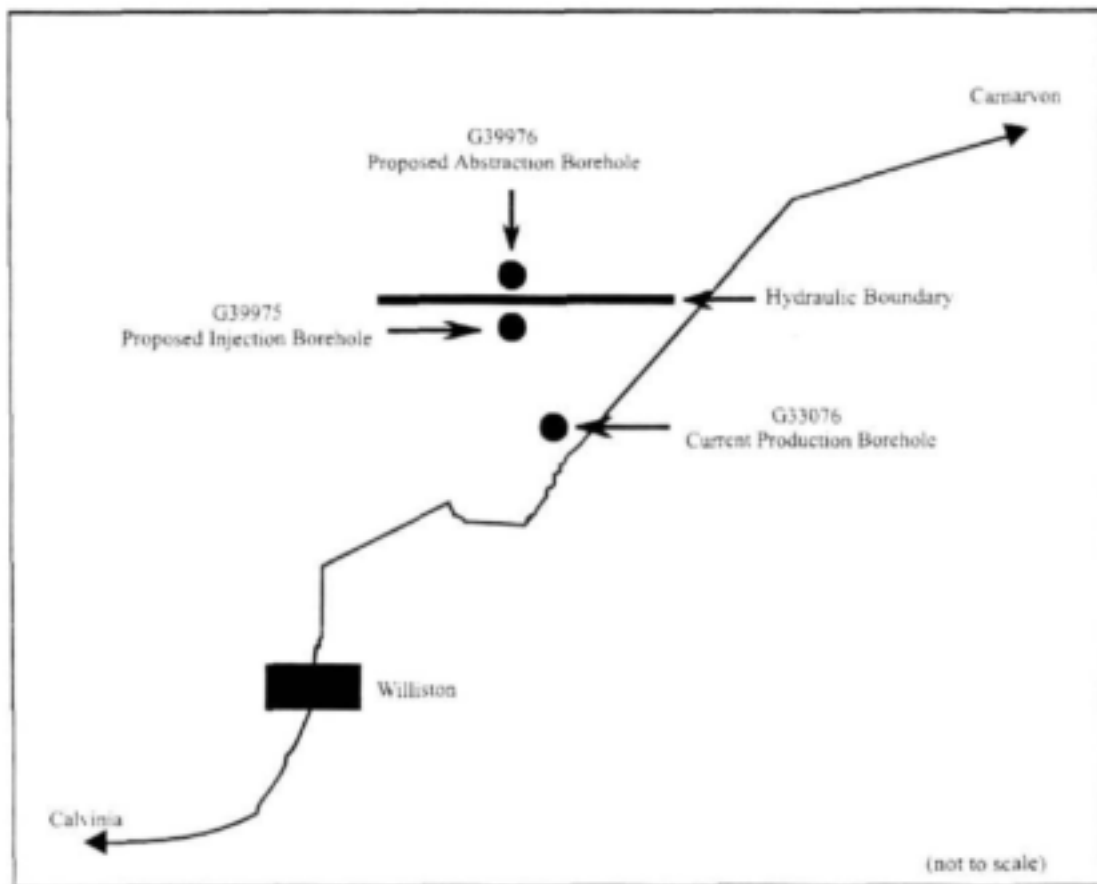


Figure 7.14 Location of boreholes G33076, G39975 and G39976

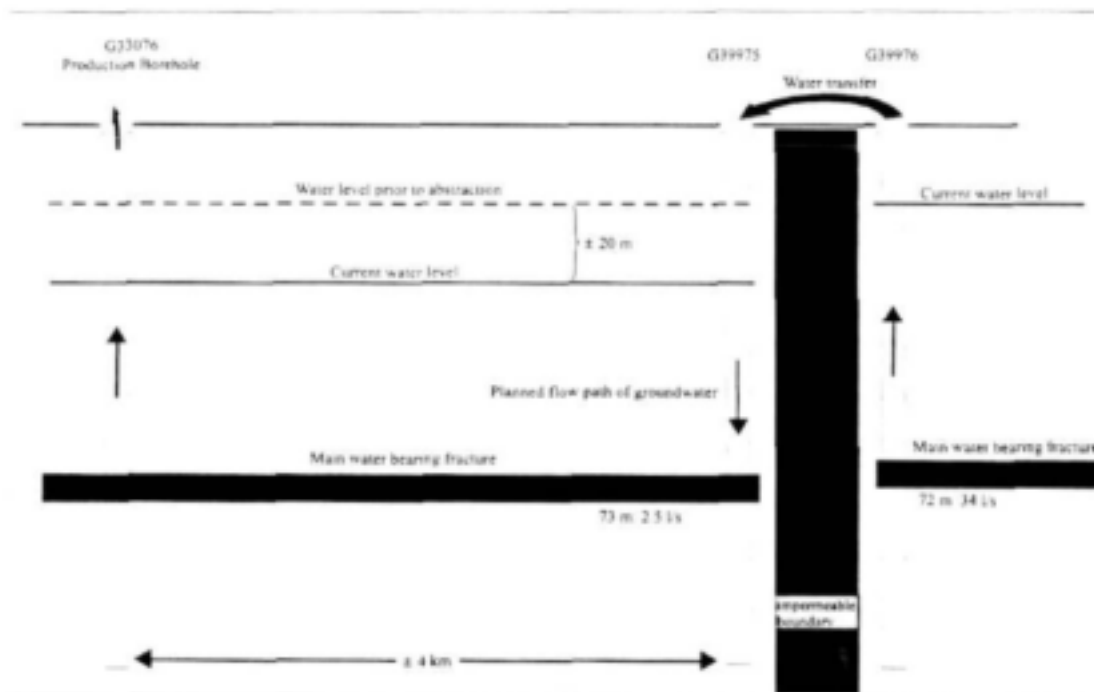


Figure 7.15 Williston's groundwater transfer plan

Chapter 8

CONCLUSIONS

This project set out to assess the feasibility of using artificial recharge as a means of enhancing groundwater resources in South Africa. As a part of this assessment, both hydrological and non-hydrological factors were studied, and information on existing and planned artificial recharge schemes were collated. The report presents a working document which can be used by hydrogeologists and water resource planners in assessing the feasibility of a particular site for artificial recharge.

In addition to this, the report describes South African hydrogeologic environments which are suitable for artificial recharge; and it presents *four possible sites where artificial recharge could be implemented*. These sites include the aquifers which support the towns of Kenhardt, Calvinia and Williston, and the city of Windhoek in Namibia.

This chapter concludes this report by highlighting the success factors which affect the potential for artificial recharge and summarising South African hydrogeologic environments suitable for artificial recharge.

8.1 SUCCESS FACTORS

It is concluded that the main factors which determine whether artificial recharge is likely to be successful are:

- The water source - availability and type;
- The hydraulic characteristics of the aquifer;
- The quality of the recharge water;
- Clogging of the recharge basins, trenches or boreholes;
- Groundwater recovery;
- Economic factors;
- Management requirements.

The main conclusions drawn for each of these success factors are given below.

8.1.1 Recharge Water Sources: Quantity, Quality and Reliability

Recharge water can be obtained mostly from surplus surface water which is currently lost by means of evaporation from rivers and dams or which flows into the sea. Water for aquifer recharge purposes has to have a consistent high quality and a fairly predictable quantity over

time. Sources considered are municipal wastewaters, storm runoff, rainfall harvesting, river flows, and water releases from dams. *Municipal wastewater* has a predictable quantity and quality, but it may not necessarily be suitable for artificial recharge purposes for unrestricted reuse. They invariably require significant chemical treatment before being considered of sufficiently high quality for aquifer recharge. *Urban storm runoff* is usually highly variable in quality, but with the exception of industrial runoff, is on average of higher chemical quality than municipal waste water and agricultural runoff.

