MANUAL ON PUMPING TEST ANALYSIS IN FRACTURED-ROCK AQUIFERS

Part A. Practical Guide for Conducting and Analysing Pumping Tests

Part B. Theoretical Background and Details for Analysing Pumping Tests

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EXECUTIVE SUMMARY

MANUAL ON PUMPING TEST ANALYSIS IN FRACTURED-ROCK AQUIFERS

1. Background and Motivation

This report presents the findings of research conducted by the Institute for Groundwater Studies for the Water Research Commission on the development of a manual to analyse pumping tests in fractured-rock aquifers. The manual provides a detailed summary of the different types of pumping tests and the methods that can be used to analyse data from these tests. The sustainable yield of a borehole can be defined as the discharge rate that will not cause the water level in a borehole to drop below a prescribed limit. Determining the sustainable yield of a borehole is important for the overall management of the aquifer and therefore a whole chapter has been devoted to discussing the term. Protection is an important consideration when managing a well field. For this purpose, computer software, BPZONE, has been developed and included in the FC_EXCEL software. The manual includes a step-by-step guide to assist the reader in planning and executing pumping tests. Finally, recommendations and conclusions are drawn.

Pumping tests are important tools that provide information on the hydraulic behaviour of a borehole, the reservoir and the reservoir boundaries. All this information is essential for efficient aquifer and well field management. In general, the objectives of a pumping test are:

- To obtain an understanding of the aquifer,
- To quantify the aquifer's hydraulic and physical properties and
- To determine the sustainable yield and efficiency of a borehole.

The interpretation of pumping test data is based on mathematical models that relate drawdown response to discharge in the abstraction borehole. The results obtained from these short duration tests can then be used to project the borehole's performance over a long period of time. In fractured-rock aquifers, the geometry and permeability of the system have a large influence on the drawdown. The scale of heterogeneity in a fractured rock system may be large in relation to the scale of the test. Therefore convention models developed for homogeneous porous aquifers might not be viable in fractured rock systems. This manual focuses on methods and software specifically developed to analyse pumping tests conducted in fractured-rock systems.

2. Statement of Objectives

The objectives of this research project are:

- To develop a scientifically sound manual to conduct and evaluate pumping tests in fractured rock aquifers. This manual will be a step-by-step guide for performing and analysing various types of pumping tests such as step drawdown and constant rate tests.
- To determine the optimal rate for a constant rate test.
- To use pumping test data to delineate borehole protection zones.

- To enhance the FC_EXCEL software. On the completion of the project, this software will include the following:
    i. Tools to delineate borehole protection zones, taking into account point source pollution such as pitlatrines.
    ii. Methods to analyse step drawdown and multirate pumping tests.
    iii. Methods to analyse slug tests.

3. Meeting the Objectives

All the objectives of this project were met. In fact, the project team did more than what was required:

- The project team also developed TPA. Test Pumping Analysis or TPA is a windows-based software program, developed in Delphi. The software is a curve fitting procedure for analysing fractured rock pumping test data. TPA includes the following methods to analyse fractured-rock pumping test data:
    i. Moench method for double porosity aquifers.
    ii. Gringarten, Kazemi, Warren and Root and Stallman methods for single and vertical fractures. Included here are uniform flux, finite- and infinite conductive fractures, boundary conditions and a solution for a dyke aquifer.
    iii. Solutions for porous aquifers.
    iv. Diagnostic plots.
- One of the initial objectives of the project was to enhance the FC_EXCEL software to include protection zones and step drawdown, multirate tests, and slug tests. However, the project team went beyond the requirements and included:
    i. Barker's generalised radial flow model, which uses non-integer flow dimensions to analyse pumping data.
    ii. A method to estimate the sustainable yield of a borehole by using the data collected during the step drawdown test (non-linear solution).
    iii. Comparing the suitability of water to the water quality classes developed by the Department of Water Affairs and Forestry.

4. Summary of Methods and Results

Theoretically, the best method to obtain fractured-rock aquifer parameters is a three-dimensional numerical flow model. However, the data required for these models are usually not available. The emphasis of the manual is therefore on using analytical procedures to analyse pumping test data.

The geometry of the fracture system is seldom known when performing a pumping test. Interpretation of test data may therefore have to include identification of the fracture system. Hydraulic conductivity and storativity values for these systems are scale dependent because the rocks are heterogeneous.

A characteristic of fractured aquifers is that most of the water flows along fractures. These fractures are usually embedded in porous matrix blocks (sandstone) or micro fissured blocks (quartzite), which are low permeable compared to the fracture conductivity but are able to store water. If fractures are densely interconnected, they form a 'fracture network continuum' characterised by a large storage capacity that contributes substantially to the yield of a borehole. Whether a fracture network can be considered as continuum or not is determined by the following three properties: the representative elementary volume, fracture connectivity and the difference in conductivity between the fracture and rock matrix.

The spatial distribution of fracture flow towards a borehole depends on the properties of the fracture network, such as fracture permeability, fracture density and connectivity. The characteristics of the fracture network determine the flow geometry, which is also known as **flow dimension**. The flow towards a fully penetrating borehole may be radial (two-dimensional) throughout a hard rock aquifer consisting of a highly connected fracture system with an isotropic distribution, or may be spherical (three-dimensional) if the borehole is only partially penetrating. Therefore, for a correct assessment of the hydraulic parameters of a fractured hard rock aquifer, it is necessary to know the flow geometry, or flow dimension.

Two kinds of models are applied in fractured-rock aquifers, namely the single fracture model and the double porosity model. However, the methods for porous media (i.e. Theis and Cooper-Jacob) are still applied for parameter estimation in fractured aquifers. Some of these models have inherent problems. The methods of Theis and Cooper-Jacob are only valid for radial flow.

Numerous field examples have been included in the manual. These demonstrate how to conduct and analyse pumping tests. They also highlight the limitations of the various tests and methods. The results from these tests indicate that *monitoring* must always be an integral part of aquifer management.

When determining the sustainable yield of a borehole, field examples indicate the yield is a non-unique number, dependent on the abstraction rate.

5. Conclusions

The development of this manual was driven by the need to establish a scientifically sound and documented approach to performing and analysing pumping tests in fractured rock systems. The manual has resulted from extensive consultations with pumping test experts and hours of extensive research on methods to analyse these pumping tests.

It is concluded in this manual that in order to analyse a pumping test correctly, the following important characteristics need to be identified:

- The correct geological conceptual model.
- Inner boundary conditions such as well bore storage effects.
- Outer boundary conditions, for example no-flow boundaries.
- Characteristic flow regimes.

In South Africa, pumping tests are performed for mainly two reasons: to determine the long-term sustainable yield of a borehole and to estimate aquifer parameters. The manual highlights the non-linear relationship between the abstraction rate and the drawdown, which is common in most South African aquifers. Fracture dewatering is dependent on the abstraction rate, which, in turn, will have an effect on the water level in the borehole. This feature should be treated with caution when assigning sustainable yields.

It will always be a challenge to perform and analyse pumping tests in a fractured-rock environment. This manual therefore highlights many of the limitations involved with each of the methods discussed. Hopefully the reader will be aware of many of the mistakes that can so easily be made when conducting these valuable tests.

6. Recommendations

When the objective of a pumping test is to determine aquifer parameters, the following is recommended:

- Perform a minimum one-hour step drawdown test to determine the position of the fracture and abstraction rate for the constant rate test.
- The recommended minimum duration of the constant rate pumping test is 8 hours. RPTSOLV is an excellent two-dimensional model which analyses pumping test data and calculates an accurate storativity value.

When calculating the sustainable yield, it is recommended that on the completion of the constant rate pumping test, the FC_EXCEL software is used to analyse the results.

Sustainable development of groundwater resources refers to a holistic approach to development, conservation and management of these resources. Good sustainable yield estimates are the result of scientifically sound pumping test analysis. It must be noted that these quantities vary with time and location and can only be estimated, and thus may carry a degree of uncertainty. All sources of uncertainty need to be recognised and their impact on water quality or the sustainability of the aquifer must be evaluated.

An essential consideration in the development of an aquifer is the chemical quality of water produced, as the quality can limit its use. Groundwater movement induced by pumping may change the groundwater chemistry. Currently, the FC_EXCEL software includes methods to delineate borehole protection zones from certain microbial pollution sources. This section can be developed further to include chemical contaminants.

A large number of pumping tests that were performed on boreholes situated in fractured rocks in South Africa indicate that a non-linear relationship between drawdown and abstraction rate usually exists. Such non-linearities must be treated with caution when assigning sustainable yields for boreholes or when applying numerical flow models.

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INTRODUCTION

1 PROJECT OBJECTIVES

The aims of the project are:

- Compilation of a manual how to conduct and evaluate pumping tests in fractured-rock aquifers.
- Choice of the length of a constant rate test.
- Use of pumping test data to delineate borehole protection zones.
- Enhancement of the current used FC_EXCEL program.
- Development of windows software for fitting analytical solutions.

2 SOFTWARE DEVELOPED

Two software packages were developed or enhanced during the current study:

- FC (Flow Characteristic method).
- TPA (Test Pumping Analysis).

TPA is a windows program in DELPHI and was written by Ingo Bardenhagen as part of his Ph.D. study at the Institute for Groundwater Studies, while Gerrit van Tonder, Harald Kunstmann and Yongxin Xu developed the original FC_EXCEL spreadsheet for the Department of Water Affairs and Forestry (SA).

During the current project, the **FC** software was enhanced and currently includes the following procedures:

- Porous aquifer solutions (Theis, Cooper-Jacob I and II and Hantush methods and a solution for water-table aquifers).
- Step drawdown and multirate analyses.
- Fractional pumping test analysis (Barker's Generalized Radial Flow Model).
- Slug test analysis (Bouwer and Rice method).
- Estimation of a risk-based sustainable yield of a borehole by using drawdown derivatives, boundary information and error propagation.
- Testing of the suitability of the water according to the Classes as prescribed by the DWAF.
- Different diagnostic plots for flow regime identification (e.g. derivatives, second derivatives, LogLog (Theis) plot, LinLog (Cooper-Jacob) plot, square root of time plot, fourth root of time plot, spherical and recovery plot).
- Delineation of borehole protection zones in fractured aquifers.

The main emphasis of the FC program is to estimate a risk-based sustainable yield for a borehole by using different methods. The estimation of the sustainable yield of a borehole is very important for especially rural communities.

TPA was developed with the aim to fit pumping test data in fractured aquifers and include the following fractured aquifer methods:

- Double porosity aquifer (Moench method).
- Solutions for single vertical and horizontal fractures (Gringarten, Kazemi, Warren and Root, Stallman, including uniform flux, finite conductive and infinite conductive fractures as well as boundary conditions and a solution for a dyke aquifer).
- Porous solutions.
- Diagnostic plots.

TPA was specially developed as a curve fitting procedure for pumping tests performed in fractured-rock aquifers to estimate aquifer parameters.

It is well recognised that on a theoretical basis the best method to obtain fractured-rock aquifer parameters is by the use of a 3D-numerical model, like MODFLOW. However, the data required for such a numerical model may not always be available and the application of the model also requires an experience user and the construction of the correct conceptual model for the geological set-up. The emphasis of this document will thus be on the application of analytical procedures to analyse pumping test data as well as on the correct procedures to conduct pumping tests.

3 PREAMBLE

The term fracture in this document refers to cracks, fissures, joints and faults, which are caused by (a) geological and environmental processes, e.g. tectonic movement, secondary stresses, release fractures, shrinkage cracks, weathering, chemical action, thermal action; and (b) petrological factors like mineral composition, internal pressure, grain size, etc. From a hydrogeological point of view, a formation rock mass can be considered a multi-porous medium, conceptually consisting of two major components: matrix rock blocks and fractures. Fractures serve as higher conductivity conduits for flow if the apertures are large enough, whereas the matrix blocks may be permeable or impermeable, with most of the water usually contained within the matrix. Actually, a rock mass may contain many fractures of different scales and the conductivity of the matrix blocks is in most cases only dependent on the presence of smaller or micro-fractures.

Pumping tests are the most important experiments for aquifer investigation in the groundwater industry. They are the only method that provides simultaneous information on the hydraulic behaviour of the well (borehole), the reservoir and the reservoir boundaries, which are essential for efficient aquifer and well field management. The complex situation in fractured aquifers requires a decent understanding of the drawdown behaviour if reliable reservoir information is desired. This can be achieved by a detailed diagnosis of drawdown and recovery data in combination with a conceptual model of the geological set-up.

The general objectives of a pumping test are:

(i) A better understanding of the aquifer system,
(ii) Quantification of its hydraulic characteristics and properties and
(iii) An assessment of both the sustainable yield and the condition or efficiency of the borehole. The **sustainable yield** is defined as the discharge rate that will not cause the water level in the borehole to drop below a prescribed limit (the position of a major water strike, for example). It is also important that the total abstraction rates of boreholes situated in an aquifer must not exceed the sustainable yield of the aquifer in total (i.e. the average annual recharge).

The interpretation of pumping test data is based on mathematical models that relate the drawdown response to the discharge of the pumped well and the results obtained from this short duration test are then used to estimate the borehole performance over a period of many months (even years). The mathematical model could be solved by the application of analytical or numerical techniques.

There is no doubt that a hydraulic test in a fractured aquifer should be analysed with a three-dimensional numerical model from the theoretical point of view. However, the data required for such a numerical model may not always be available and the application of the model is also not without its problems (e.g. the choice of the correct conceptual model and the non-uniqueness of the solution; Chiang and Riemann, 2001).

The drawdown response to stressing a fractured rock aquifer is strongly influenced by the geometry and permeability of the fracture system, i.e. by flow geometry. The scale of heterogeneity in a fractured-rock aquifer may be large in relation to the scale of the test and the conventional well flow equations, developed for homogeneous porous aquifers, will usually not adequately describe the drawdown response in fractured-rock aquifers. In pumping test practice, the geometry of the fracture system is seldom known beforehand. Interpretation of test data may therefore require identification of this unknown fracture system. The values of the hydraulic parameters (hydraulic conductivity and storativity) for hard rock aquifers are scale-dependent (Verweiji and Barker, 1999) because of the heterogeneity of the rocks.

On the field scale involved in pumping tests, the groundwater flow through porous media is commonly described by applying the **continuum approach** and the flow to a borehole is then described by the diffusion equation, which is a combination of Darcy's equation and equations of state for the groundwater and the porous medium. Characteristic for fractured aquifers is the fact that most of the water flows along fractures. Those fractures are usually embedded in porous matrix blocks (sandstone) or micro fissured blocks (quartzite), which are low permeable compared to the fracture conductivity, but capable to store water in the uncountable pores or micro fractures. In extreme cases, the blocks between the fractures are so low permeable (granite) that very little water can be exchanged between fracture network and matrix, which is then called 'inert'.

If fractures are densely interconnected, they conform a 'fracture network continuum' characterised by a large storage capacity that contributes substantially to the volume extracted by a pumped well. Whether a fracture network can be considered as continuum or not is determined by the following three properties:

- Representative elementary volume (REV) - also called Proper Sample Volume.
- Fracture connectivity.
- Conductivity contrast between fracture and matrix.

The REV is the characteristic volume of fractured rock that can be represented by a homogeneous anisotropic medium whose hydraulic properties do not change significantly if an additional volume of rock is added (Long *et al*., 1982). In some instances, a fractured rock can have various REV depending on the scale of the investigation and, in some instances, it is not possible to define a REV at all (Long and Witherspoon, 1985). The fracture connectivity describes the interconnection between fractures in a given volume of rock, which is a function of the fracture length and fracture density. Generally, the fracture network continuity of a rock volume increases with increasing fracture length and fracture density (Long and Witherspoon, 1985). The conductivity contrast between fracture and matrix can diminish or increase the continuous behaviour of a fracture network.

Verweiji and Barker (1999) give an excellent summary of the flow behaviour and analytical techniques used in fractured-rock aquifers and interested readers are referred to their paper for more information. Boreholes used for water supply in fractured-rock aquifers are often sited on linearities such as faults, major fracture zones and fractured dykes, but may also be drilled randomly. The flow to a well pumping in a fractured aquifer takes place through the fractures but in most cases the water in storage is situated in the matrix of the formation. Initially, water is pumped from storage in the fracture system and then water is flowing from the matrix towards the fractures. .

The spatial distribution of flow through the fractures towards the well depends entirely on the properties of the fracture network, such as fracture conductivity, fracture density and connectivity (e.g. Barker, 1988; Black, 1994). The characteristics of the fracture network determine the flow geometry, which is also known as **flow dimension**. Flow towards a pumped well is usually concentrated along a certain fracture zone, while large volumes of hard rock could remain isolated from this zone of relatively high permeability activated by the pumping test. In contrast, the pattern of flow towards a fully penetrating well may be radial (two-dimensional) throughout a hard rock aquifer consisting of a highly connected fracture system with an isotropic distribution, or may be spherical (three-dimensional) if the well is only partially penetrating. For a correct assessment of the hydraulic parameters of a fractured hard rock aquifer using pumping test data, it is necessary to know how much flow relates to a certain drawdown response of the aquifer, i.e. it is necessary to know the flow geometry or flow dimension (Verweiji and Barker, 1999). Roberts and Beauheim (2001) discussed the concept of a flow dimension and show that it actually represents the rate at which hydraulic conductance (the product of hydraulic conductivity and through-flow area) changes with distance from a test borehole. **No unique estimation of hydraulic parameters is possible from hydraulic tests without knowledge of the flow geometry**.

According to Black (1994), the area available to flow in an aquifer is a function of the distance to the pumped well and is given by:

$$A = A_O \, r^{(n-1)}$$

where,

A = through-flow area
A_O = through-flow area at the pumped well ($r = 0$)
n = flow dimension
r = distance to the pumped well

Flow dimensions range between 1 and 3 (Barker, 1988; Black, 1994) and for a one-dimensional flow geometry ($n = 1$), the area through which flow occurs will remain constant, regardless of the distance r (Fig. I.1). A period of linear flow through a fracture or dyke during a pumping test, when all the water pumped originates from storage in the dyke or fracture corresponds to a flow dimension of 1. For a flow dimension of 2, corresponding to radial flow to a well, the through-flow area will increase in direct proportion to the distance r from the pumped well. This kind of flow dimension is not an intrinsic property of a fractured hard rock aquifer because it usually changes in time during a pumping test, which is reflected in the time drawdown behaviour at the pumped well and observation wells. The drawdown response at a certain time, as observed at the pumped well and observation wells, might all reflect different flow dimensions. The flow dimension prevailing during a test is thus a function of scale (time).