Due to the highly variable flow rate, *storm runoff* is usually collected in an impoundment or basin from which controlled release of water into recharge basins takes place after settling of the bulk of the suspended solids. *Rivers* have a more consistent quantity of flow than storm runoff in their catchment areas, due partly to a base-flow contribution from groundwater/ interflow, and a wide range of rainfall-response times.

In many parts of South Africa, and in particular the dryer western areas where evaporation is high, it may be beneficial to store surface water underground. In order to assess this, both the evaporation losses and the economics associated with treating, transferring and recovering the artificially recharged water need to be determined.


8.1.2 Aquifer Hydraulics

There are two physical characteristics which determine whether an aquifer is suitable for accepting artificially recharged water. They are the aquifer's permeability and storage capacity. A third important factor is the aquifer's hydraulic gradient, which relates mostly to the recovery of the recharged water. Key questions are:

- Is the aquifer sufficiently permeable to allow the recharge water to enter it?
- Has the aquifer sufficient storage to accept the water?
- Will the water be recoverable?

Aquifers which are highly permeable and which have high storage capacities are more suitable for receiving additional recharge water than those which have low permeabilities and low storage capacities (Table 8.1).

Table 8.1 Suitability of an aquifer to receive artificially recharged water

Storativity (S)	Hydraulic conductivity (K)	Aquifer Suitability
High S	Moderate - High K	 Most Suitable
High S	Low K	
Low S	High K	
Low S	Low K	

In the case of basin infiltration, the success of recharge schemes is largely dependent upon the ability of the unsaturated zone to transmit water. In addition, the soil characteristics should be such that while the optimal infiltration rate is obtained, the contact time with the soil and aquifer material should also be sufficient for ensuring the desired quality improvements by sorption, degradation and other processes.

In relation to aquifer storage, the main concern with South African aquifers, and in particular, secondary aquifers, is whether there is sufficient storage space to accept the artificially recharged water. On the one hand, South African aquifers are typically full during or shortly after the natural recharge periods, that is, when there is surplus surface water; and on the other hand, when there is little surplus water, during the dry periods, they usually have available storage. Thus the suitability of the aquifer to receive additional water and the availability of water will often dictate whether a site is suitable for artificial recharge.

8.1.3 Quality of Recharge Water

Various factors influence the quality of the water when artificially recharging an aquifer. Infiltrating water moving through the unsaturated zone below a spreading basin, may undergo substantial quality changes before reaching the aquifer. The design of the system should be such that these changes will improve the quality of the water before it reaches the saturated zone. However, the initial quality of the recharge water often remains the main characteristic determining the final water quality. In the case of direct injection into the aquifer, quality changes could be limited to phenomena such as calcium dissolution from a carbonate aquifer.

The quality of the recharge water is a key variable in the decision on the type of artificial recharge system to be employed. High quality, low turbidity water can be utilised successfully in any kind of recharge system. However, when the water quality needs improvement, for example, to reduce the nutrient or organic compound, a surface infiltration system involving soil aquifer treatment (SAT) may be appropriate. Alternatively, pretreatment options may be exercised to reduce turbidity and improve quality for direct recharge. Also, aquifer storage and recovery systems (where the same borehole is used for recharge and recovery), sometimes allow

for a controlled degeneration in aquifer water quality where the recovered water is to be used for restricted purposes. In such cases the turbidity must still be low, but the chemical and/or microbiological quality may be impaired. In general, the extent of post treatment of water recovered from an artificial recharge operation, will mainly depend on the intended use.

8.1.4 Clogging Potential

Clogging of recharge basins, trenches or boreholes results in a reduction in the efficiency of the recharge process. Correctly dealing with the phenomenon of clogging, plays a decisive role in determining the success or failure of a scheme. Clogging of the system can be due to mechanical, physical, chemical and biological processes, as well as a combination of these. It can take place at the infiltration surface, in the unsaturated zone, or in the aquifer itself. In the case of injection, it could block the fractures leading away from the borehole. A thorough understanding of the processes involved and the consequent reversibility or irreversibility of the situation is needed in order to be able to manage the clogging phenomenon.

The processes that are primarily responsible for clogging are: deposition of suspended solids from the recharge water; air entrapment and gas binding; biological growth of bacteria on or within the infiltration media and the surrounding formation; and chemical reactions between recharge water, groundwater and the aquifer material.

8.1.5 Groundwater Recovery

The issue of recovery efficiency is usually only of concern in borehole injection schemes when the quality of the recharge water and the native groundwater are of vastly different. In the case of aquifer storage and recovery systems, recovery efficiencies are most often defined in terms of water quality, for example electrical conductivity. The limit for the abstracted water is generally set at a higher salinity than that of the injected water. This reflects the typical current practice of injecting fresh water into saline aquifers. What is being abstracted is therefore not only the injected water but a mixture of injected and native water. This is continued until the proportion of native saline water becomes unacceptable.

Typical recovery efficiencies in aquifer storage and recovery systems are found to be up to 70 percent. However, most schemes can be developed to 100 percent. Exceptions are very transmissive, highly saline aquifers which reached 70 to 80 percent.

Where the artificial recharge scheme affects only a part of the aquifer which is centrally located and hydraulically up-gradient of the production well field, losses of recharged water from the aquifer are, in general, regarded as being insignificant.

Recovery efficiency should be compared to the efficiency of surface storage facilities, such as dams, which lose vast volumes of water to evaporation and leakage.

8.1.6 Economic Factors

Unused aquifer storage capacity can be developed at a significantly lower cost than surface storage facilities, and without the adverse environmental consequences frequently associated with surface storage. Often the overall costs of artificial recharge operations are less than half the capital cost of conventional water supply alternatives, especially those involving development of new reservoirs, treatment facilities or extensive pipelines (National Research Council, 1994).

It is important when undertaking a cost benefit analysis on various water supply options to ensure that all the costs are considered. These should not only include the costs associated with developing and operating the schemes, but also the savings from minimising water losses. Because evaporation losses are so high in South Africa, in many cases it will be cost effective to store water below the ground.

8.1.7 Management of the scheme

Artificial recharge schemes commonly involve surface or waste water capture, treatment, pumping, distribution and water quality monitoring. In order for these processes to be efficient, careful planning and management is needed. One of the key management functions is to minimise clogging. In the case of *surface infiltration systems*, a wetting and drying cycle with periodic cleaning of the bottom is used to reduce clogging by accumulated suspended material. It is also essential to maintain the quality of the recharge water.

In *subsurface infiltration systems*, it is essential that poor quality water be kept out of the pits/trenches. The key management tasks include monitoring the quality of the recharge water, and maintaining good quality recharge water.

The potential for clogging is especially high in *borehole injection systems*. For this reason it is essential that the high quality of the recharge water be maintained. The key management issues are: maintaining the water treatment works which supplies recharge water; monitoring the recharge water quality; monitoring injection rates (injection efficiency); and restoring injection efficiency by backflushing, and other methods.

8.2 SOUTH AFRICAN GEOHYDROLOGICAL ENVIRONMENTS SUITABLE FOR ARTIFICIAL RECHARGE

South African aquifers vary considerably from those with high a permeability and storativity like the primary aquifers at Atlantis and the Cape Flats in the Western Cape Province and some of the dolomitic aquifers in the Northwest Province, to those with low permeability and storativity like most of the hard rock aquifers which are found throughout the country. While aquifers with high permeabilities and storativities are most suitable for receiving recharge water, aquifers with limited permeability and storativity can also be artificially recharged.