The flow to a well in a hard rock aquifer that consists of a fracture system having **fractal properties** may not be adequately characterised by a one-, two- or three-dimensional flow geometry. Instead, non-integer fractional flow dimensions will describe the flow behaviour more accurately (Barker, 1988). The value of the fractional flow dimension depends on the fracture geometry like orientation, connectivity and variability of aperture.

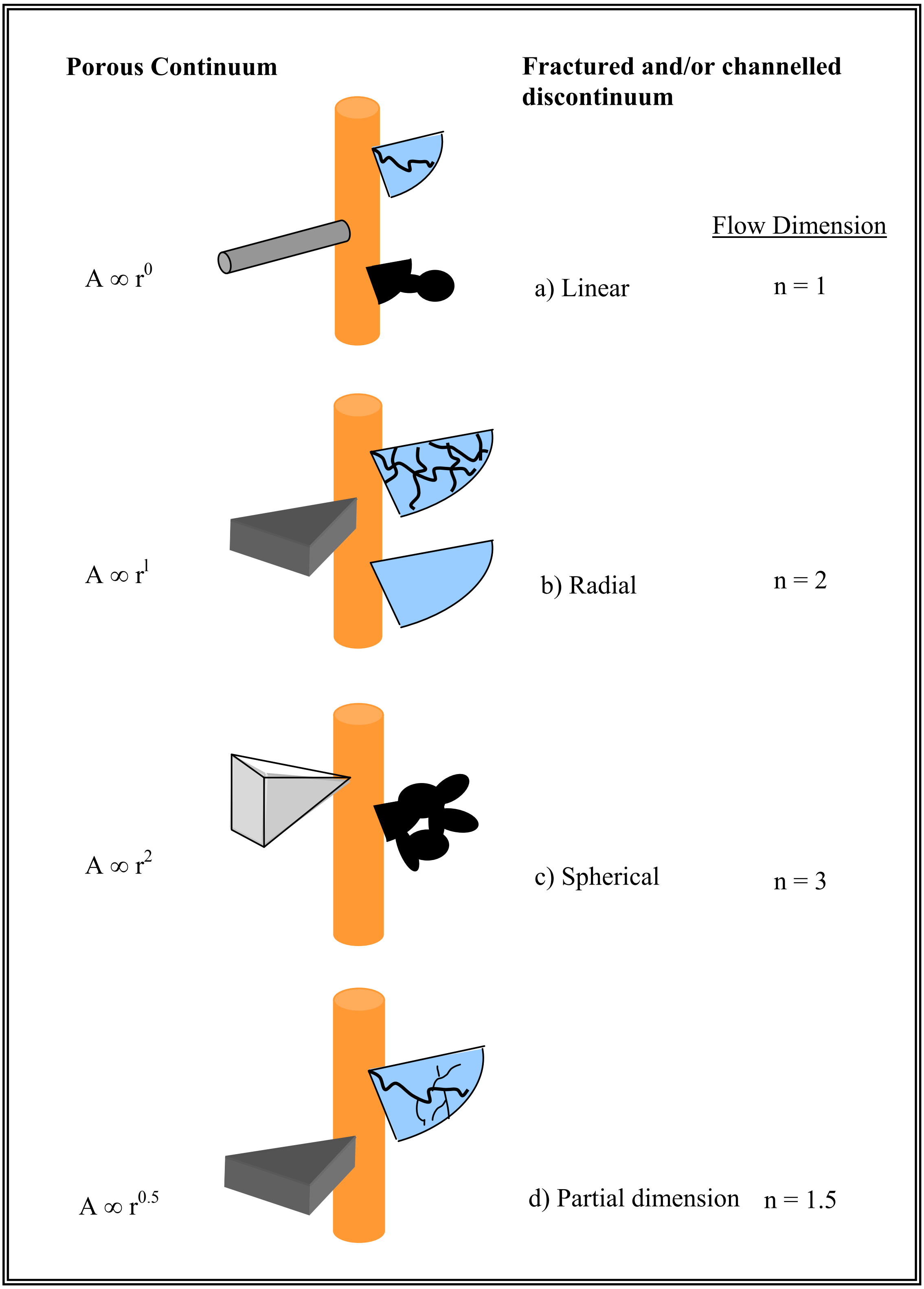


Fig. I.1 Flow dimension definition in well testing (after Doe, 1991)

The time and scale dependence of flow dimension can be illustrated by looking at the flow pattern created when a fractured dyke or fault bisecting an aquifer, the transmissivity of which is several times less than that of the fault, is pumped (Boehmer and Boonstra, 1986; Kruseman and de Ridder*,* 1991). At early pumping times, when all the water pumped originates from storage in the fault and none is contributed from the aquifer, the flow towards the well takes place in the fault only and is parallel (i.e. linear in the fault with a flow dimension = 1). With continued pumping, the water is also supplied from the aquifer. The flow in the aquifer will be predominantly parallel but oblique to the fault (i.e. linear formation flow). This bi-linear flow period corresponds to the non-integer flow dimension of 1.5. Finally, the flow in the aquifer may become pseudo-radial, corresponding to a flow dimension of 2. Information on flow dimensions can be derived from the pumping test data. Special analytical methods have been developed for different kinds of flow dimensions. The concept of flow dimension plays an important role in selecting the appropriate time-drawdown data to be used in a certain method.

Currently usually two kinds of models are applied in fractured-rock aquifers, namely the single fracture model and the double porosity model as described in Chapter 2.7 (Part B), while the methods for porous media (i.e. Theis and Cooper-Jacob) are still applied for parameter estimation in fractured aquifers. Some of these models have inherent problems, as the following short discussion indicates (see also Chapter 2.8 in Part B).

Single fracture models

If a pumped well intersects a major fracture, fault or dyke, embedded in a very hydraulic conductivity matrix, it may significantly influence the part of the time drawdown behaviour of the fractured hard rock aquifer. Single fracture models deal with such a situation. The main types of single fracture models are: infinite conductivity fracture, uniform flux fracture, finite conductivity fracture, and dykes. The first three models have been developed for artificially fractured wells in petroleum reservoirs (Verweiji and Barker, 1999). Only the dyke model has been developed for groundwater purposes. The equivalent system in a single fracture model is a homogeneously confined aquifer of large areal extent, which is dissected from top to bottom by a vertical fracture of relatively short length or by a dyke of infinite length. The pumped well fully penetrates the fracture or dyke.

Infinite conductivity fracture

A pumped well tapping an assumed plane fracture of infinite conductivity will initially receive water from the aquifer by linear flow towards the fracture (and the well) and after continued pumping, the water will flow towards the well by radial flow (e.g. Gringarten *et al.,* 1974). **The assumption of an infinite conductivity in the fracture, however, cannot be considered realistic for highly conductive fractures in hard rock aquifers with low permeable rock matrices (Verweiji and Barker, 1999)**.

Uniform flux fracture

In the Gringarten vertical fracture model (Gringarten *et al.,* 1972), a uniform flux condition is assumed to exist, i.e. water from the aquifer is assumed to enter the fracture at the same rate per unit area. **In reality, the flux distribution along a fracture may approach uniformity only if there is a skin, and a low permeability layer between the fracture and the aquifer matrix** (Cinco and Samaniego, 1981b).

Finite conductivity fracture
The flow to a well for a model with a single vertical fracture of finite conductivity is decribed by Cinco *et al.* (1978) and Cinco and Samaniego (1981a). Three different flow periods are reconised: an initial bilinear flow period reflecting the combined result of linear flow within the fracture and linear aquifer to fracture flow, followed by a rarely observed linear flow period and a final radial flow period.

Double porosity model

The Moench model (1984) is usually applied for double porosity aquifers, but it was found from practical applications of the method that the **estimated S-values still show the same distance dependency that is experienced when the Theis method (using the real observation distances) is applied to pumping tests in fractured aquifers (see e.g. Kirchner and Van Tonder, 1995). Six parameters have also to be fitted which makes the solution non-unique.**

To analyse a pumping test correctly, the geohydrologist should be able to identify the following important characteristics:

- The correct geological conceptual model (for use of the correct analytical model).
- Inner boundary conditions (i.e. well bore storage effects, well bore skin, fracture skin and the lateral extent of the fracture or fracture zone).
- Outer boundary conditions (i.e. especially no-flow boundaries but also fix head boundaries).
- Characteristic flow regimes (the choice of the correct part of the curve to be fitted by looking at the flow dimension that is prevailing during that specific part of the test).

One of the main problems with all analytical models is the underlying theoretical assumptions. In most cases, a confined situation is assumed and practical experience in SA has shown that the shallow fractured rock aquifers are usually semi-confined (the fractures) with a water-table aquifer (the matrix) situated on top of it. In many cases during the execution of a pumping test, some of the fractures are dewatered, with the result that the conditions changed from semi-confined to unconfined at the fracture position (a flattening of the water level is experienced at the position of a fracture because the specific yield of the fracture is much higher than the confined storage coefficient of it). Dewatering of some fractures during the test also implies that the effective transmissivity is becoming smaller as time is progressing. This phenomenon could result in a wrong interpretation of a step drawdown or multirate test, which implies that the non-linear relationship between drawdown and abstraction rate is due to turbulent flow (non-linear well loss coefficient C) in the borehole. However, it will be shown in this document that such a non-linear relationship between drawdown and abstraction rate could also occur during laminar (Darcian) flow in fractured-rock aquifers.