In many parts of the country where only hard rock aquifers exist, artificial recharge may be an appropriate method to enhance limited natural groundwater resources. In such areas, average borehole yields may only be in the region of 1 l/s. If these aquifers were artificially recharged at 1 l/s from a number of recharge points, it could make a considerable difference to the exploitable groundwater resource. Table 8.2 presents geohydrological environments which may be suitable for artificial recharge and it lists the main factors which will determine the areas' suitability.

Table 8.2 Geohydrological environments suitable for artificial recharge

Possible South African environments	Key prerequisites for application
Induced recharge schemes (eg. river bank filtration)	
Perennial rivers flowing over sandy flood plains. Fresh water lakes with surrounding sandy soils.	River water with little suspended matter; Permeable river beds/banks; Permeable riverine sands or gravels extending away from the river; Regular & sufficient flows.
Infiltration schemes (eg. infiltration basins, land flooding and subsurface trenches)	
Thick sand deposits in ephemeral river beds and coastal dunes. Weathered and fractured rock aquifers overlain by sandy soils or gravels.	The quantity, quality and reliability of the raw water must be acceptable; The soils must be sufficiently permeable with no continuous impermeable layers; The aquifer must have sufficient storage available, and have sufficient permeability to accept the recharge water.
Borehole injection	
Permeable secondary aquifers. The aquifers can be confined and deep seated.	The quantity and reliability of the raw water must be acceptable; The turbidity of the injected water must be very low; The chemistry of the injected water should be such that it does not lead to excessive mineral precipitation within the aquifer; The aquifer must be sufficiently transmissive and have available storage in order to receive the water.
Artificial aquifers (sand storage dams)	
Rugged, arid areas with high runoff and ephemeral sandy river beds.	Arid area with high runoff resulting in ephemeral river flow; The predominant parent rock in the area should weather to produce a significant portion of coarse sandy sediment; The dam should be underlain either by a low permeability bedrock to prevent seepage losses, or bedrock which hosts a secondary aquifer to be recharged by the sand dam.

8.3 OVERALL CONCLUSION

Artificial groundwater recharge is an internationally recognised method for managing water resources. Main uses are for conserving water for future use, for improving water quality and for averting saline water intrusion into over-exploited aquifers. At present there are only a few fully operational artificial recharge schemes in Southern Africa, the largest one being in the town of Atlantis, but the potential exists to make wide use of this technology.

There have been many attempts to artificially recharge aquifers in Southern Africa. In most cases, however, the schemes have been poorly designed, and as a result they have either not had the desired effect, or it is not possible to establish the effect that they have had on recharge. The most common problem seems to be that the recharge water used has not been of sufficient quality, and as a result, it has led to clogging of boreholes and infiltration basins.

Sufficient motivation exists to support the continuation of pilot scale artificial recharge studies in secondary aquifers. Sites which have been identified, and which seem suitable with respect to the success factors mentioned above, include Kenhardt, Calvinia, Williston and Windhoek. One or more of these should be developed as demonstration projects.

Chapter 9

RECOMMENDATIONS

The results of this study prompted the following recommendations:

- i. Most existing schemes throughout the world are in highly permeable, porous aquifers. Since most of South Africa's aquifers are secondary, fractured rock aquifers, it will be necessary to test artificial recharge in these hydrogeologic conditions. It is therefore recommended to implement pilot artificial recharge schemes in secondary aquifers, and in particular fractured rock aquifers. Pilot studies should, therefore, include borehole injection schemes. The successful pilot sites should be used as demonstration schemes in order to make water resource planners and the public aware that artificial recharge can form a viable part of water resource management.
- ii. Borehole clogging using surface water should be studied. For borehole injection schemes the water must be of high quality, particularly with respect to suspended solids. The degree of surface water treatment required prior to injection needs to be assessed during pilot studies.
- iii. Chemical equilibrium modelling should be done to predict the outcomes of chemical reactions between recharged water, native groundwater and the host rock in order to assess the possibility of chemical clogging of the borehole and the aquifer.
- iv. Factors which affect recovery efficiency should be studied during the implementation of pilot artificial recharge schemes in secondary aquifers. In fractured rock aquifers, one of the issues when considering artificial recharge is: where will the recharge water go, and will it be recoverable? This concern needs to be addressed in pilot studies in fractured rock aquifers.
- v. When using water of impaired quality for recharge of fractured aquifers, research should be carried out to establish the fate of micro-organisms.

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Glossary of Technical Terms

aerobic	a process taking place in the presence of oxygen.
anaerobic	a process taking place in the absence of oxygen.
advection (time)	refers to the transport of a solute at a velocity equivalent to that of the groundwater movement.
alkalinity	the acid neutralising/consuming capacity of a water sample. Carbonate or phenolphthalein alk. represents acid consumption by titration to pH 8.3 and bicarbonate or methyl orange alk. given by titration to pH 4.5. Total is the combined C + B but usually C is minimal because of the pH range of most natural groundwaters.
alluvial	detrital material which is transported by a river and deposited-usually temporarily - at points along the flood plane of a river. Commonly composed of sands and gravels.
anisotropy	in a formation, the nature of the material and its orientation causes a preferential direction for the movement of the water.
aquifer	strata or a group of interconnected strata comprising of saturated earth material capable of conducting groundwater and of yielding usable quantities of groundwater to borehole(s) and / or springs.
aquifer system	a heterogeneous body of intercalated permeable and less permeable material that acts as a water-yielding hydraulic unit of regional extent.
aquitard	a geological formation or group of formations which does not readily transmit water and hence restricts groundwater movement.
artesian aquifer	an aquifer whose rest water level is higher than the ground level.
base flow	sustained low flow of a stream, usually groundwater inflow to a stream channel.
borehole	generic term used for any drilled or hand-dug hole used to abstract or monitor groundwater, irrespective of diameter or construction.
brackish	water that contains between 1 000 and 10 000 mg/L of dissolved solids.

buffer zone	zone of mixing of injected and native water.
calcrete	superficial gravels cemented by calc tufa.
clay	a detrital sedimentary rock, with particle size of less than 1/256 mm. Material which is plastic when wet and has no well-developed parting along the bedding plane, although it may display banding.
coefficient of storage	<i>see storage coefficient.</i>
confined aquifer	an aquifer which is overlain by a confining layer of significantly lower hydraulic conductivity, the groundwater is confined under pressure greater than atmospheric pressure such that if the aquifer is penetrated the water level may rise above the top of the aquifer, also known as an artesian aquifer.
confining layer	a layer of low permeability material adjacent to an aquifer which restricts the vertical movement of water.
consolidated	a compact, harder material converted from a loose or soft material.
contamination	the introduction into the environment of any substance by the action of man.
degradable pollutants	pollutants which readily breakdown.
discharge area	an area in which subsurface water, including water in the vadose zone and saturated zones, is discharged to land surface, to surface water or the atmosphere.
dissolved solids	minerals and organic matter dissolved in water.
dolerite	a medium grained basic igneous rock, mineralogically and chemically the same as gabbro and basalt.
drawdown	the difference between the observed water level during pumping and the non-pumping water level.
dual permeability	the occurrence of two scales or types of permeability in an aquifer. For example primary interstitial permeability and secondary fracture permeability.
dyke	a sheet-like body of igneous rock which is discordant (cuts across the bedding or structural planes of the host rock).