4 WAY TO USE THE MANUAL

There are mainly two objectives in doing pumping tests:

- Estimation of aquifer parameters
- Recommendation of a sustainable yield for a borehole

The way to perform and analyse the pumping test depends mainly on the objective and on the site where the test should take place. The manual (Fig. I.2) is divided into a practical guide for conducting and analysing pumping tests (Part A), the theoretical backgrounds and detailed explanation for analyses (Part B) and the detailed explanation for conducting pumping tests (Appendix A).

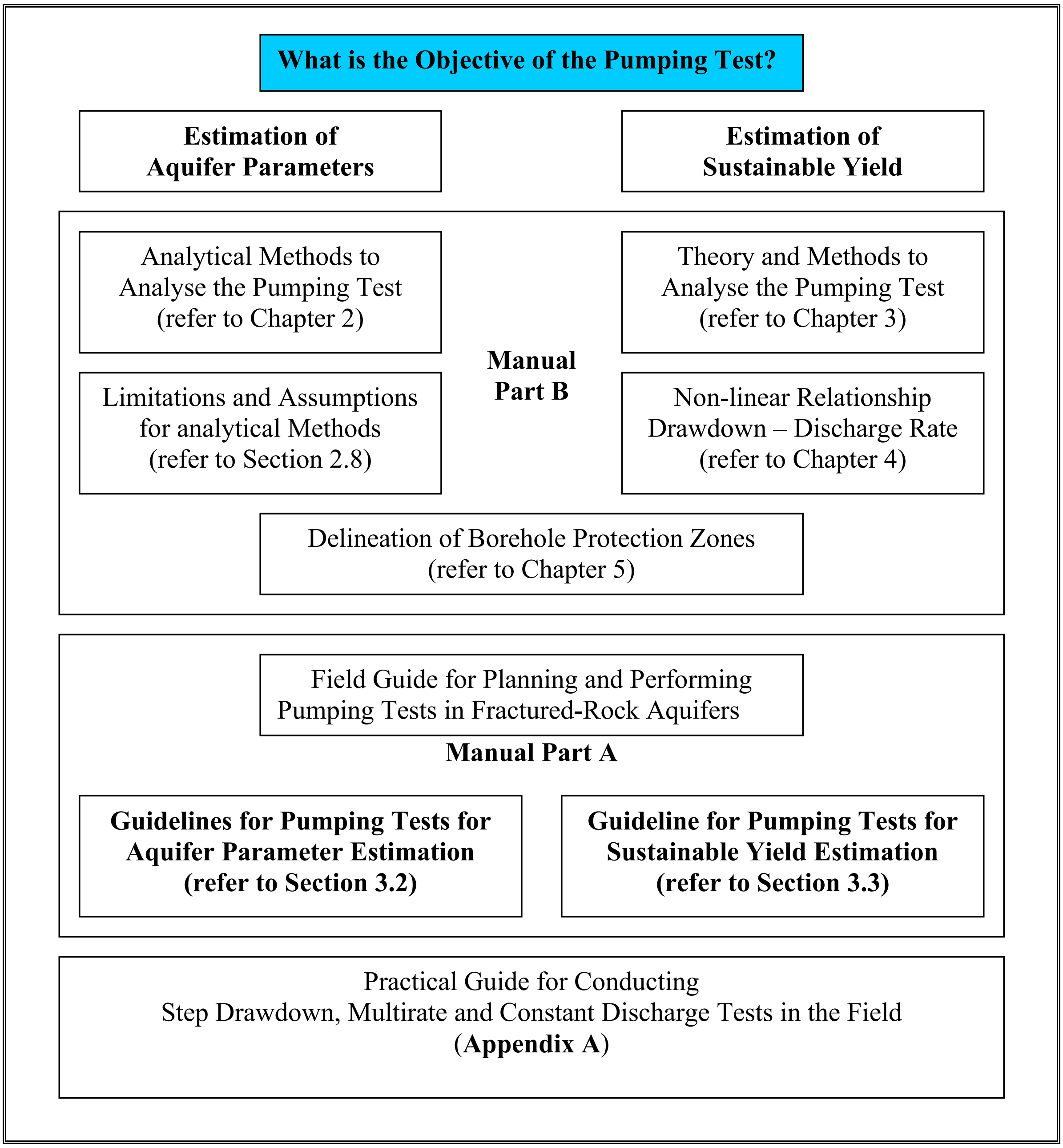


Fig. I.2. Flow chart for the different parts of the manual

While Part A is a short guideline in how to perform and analyse pumping tests in fractured-rock aquifers, Part B will focus on the application of different methods for analysing pumping test data. Part A consists of a field guide on planning and performing the test (Chapter 2) and a guideline for pumping test analysis, depending on the objective of the test. In Chapter 3, the reader could find a discussion on the optimal planning of hydraulic tests (with special emphasis on the choice of an abstraction rate for the constant rate tests and the choice of the available drawdown to use for recommending a sustainable yield).

Readers that are interesting in all the analytical methods and the different flow diagnostics are refered to Part B, Chapter 2. Chapter 3 is focussed on the estimation of the sustainable yield of a borehole, while Chapter 4 is devoted to variable rate tests and non-linear relationships between drawdown and abstraction rate. Chapter 5 is dealing with the delineation of borehole protection zones in fractured aquifers and in Chapter 6, the reader could find the recommendations concluded from the whole manual.

Appendix A describes the practical execution of the different kind of pumping tests in the field.

Both the FC- and TPA- programmes can be downloaded from the following website:

http://www.uovs.ac.za/faculties/igs/software.htm

The RPTSOLV program developed during the project of Botha *et al*. (1998) can also be downloaded from this website.

Note: No written manual of how to use both the FC- and TPA-program is given in this manual. The reason is that such a manual is always changing**. A step-by-step manual of how to use the software is built into the programmes**.

MANUAL ON PUMPING TEST ANALYSIS IN FRACTURED-ROCK AQUIFERS

PART A

PRACTICAL GUIDE FOR CONDUCTING AND ANALYSING PUMPING TESTS IN FRACTURED-ROCK AQUIFERS

CHAPTER 1

GENERAL PRINCIPLES

1.1 GENERAL

Pumping tests are the most important experiments for aquifer investigation in the groundwater industry. They are the only method that provides simultaneous information on the hydraulic behaviour of the well (borehole), the reservoir and the reservoir boundaries, which are essential for efficient aquifer and well field management. The complex situation in fractured aquifers requires a decent understanding of the drawdown behaviour if reliable reservoir information is desired. This can be achieved by a detailed diagnosis of drawdown and recovery data in combination with a conceptual model of the geological set-up.

As stated before, the two main objectives of pumping test analysis are:

- **Estimation of aquifer parameters**
    - The aquifer parameters are important for management purposes (yield and pollution management). In combination with the geological set-up, they are used for the construction of the correct conceptual model.
    - Usually numerical models are used for well-field management with the objective to optimise pumping rates subject to certain drawdown constraints.
    - In cases of contaminated groundwater, the aquifer parameters are important for risk assessments and the planning of groundwater remediation.
    - Depending on the demands, several observation boreholes and piezometers at different depths will be used for measurements.

- **Estimation of the sustainable yield of a borehole**
    - If single boreholes are used for private purposes, for example irrigation on farms, it is necessary to estimate the sustainable yield of the borehole, avoiding the drying up of the borehole.
    - In this case, only the abstraction borehole is measured, as no observation boreholes are available due to the high cost associated with the drilling of boreholes.

From the above-mentioned points, it is clear that the principles for conducting and analysing pumping tests depend on the objectives.

The main goal of Part A is to suggest general rules for the practical geohydrologist on how to perform and analyse a pumping test considering the two objectives. Every borehole in a fractured-rock aquifer reacts differently, and therefore it is of no use to give one general recipe on how to conduct and analyse it. In the end, the expert analysing the test is totally responsible for the way in which the test is conducted and analysed. He or she must use all their practical experience and knowledge to come up with an answer. The aim is also to demonstrate the **limitations** of analytical analysing methods. Usually, the limitations are due to the assumptions underlying the theory of the method.

1.2 PRINCIPLES FOR PARAMETER ESTIMATION

If the objective of the pumping test is to estimate aquifer parameters that are to be used in e.g. a numerical management model, the constant rate test is the most important test and is set as **minimum requirement** for parameter estimation (Fig. 1.1). Although a slug test and step drawdown (or multirate) test can also be conducted, it is not of much practical value. If the interest is setting up a 3D numerical model, a number of piezometers must be installed (to measure pressure heads and vertical K-values of each layer). One of the most important factors of a constant rate test in this case is selecting the abstraction rate during the test. The yield must be chosen in such a way that no main water-yielding structures will be dewatered during the test.

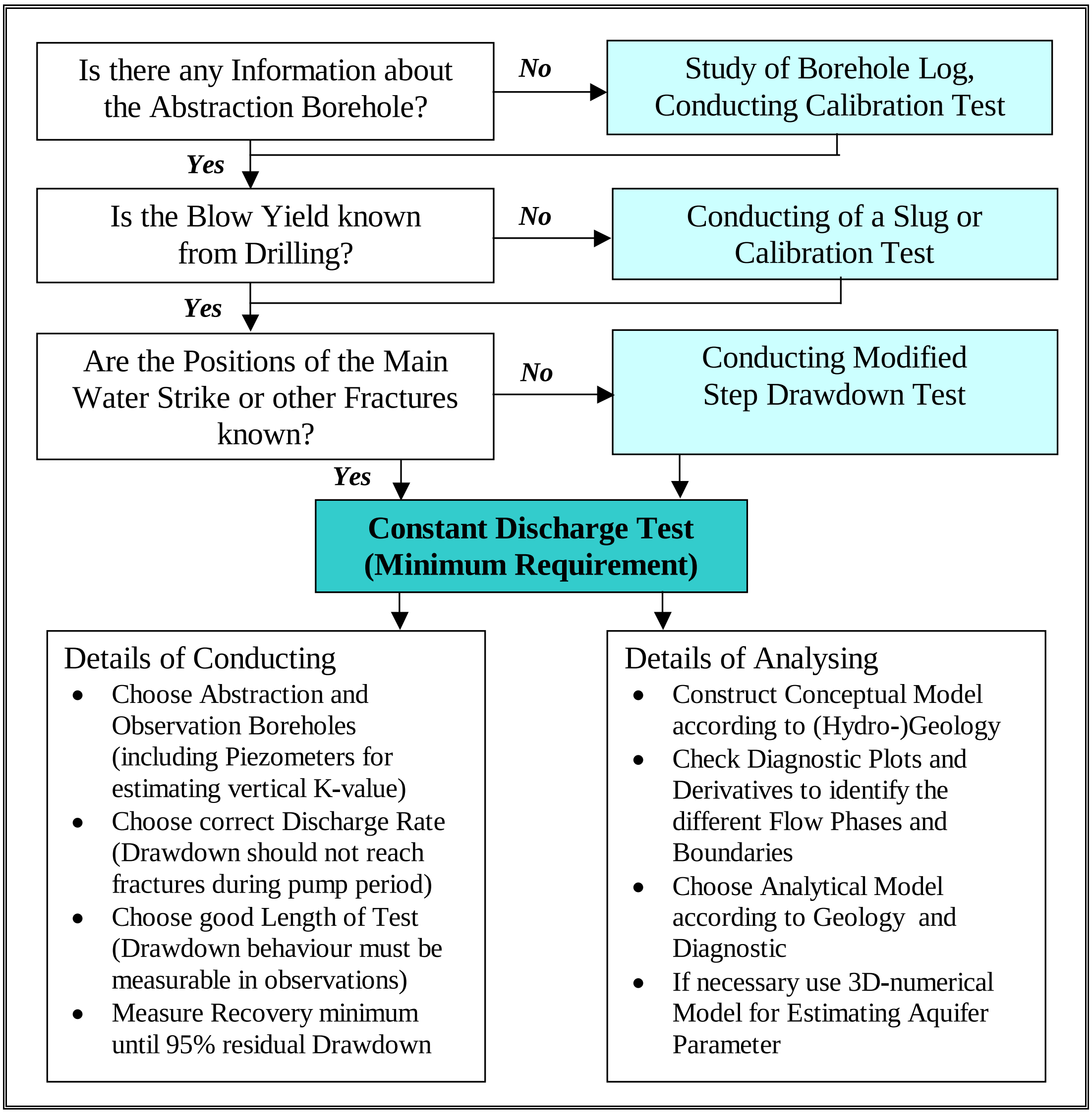


Fig. 1.1 **Steps for Parameter Estimation**

1.3 PRINCIPLES FOR SUSTAINABLE YIELD ESTIMATION

If the objective of the pumping test is to estimate the sustainable yield of a single borehole, it is not necessary to use different analytical or numerical models to estimate aquifer parameters. According to the definition of the sustainable yield, it is necessary only to obtain the relationship between abstraction rate and drawdown in the borehole. Therefore as **minimum requirement** for estimating sustainable yield, a **minimum** of one constant rate test must be conducted, stressing the aquifer (Fig. 1.2). A step drawdown test with a minimum duration of one hour is also set as **minimum requirement**. To get prior information, a slug test can also be performed. However, it is normally not of much practical value. One of the most important factors to consider is selecting an appropriate abstraction rate, when performing a constant rate test. The yield must be chosen so that the main water strike will be reached during the constant rate test.

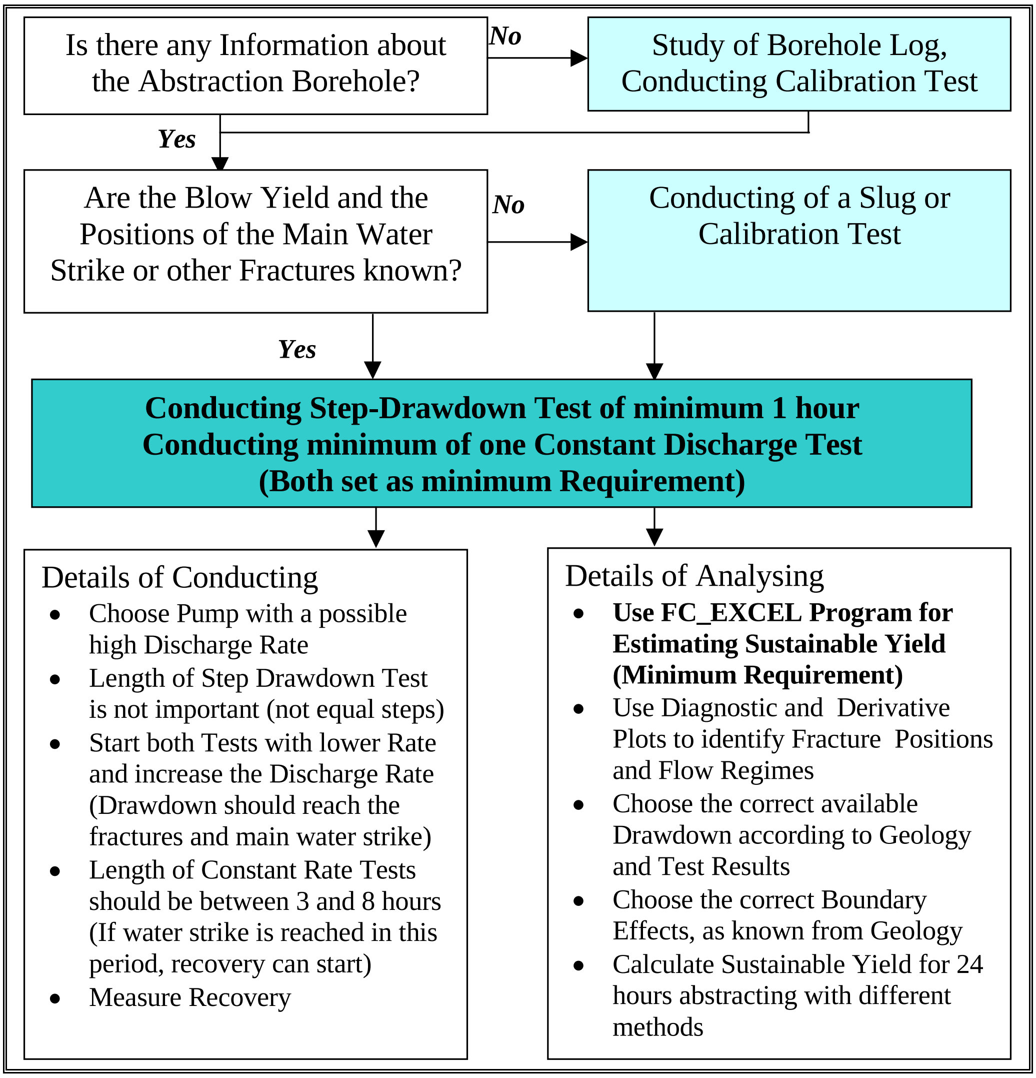


Fig. 1.2 Steps for Sustainable Yield Estimation

CHAPTER 2

FIELD GUIDE ON PLANNING AND PERFORMING PUMPING TESTS

This is a condensed field guide explaining the different steps involved in planning and executing a pumping test. For more detail on the different steps, Appendix A should be consulted.

2.1 PLANNING OF PUMPING TESTS

The purpose of the pumping test should be identified in the beginning of the planning stage because this will influence the execution of the test. The objectives can either be to estimate aquifer parameters to use in a numerical management model or it can be to estimate the long-term sustainable yield for the borehole.

- If the objective of the pumping test is to estimate aquifer parameters, Figure 1.1 gives an indication of the steps that should be followed.

- If the objective of the pumping test is to estimate the long-term sustainable yield for a borehole, Figure 1.2 gives an indication of the steps that should be followed.

When the scope of the test or tests are fixed in a contract between the party that requires the test, the contractor who will perform the test and the geohydrologist who has to analyse the test, the planning stage can start.

The first and most important step in the planning phase is to determine the kind of pumping test (calibration test, slug test, step-drawdown test, multirate test, constant discharge test) and to choose the abstraction and observation boreholes. The geohydrologist should visit the site and check the chosen boreholes to make sure that the test can be performed in the suggested way.

The second step is to gather all information, which is available about the different boreholes and the aquifer, to ensure the correct planning of the depth and abstraction rate for the pump.

Step third step in the planning phase is to gather the necessary equipment for the pumping test and to check their ability to ensure the successful and accurate performing of the pumping test.

The different steps and the responsibilities of the geohydrologist and the pump test contractor are described in detail in Section 2.3.1.

2.2 PERFORMING OF PUMPING TESTS

The performing phase can be divided into three parts:

- Installing equipment and preparing the test.
- Conducting the tests and measurement.
- Removal equipment and check the field data.

Prior to installing the equipment for the pumping test, the abstraction borehole should be checked for damages (e.g. incorrect depth, damaged casing, damaged column).