evapotranspiration	outflow from a hydrologic system as a combination of evaporation from open bodies of water and soil surfaces, and transpiration from the soil by plants.
fault	a fracture in rock along which there has been an observable amount of displacement.
fissures	a general term to include natural fractures, cracks and openings in consolidated rock caused by bedding planes, joints, faults, etc.
fitness-for-use	water quality is such that it meets the requirements for a particular use; five major groups of water users recognised as domestic, agricultural, industrial, recreational or environmental users.
formation	general term used to describe a sequence of rock layers.
fracture	cracks or breaks in the rock which can enhance water movement.
fracture flow	water movement that occurs predominantly in fractures of fissures.
freshwater	water that contains less than 1 000 mg/L dissolved solids.
gravel	is applied to grains larger than coarse sand (>2 mm) and finer than pebbles (<4 mm).
groundwater	all subsurface water occupying voids within a geological stratum.
groundwater flow	the movement of water through openings and pore space in rocks.
hardness	the soap consuming capacity of the water, shown by the conc. of Ca and Mg in solution plus Sr, Ba, Fe 2 ⁺ but these are usually insignificant. Temporary hardness is the dissolved conc. of Ca and Mg which is equivalent to the alkalinity (ie would precipitate out to form carbonates) and permanent hardness is the dissolved conc. of Ca and Mg which is in excess to the alkalinity (ie would remain in soln. even after CaCO ₃ etc precipitated out).
hard-rock	igneous, metamorphic and sedimentary rocks which lack adequate primary interstices to function as a primary aquifer.
heavy metals	those elements with atomic numbers greater than 36 in Group III through V of the Periodic Table.
hydraulic conductivity	measure of the ease with which a fluid will pass through earth material, defined as the rate of flow through a cross-section of one square metre under a unit hydraulic gradient (in m/d).

hydraulic gradient	the rate of change of hydraulic head per unit distance in a particular direction.
hydraulic head	the height of a column of water above a reference plane.
hydrochemistry	the study of the chemistry of water.
hydrodynamic dispersion	the changes in concentration of a solute flowing in groundwater by processes of molecular diffusion and mechanical dispersion.
infiltration	movement of water into soil or a porous rock.
in situ	(Latin, "in place") used to distinguish between material e.g. rocks, minerals, fossils, etc., found in their original position of formation, deposition or growth, as opposed to loose, disconnected or derived material.
integrated management	a management approach which serves to co-ordinate management of the environment as a whole, as opposed to individual parts.
intergranular flow	flow that occurs between individual grains of rock.
intrusion	a body of igneous rock which has forced itself into pre-existing rocks, either along some definite structural feature, e.g. bedding planes, joints, cleavages, etc., or by deformation and cross-cutting of the invaded rocks.
isotropy	the condition of having properties that are uniform in all directions.
joint	a fracture in a rock between the sides of which there is no relative movement.
Karoo	a stratigraphic term used for a series of sediments and lava flows of considerable extent in Southern and Central Africa.
leachate	any liquid, including any suspended components in the liquid, that has percolated through or drained from human-emplaced materials.
lithology	the physical character of rocks.
mass transfer	the transport of solutes dissolved in groundwater

mechanical dispersion	mixing of waters of different qualities due to the movement of the fluids. This includes longitudinal dispersion (in the direction of advective flow) and transverse dispersion (perpendicular to the flow direction) and results from variable flow velocity and flow paths.
molecular diffusion	the spreading of molecules in a direction tending to equalize concentrations in all parts of the system.
mudstone	rocks which do not form a plastic mass when wet, but may disintegrate when immersed in water.
native water	groundwater that occurs naturally in the aquifer (as opposed to injected water).
non-degradable pollutants	pollutants that do not readily breakdown.
outcrop	the occurrence of rock at the ground surface; when a rock is visible, for instance, cliffs and quarries, the rock is said to crop out.
perched ground water	unconfined groundwater separated from an underlying main body of groundwater by an unsaturated zone, generally perched on an impermeable layer.
percolation	slow laminar movement of water through openings in a porous media.
piezometer	a tube or pipe in which the elevation of a water level can be determined.
piezometric level	the elevation to which the water level rises in a borehole which penetrates confined or semi-confined conditions.
piezometric surface	an imaginary surface representing the hydraulic head throughout all or part of a confined or semi-confined aquifer, analogous to the water table of an unconfined aquifer.
porosity	ratio of the volume of void space to the total volume of the rock.
primary aquifer	an aquifer in which water moves through the original interstices of the geological formation.
primary interstices	interstices that were made contemporaneously with the rock formation.