The best type of pump to be used for pumping tests is a positive displacement pump like the Mono-type pump. The yield of this type of pump remains constant and will not change if the water level drops. The main drawback of a positive displacement pump is that it is not easy to specify a pre-selected abstraction rate. Submersible (negative displacement) pumps can be used, but the abstraction rate will decrease with an increase in drawdown. It is, however, a simple task to set an exact initial pumping rate.

The maximum yield of the pump must be chosen so that the drawdown, which is necessary to achieve the goal of the pumping test, will be reached in time.

During the test measurements, the water levels in the chosen boreholes and the discharge rate must be taken in fixed time intervals, which are prescribed by the geohydrologist in the planning phase. The duration of the test, the chosen discharge rate and the conditions of the measurements (manually, electronically) depend on the kind and the goal of the test.

Every measurement and observation on site should be recorded on the pump test form. Especially changes in the condition of the test, like change of discharge rate, heavy rainfall or interruptions of the test due to technical problems, must be recorded to ensure that the geohydrologist can analyse this test correctly.

After finishing the test, the equipment should be removed from the borehole, cleaned or decontaminated (if necessary) and the site should be left in the same condition than before the test. The field data and all gathered information should be checked direct after the test and before beginning the analysis. Any question of uncertainty should be solved between the pumping test contractor and the geohydrologist as soon as possible.

The different steps and the responsibilities of the geohydrologist and the pump test contractor are described in detail in Sections 2.3.2 to 2.3.4.

2.3 RESPONSIBILITIES

In order to perform a successful pumping test the active participation of different people is required. On the one hand there is the pumping test contractor who is responsible for gathering the field data and on the other hand there is also the geohydrologist who is responsible for the initial planning of the test as well as the evaluation and interpretation of the field data. This guide will look at each person's responsibilities as well as the necessary interaction between the two parties to ensure the success of the pumping test.

2.3.1 Before the start of the pumping test

2.3.1.1 Geohydrologist

- The different tests that are to be performed must be determined before the commencement of the testing and the minimum length of each test must be specified.
- The geohydrologist must ensure that a proper contract exists between the interested parties. On the one hand the party that requires the pumping test and on the other hand the pumping test contractor that will perform the work. The scope of the work that is to be performed should be described in detail.
- The geohydrologist must determine the exact position of the pumping test borehole as well as the positions of possible observation boreholes. These positions should be supplied to the pumping test contractor and during a site meeting these positions can be mutually agreed upon.
- Existing pumping equipment and the removal and reinstallation of this equipment to enable the contractor to perform the pumping test must be described in the contract.
- As much information as possible regarding the pumping borehole and the aquifer must be gathered by the geohydrologist prior to the beginning of the pumping test. Information on borehole logs, maps and aerial photographs, existing pumping tests, blow yields, possible abstraction rates, positions of fractures as well as the depth of the borehole should be obtained in order to assist the pumping test contractor.

2.3.1.2 Pump test contractor

- Enter into a proper contract to ensure payment and correct project information. Issues such as the removal and reinstallation of existing equipment must be included in the contract.
- Obtain as much information as possible regarding the geology, borehole logs, blow yields, possible abstraction rates, positions of fractures as well as the depth of the borehole from the geohydrologist.
- Get permission from the owner of the property to perform the pumping test.
- Gather the equipment to do the pumping test. A list of the equipment is included in ***Appendix A1***.
- Ensure that a pump, capable of pumping sufficient volumes of water for long enough periods of time, is available to perform the testing.
- Borehole co-ordinates and numbers must be recorded.

2.3.2 Arrival on site actions

2.3.2.1 Pump test contractor

- Arrange a site meeting with the geohydrologist. Locate the correct borehole that is to be pump tested as well as the identified observation boreholes. If no observation boreholes had been specified, locate possible observation boreholes in the vicinity of the designated borehole to be pump tested.
- Remove existing pumping equipment from the designated abstraction borehole and make notes about the condition and type of equipment as well as the depth of installation.
- Determine the depth and diameter of the abstraction borehole as well as the observation boreholes.
- Measure the distances from the abstraction borehole to the different observation boreholes as well as the collar heights above the natural ground level.
- Measure the static water levels in the abstraction borehole as well as the observation boreholes.
- If no information regarding the blow yield is available, perform a slug test to get a first estimate of the possible maximum yield of the pumping borehole. If the possible maximum yield is found to be less than 0.3 L/s (guideline value), the geohydrologist must be contacted and a decision whether to stop or continue with the testing must be taken.
- If testing is to continue, the pumping equipment should now be installed into the abstraction borehole. The intake of the pump should be placed at the correct depth.
- Discharge piping must be attached to the pump to ensure that the water pumped from the borehole is discharged away from the borehole. The length of discharge piping as specified by the geohydrologist must be used.
- Equipment to measure the amount of water that is pumped from the abstraction borehole should be installed. The discharge from the pump should be measured and recorded at regular intervals.
- In some cases, electronic data logging equipment is prescribed to do water level measurements. If prescribed, this equipment should now be installed. If not required, water-level measurements can be taken with a dip tape and time intervals recorded with a stop-watch.
- The actual pumping test can now start.

2.3.3 During testing

2.3.3.1 Pump test contractor

- If the objective of the pumping test is to determine parameter values for the aquifer, the pumping test contractor will proceed as follows:
  - As mentioned before, a slug test will be performed to give a first estimate of the possible maximum yield for the borehole.
  - To perform a proper constant rate pumping test, the correct pumping rate should be chosen. The pumping rate should allow for sufficient drawdown during the constant rate test, without allowing the water level to reach the position of the main water strike. To determine the optimum abstraction rate, the pump test contractor will perform a short calibration test.

  - If the position of the main water strike or fracture is not known, a revised minimum one-hour step drawdown test will be performed. No equal time steps are required and the pumping rates can be increased at any time during the test. A flattening of the water level usually indicates the positions of the fractures.
  - Next a constant rate pumping test must be performed. The abstraction rate should not allow the water level to reach the position of the main water strike.
  - The duration of the constant rate pumping test must be long enough to ensure that interpretable drawdown curves are obtained at the observation points (observation boreholes). The minimum proposed duration of pumping should be approximately 8 hours (this is a guideline value – the geohydrologist must decide on the duration of the test). If the impact of inner boundaries (extent of fracture, matrix) or outer boundaries (no-flow or recharge boundaries) is to be estimated, the duration of the constant rate pumping test should be several days.
  - Measure the recovery inside the abstraction borehole until the water level has recovered to 95 percent of the original static water level. A recovery test is an independent test without any external interferences and it can yield important information.

- If the objective of the pumping test is to determine the long-term sustainable yield for a specific borehole, the pumping test contractor will proceed as follows:
  - As mentioned before, a slug test will be performed to give a first estimate of the possible maximum yield for the borehole.
  - To perform a proper constant rate pumping test, the correct pumping rate should be chosen. In this case, the pumping rate should allow for as much drawdown as possible during the constant rate test, ensuring that the water level will reach a position below the main fracture. To determine the optimum abstraction rate, the pump test contractor will perform a short calibration test.
  - If the position of the main water strike or fracture is not known, a revised minimum one-hour step drawdown test will be performed. No equal time steps are required and the pumping rates can be increased at any time during the test. The main objective of this test is to identify fracture positions and to choose a suitable rate for the constant rate pumping test. A flattening of the water level usually indicates the positions of the fractures.
  - The pump test contractor will now proceed with a short duration constant rate test. The objective is to drop the water level to the position of the main fracture within 8 hours. If this position is reached between 2 and 8 hours (guideline duration – the Geohydrologist must decide on the duration), measuring of the recovery phase could start. If not, a higher pumping rate must be chosen for the follow-up constant rate pumping test. Many times it is impossible to lower the water level to the main water strike due to the limitations of the pump in a 160 - 165 mm drilled borehole. In this case the end drawdown level must be used as available drawdown.
  - Measure the recovery inside the abstraction borehole until the water level has recovered to 95 percent of the original static water level. A recovery test is an independent test without any external interferences and it can yield important information.

- Proper water-level measurements should be taken in both the abstraction and observation boreholes during testing. These measurements should be recorded on the prescribed data sheets and the information should be handed to the geohydrologist on completion of the tests. The discharge rate from the pump should also be measured regularly.
- Additional information such as other pumping activities or abstractions during testing, irrigation activities prior to testing, change in water colour and temperature as well as visible boundaries, should be recorded. As much information as possible should be supplied by the pump test contractor.
- If specified in the contract, the pump test contractor must take water samples. The samples should be taken close to the end of testing and the samples should be submitted to the geohydrologist.

2.3.3.2 Geohydrologist

- The geohydrologist can visit the test site while the contractor is busy with the testing. Control water-level measurements against time as well as discharge measurements from the pump can be taken.
- The pumping test contractor must consult the geohydrologist on important issues such as the duration and abstraction rate for the constant rate pumping test.
- The positions of the main fractures can be verified once the modified step drawdown test had been performed. This should be done in conjunction with the geohydrologist.

2.3.4 After the pumping test had been performed

2.3.4.1 Pump test contractor

- The pump testing and water-level measuring equipment must be removed from the boreholes.
- It is the responsibility of the pump test contractor to ensure that the equipment installed on the boreholes prior to testing be reinstalled to the same condition as before removal.
- The terrain should be properly cleaned and it should be left in the same condition as during the start of the test.
- The information gathered during the pumping test should be submitted, with a detailed account for the work done, to the geohydrologist.

2.3.4.2 Geohydrologist

- The geohydrologist must obtain the results of the different pumping tests from the pumping test contractor.
- The raw data should be checked, edited and prepared before the tests can be analysed.
- If the objective of the pumping test is to obtain parameter values for the aquifer the rules and steps are given in Section 3.2.
- If the objective of the pumping test is to estimate a long-term sustainable yield for a specific borehole the rules and steps are given in Section 3.3.

CHAPTER 3

GUIDELINE FOR PUMPING TEST ANALYSIS IN FRACTURED AQUIFERS

3.1 GENERAL

As stated in Chapter 1, the two main objectives of pumping test analysis are:

- **Estimation of aquifer parameters.**
- **Estimation of the sustainable yield of a borehole.**

The principles for analysing pumping tests depend on the objectives.

The main goal of this chapter is to suggest general guidelines for the practical geohydrogist on how to perform and analyse a pumping test considering the two objectives. Every borehole in a fractured-rock aquifer reacts differently, and therefore it is of no use to give one general recipe on how to conduct and analyse pumping test data. In the end, the expert analysing the test is totally responsible for the way in which the test is conducted and analysed. He or she must use all their practical experience and knowledge to come up with an answer. The aim is also to demonstrate the **limitations** of analytical analysis methods. Usually, the limitations are due to the assumptions underlying the theory of the method. In the rural areas of SA, pumping tests are performed on a single well (due to the costs involved in drilling observation boreholes). Single well tests have limitations, such as that no accurate S-value can be estimated.

At the end of the Chapter, a few typical examples will be discussed.

To analyse a pumping test correctly, the geohydrologist should be able to identify the following important characteristics:

- The correct geological conceptual model (for use of the correct analytical model).
- Inner boundary conditions (i.e. well bore storage effects, well bore skin, fracture skin and the lateral extent of the fracture or fracture zone).
- Outer boundary conditions (i.e. especially no-flow boundaries but also fix head boundaries).
- Characteristic flow regimes (the choice of the correct part of the curve to be fitted by looking at the flow dimension that prevails during that specific part of the test).

In the following sections, some guidelines and principles are given on how to perform and analyse hydraulic tests in fractured aquifers for the above-mentioned goals and characteristics to be achieved.

3.2 PARAMETER ESTIMATION

If the objective of the pumping test is to estimate aquifer parameters that are to be used in e.g. a numerical management model, the constant rate test is most important and is set as **minimum requirement** for parameter estimation. Although a slug test and step drawdown (or multirate) test can also be conducted, it is not of much practical value. If the interest is setting up a 3D numerical model, a number of piezometers must be installed (to measure pressure heads and vertical K-values of each layer). One of the most important factors of a constant rate test in this case is selecting the abstraction rate during the test. The yield must be chosen in such a way that no main water-yielding structures will be dewatered during the test.

Minimum requirement: Constant Rate Test of minimum 8 hours

Note: the minimum duration of 8 hours is a guideline value.

3.2.1 Steps and Guidelines for Parameter Estimation

General guidelines are listed in Fig. 1.1, Chapter 1:

- If no information is available on the maximum yield of the abstraction borehole, a slug test can be performed. The time to achieve a 90% recovery in the water level can then be used to estimate a maximum yield for the borehole (Vivier *et al*., 1995):

  $$Q\ (L/h) = 117155t^{-0.824} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(3.1)$$

  where t is the time to achieve a 90% recovery of the water level in seconds and Q is the abstraction rate in litre/hour. The relationship was derived for boreholes with a 160 mm diameter and is valid for all aquifer types.
- To locate the main water strike and choose a rate that will not allow the water level to reach a fracture position, a minimum **one-hour step drawdown test** is suggested (the duration of one hour is a general guideline and could be changed by the geohydrogist). This suggested test is slightly different from the usual step drawdown test. Equal time increments are normally selected, but in the proposed test, this is not important. Start with a very low rate and increase the abstraction rate after any time (e.g. 5 minutes, 15 minutes, 40 minutes, 50 minutes). A flattening of water level will usually occur at the position of a fracture.
- For the **constant discharge test, which is set as a minimum requirement**, choose an abstraction rate so that the drawdown in the abstraction borehole will not reach the position of a fracture (if the position of a fracture is reached, the conditions could change from confined (semi-confined) to unconfined which makes analysis of the data extremely difficult).
- The duration of the test must be **long enough** to ensure that interpretable drawdown curves are obtained at the observation points (observation boreholes and piezometers). The minimum duration of pumping is approximately 8 hours (guideline duration that could be changed). If the impact of inner (extent of fracture, matrix) or outer boundaries (no-flow or recharge boundaries) is to be estimated, the duration of the constant rate test should be several days.

- Diagnostic plots (e.g. log-log and derivatives) can assist when identifying the different characteristic flow regimes (e.g. WBS, linear, bilinear, semi-radial flow as well as boundary conditions – see Chapter 2, Part B).
- Construct the most suitable **conceptual model** for the aquifer considering the above and the geology (it is not always possible to identify the type of flow. Matching different models with the data could shed more light on the problem).
- Select the **most suitable analytical model** (e.g. double porosity, single fracture of infinite or finite conductivity) and use the **TPA program** (including skin factors and boundary conditions) to fit the measured drawdown and recovery data. Time drawdown data associated with flow period representative of typical fractured behaviour for the aquifer can only be analysed using one of the analytical models discussed in Chapter 2, Part B.
- If the fractured aquifer reacts as a homogeneous porous aquifer, the Theis or Cooper-Jacob methods can be used to estimate parameters. Remember that the Theis method is only applicable in the case of radial acting flow (flow dimension = 2; i.e. where the derivative is a horizontal line parallel to the time axis. This is usually at late times of the pumping test, if a boundary was not reached). The estimated T- and S-values represent the entire aquifer without differentiating between fracture and matrix (i.e. some average value for the formation).
- **No unique estimation of hydraulic parameters is possible from hydraulic tests without knowledge of the flow geometry (i.e. flow dimension).**
- For more accurate fits, a 3D numerical model must be used (see e.g. Chiang and Riemann, 2001). The numerical models are not without problems and the solutions will depend on the conceptual model constructed for the aquifer.
- Measurements during the recovery phase must never be omitted and should be analysed independently. Application of the Cooper-Jacob or Theis method to the radial acting flow phase of the recovery data, gives the T-value of the formation.
- As a first estimate of the T-value of the formation, the Logan equation can be used: T (m^2/d)=1.22Q/s , where Q = abstraction rate in m^3/d and s is the drawdown at the end of the test. A qualified guess of the T-value can also be obtained if the maximum yield of the borehole is known [T (m^2/d) = 10*Q, where Q is measured in L/s].

After analysing pumping tests in fractured-rock aquifers with a numerical 3D model, Chiang and Riemann (2001) made the following suggestions to estimate the aquifer parameters with the Cooper-Jacob or Theis methods:

For aquifers with a **single horizontal bedding-plane fracture**:

- To estimate the **transmissivity of the matrix**, the Cooper-Jacob or Theis method can be used, when the assumptions for applying these methods are valid. That means the drawdown curve must show radial acting flow and there are no boundary effects.
- To estimate the **transmissivity of the fracture**, the drawdown-distance method (known as Cooper-Jacob II) can be used. At least 2 observation boreholes intersecting the fracture are required in addition to the abstraction borehole and early drawdown data must be used.
- To estimate the **storage coefficient of the fracture**, the Cooper-Jacob or Theis method can be used. It requires an observation borehole intersecting

the fracture far away from the abstraction borehole. The problem is that the definition of <u>'far away'</u> is unclear.

- To estimate the **storage coefficient of the matrix:**
  1. The Cooper-Jacob or Theis method: Both methods require an observation borehole intersecting the fracture close to the abstraction borehole. The problem is that the definition of <u>'close'</u> is unclear. These methods can also be applied by using the effective borehole radius of the abstraction borehole itself. The effective borehole radius must be calculated from the early data of the hydraulic test.
  2. For the parameters of the matrix **(T, K_v and S**), the Gringarten method can be used, but it is non-unique and difficult to fit the data without prior information. If the parameters T matrix, S matrix and half-length of the fracture are estimated with other analytical methods, it is possible to obtain a unique fit of the drawdown data.

In the case of a **vertical fracture**, situated along a dyke, the comparison of the results from the numerical model with the analytical methods shows the relationship between the different aquifer features. To use analytical methods for parameter estimating in a case of a vertical fracture is more complicated than for a horizontal fracture, because more than one aquifer system are involved.

- Applying the Cooper-Jacob or Theis method to the hydraulic test data from the abstraction borehole yields a geometric mean of the transmissivity of all aquifer systems.
- Applying the Cooper-Jacob II method to two observations boreholes drilled in the same vertical fracture, yields the T-value of the fracture (if the early distance-drawdown data are used).
- Applying the Cooper-Jacob or Theis method to the hydraulic test data from observation boreholes in the fracture (during the radial acting flow phase) yields the T-value of the matrix. Due to its distance dependency, the storage coefficient of the fracture cannot be obtained with analytical techniques.
- An analytical method for an accurate estimate of the storage coefficient of the matrix could not be determined. An observation borehole drilled in the same vertical fracture 'close' to the abstraction borehole, will give the order of the S-value of the matrix if analysed with the Cooper-Jacob method.