pumptest	a method of parameter measurement that is specifically suited to the determination of transmissivity and storativity in confined and unconfined aquifers.
recharge	process of the addition of water to the groundwater system by natural or artificial processes.
recharge area	an area over which recharge occurs.
recovery efficiency	the percentage of injected water volume stored that is subsequently recovered while meeting a target water quality criterion.
renewable water supply	rate of supply of water available in area on an essentially permanent basis.
rest water level	the static water level in a borehole when not pumped.
rock	any consolidated or unconsolidated earth material, specifically excluding soil.
safe yield	amount of water that can be withdrawn from an aquifer without producing an undesired effect.
saline intrusion	replacement of freshwater by saline water in an aquifer, usually as a result of groundwater abstraction.
saline water	water that is generally considered unsuitable for human consumption or for irrigation because of its high content of dissolved solids (> 10 000 mg/L).
salinity	the dissolved solid content.
sand	a detrital deposit in which the particle size range from 1/16 mm to 2 mm.
sandstone	a detrital rock composed of grains from 1/16 to 2 mm in diameter, dominated in most sandstones by quartz, feldspar and rock fragments, bound together by a cement of silica, carbonate or other minerals or a matrix of clay minerals.
saturated zone	that part of the geological stratum in which all the voids are filled with water under pressure greater than that of the atmosphere.
secondary aquifer	an aquifer in which water moves through the secondary interstices, which are a result of post-depositional processes, such as joints and faults.

secondary interstices	openings in the rock that were developed by processes that affected the rocks after they were formed, such as joints and faults.
sediment	particles derived from rocks or biological material that have been transported by air or water.
semi-confined aquifer	an aquifer that is partly confined by layers of lower permeability material through which recharge and discharge may occur.
soil	the usually thin surface layer of the earth, comprising of mineral products formed by the break-down of rocks, decayed organic matter, living organisms, water and the atmosphere.
solute	dissolved inorganic or organic constituents in a solvent.
specific capacity	the rate of discharge of a water well per unit of drawdown, usually expressed as $m^3/d/m$.
specific yield	ratio of the volume of water that given mass of saturated rock or soil will yield by gravity from that mass.
spring	water from an underground source issuing at the surface.
storage coefficient	the volume of water an aquifer releases from or takes into storage per unit surface area of the aquifer per unit change in head.
tortuosity	a measure of the effect of the shape of the flow path followed by water molecules in a porous media. It is equivalent to the actual length of the flow path divided by the straightest distance between the start and finish of the flow path.
transition zone	zone of mixing of injected and native water.
transmissivity	the rate at which a volume of water of the prevailing kinematic viscosity is transmitted through a unit width of aquifer under a unit hydraulic head.
unconfined aquifer	an aquifer whose upper surface (water table) is free to fluctuate.
unconsolidated	non-lithified sediment that has no mineral cement or matrix binding.
vadose zone	that part of the geological stratum above the saturated zone in which the voids contain both air and water.
vug	a cavity in a rock usually with a lining of crystalline minerals.

water bearing	water-yielding in terms of carrying and conveying.
water table	top of the saturated zone in an unconfined aquifer at which pore water pressure is at atmospheric pressure; marked by the position of the water surface.
weathering	the set of all processes that decay and break up bedrock, by a combination of physical fracturing and chemical decomposition.
well	see borehole; often refers to a shallow, wide diameter borehole or a hand-dug water source.
wellfield	a group of boreholes in a particular area, usually used for groundwater abstraction purposes.
yield	quantity of water removed from an abstraction source.