3.2.2 Pitfalls and Limitations

As discussed in Chapter 2, Part B, most of the analytical models have inherent problems and limitations. Therefore they have to be used with caution. Some important limitations, which are valid for all or almost all methods, are listed below:

- All analytical models for fractured-rock aquifers assume confined conditions (a very strict condition) which is usually violated in pumping tests in SA fractured aquifers. Mostly the main fracture zone (horizontal or vertical) is connected with the surface or a phreatic aquifer above through smaller perpendicular fractures, which implies semi-confined or unconfined conditions in the fracture zone.

- It is important to make sure of the assumptions underlying each analytical model before applying it to the time drawdown or time recovery data.
- When a fracture is dewatered, the aquifer system at that point has changed from confined (or semi-confined) to unconfined, resulting in the T-value decreasing (the slope in the diagnostic plots before and after reaching the fracture position is normally not the same).
- The T-value estimated with any analytical technique for an observation borehole situated in the tight part of the matrix formation or which is badly connected to the fracture, will usually yield a too high T-value. Here a correct estimation is only possible when using a 3D numerical model.
- The **estimated S-value will show distance dependency** in single fracture or double porosity aquifers if analysed with an analytical model. If a single fracture model is used, the fracture half-length must replace the observation distance. For an abstraction borehole situated in a double porosity aquifer, the effective borehole radius must be used as observation distance.
- The estimation of a reliable S-value with an analytical model is currently still problematic. The use of a numerical model to estimate an accurate S-value is the best option (if the correct conceptual model is used).
- If the T-(or K-) value of a matrix is very small (orders smaller than that of the fracture), the drawdown in matrix piezometers will show a delayed response (i.e. after abstraction has ceased, the hydraulic level will still show a drawdown). This behaviour can only be seen if the fracture has a limited extent (i.e. acting like a closed boundary). Analysing this type of time drawdown data is not possible using an analytical method. A 3D numerical model must be used.
- The non-uniqueness of the double porosity analytical model seriously restricts the practical applications of these models when interpreting pumping test data for fractured-rock aquifers. Heterogeneous aquifers in which the layers have highly contrasting flow characteristics will react as a double porosity aquifer.
- The two-dimensional numerical model, RPTSOLV, can be used to estimate the S-value.
- Whether outer boundaries will have an influence on the time drawdown behaviour depends on the radius of influence. As a first estimate of the radius of influence of a borehole, the following equation can be used:

$$R_i = 1.5\sqrt{\frac{Tt}{S}} \qquad (3.2)$$

where T is the transmissivity (m^2/d), t is the time (d) and S is the storativity.

Equation (3.2) can determine if a no-flow boundary will have an influence on the time drawdown data. If a no-flow boundary intersects the same fracture as the abstraction borehole, the time drawdown data will show the influence of the no-flow boundary. The reason for this is the T-value of the fracture is usually very high (in the order of hundreds or thousands) while the storage coefficient is very small (e.g. in the order of 10^{-5}). If the no-flow boundary does not intersect the same fracture as the abstraction borehole, its influence will normally not be seen in pumping test as the drawdown wave propagates with the T-value of the matrix (which is much smaller than that of the fracture) and the S-value of the matrix which is very high (usually in the order of 10^{-3}). The radius of influence in an aquifer with a matrix T-value of 5 m^2/d and S=0.005 is 82 m, after a time of 3 days of pumping. This

implies that a no-flow boundary situated 100 m away from the abstraction borehole will not have an influence on the time drawdown data after only three days of pumping.
An upward trend in the time drawdown data after a certain time is normally interpreted as the influence of a no-flow boundary, and this can be incorrect. This is because the end of the fracture zone was attained during the pumping test and the steeper drawdown is due to the lower T-value of the matrix.

If parameter estimation is to be used for pollution management purposes, it is important to conduct the pumping test such that the parameters for the fracture and matrix can be calculated. For example, the vertical movement of pollution is controlled mainly by the vertical K-value and the pressure gradient in the vertical direction. The only way to measure this pressure gradient is by installing piezometers at different depths in a borehole, and to perform a pumping test during which the hydraulic heads are measured in the piezometers. The data can only be analysed using a 3D numerical model, as no analytical method can estimate the vertical K-value accurately.

3.3 SUSTAINABLE YIELD ESTIMATION FOR A SINGLE BOREHOLE

If the objective of the pumping test is to estimate the sustainable yield of a single borehole, it is not necessary to use different analytical or numerical models to estimate aquifer parameters. According to the definition of sustainable yield, it is only necessary to obtain the relationship between the abstraction rate and drawdown in the borehole. Therefore as **minimum requirement** for estimating the sustainable yield, a **minimum** of one constant rate test must be conducted, stressing the aquifer. A step drawdown test with the minimum duration of one hour is set as **minimum requirement**. To get prior information, a slug test can also be performed. However, it is normally not of much practical value. One of the most important factors is selecting an appropriate abstraction rate, when performing a constant rate test. The yield must be chosen so that the main water strike will be reached during the constant rate test.

Minimum requirements:	**Step Drawdown Test of minimum 1 hour**
	Constant Rate Test to stress the aquifer
	FC-Program for Analysis

Note: the minimum duration of one hour is a guideline value.

3.3.1 Steps and Guidelines for Sustainable Yield Estimation

The sustainable yield is defined as the discharge rate that will not cause the water level in the borehole to drop below a prescribed limit (usually the position of a major water strike). To estimate the sustainable yield for a single borehole, the following general guidelines should be followed (as listed in Fig. 1.2, Chapter 1):

- It is important to note that one cannot select the duration of the constant rate test beforehand. It is dangerous and not cost-effective to decide beforehand that a test, of say 72 hours, is required. The minimum duration of the test must be at least 2 hours but preferably no longer than 8 hours (guideline duration). The idea is thus to choose an abstraction rate for which all the important characteristics can be seen within an 8-hour period.
- If there is no blow yield information for the borehole, use a slug test (Eq. 3.1) to obtain a first estimate of the yield. A calibration test could be performed, but it is not set as a minimum requirement
- The conductance of a minimum **one-hour step drawdown test** is set as a **minimum requirement** (see point under 3.2 above). As previously discussed under Section 3.2, there is no reason for the selected time increments to be constant. The main objective of the test is to identify fracture positions and to choose a suitable rate for the constant rate test. If a non-linear well loss coefficient has to be estimated, the Helweg method (Helweg, 1994) can be used. The FC-program also uses the data of this test to estimate a sustainable yield for the borehole (non-linear FC-method – see Part B, Chapter 4).
- The only way that an abstraction borehole can dry up is if the water level in the borehole drops below the main water strike. It is therefore important to stress the aquifer during the constant rate test, ensuring that the water level reaches the main water strike after some time (this time must not be too short, preferably after more than 2 hours).

- For sustainable yield estimations, **at least** one **short duration constant rate test** is proposed as **minimum requirement**. The objective is to drop the water level to the position of the main fracture within 8 hours. If this position is reached between 2 and 8 hours, the measurement of the recovery phase could start. If not, a higher rate must be chosen for the following constant rate test.
- It is frequently impossible to lower the water level to the main water strike due to the limitations of the pump in a 160 mm drilled borehole. Using this end drawdown value as available drawdown will give a conservative value for the "safe" yield of the borehole. A better option is to use the geometric mean of the end drawdown and the distance to the main water strike as available drawdown (see Section 3.3.3).
- It is very important that a water sample be analysed to see if the water is fit for human consumption (the FC-program contains a sheet for chemistry input and dividing the water into one of the five different classes according to the DWA&F classification system).
- For estimating the sustainable yield of the borehole, the **minimum requirement** (for DWA&F) is using the FC-program.
- The FC-program will estimate the sustainable yield of the borehole for 24 hours per day by using different methods. If the owner of the borehole only wants to abstract water, for say, 8 hours per day, the FC-program will estimate a rate according to the reduced pumping hours per day.
- The main objective is to recommend an abstraction rate for the borehole so that the water level in the borehole does not reach a specified position after a long time of abstraction (minimum of 1 year and maximum of 5 years for arid regions) without taking recharge into account.
- The distances and production rates of other boreholes in the area must be supplied to the FC-program, which then estimates a sustainable yield by taking these influences into account.
- After estimating the sustainable yield for a large number of boreholes in fractured-rock aquifers, it was found that, as a first approximation, the sustainable yield is usually in the order of 20 to 25% of the blow yield of the borehole if the blow yield is less than 20 L/s. For blow yields higher than 20 L/s, the estimated sustainable yields were in the order of 15 to 20% of the blow yield.
- **Monitoring** the water level during the operation phase is of the utmost importance because of the uncertainty in the recommendation of the production abstraction rate.

3.3.2 Pitfalls and Limitations

There are some limitations when using analytical methods to estimate the sustainable yield:

- The sustainable yield estimate is **non-unique** and will depend on the abstraction rate during the constant rate test (the higher the rate the lower the sustainable yield, and vice versa). Also remember that the relationship between drawdown and abstraction rate is usually non-linear. This is due to the fact that fractures are dewatered faster at the higher rate and that the effective T-value will be smaller.
- If the drawdown curve shows an increasing slope, it is usually due to (i) dewatering of fractures or (ii) reaching the lateral extent of the fracture with the result that the effective T-value will become smaller and finally converge to the T-value of the matrix.
- It is important to consider the choice of the available drawdown in the abstraction borehole for the estimation of a sustainable abstraction rate for the borehole (see Section below).

- If there are more than one abstraction boreholes in the aquifer system, the sum of the sustainable yields for all boreholes must not be higher than the annual recharge for the area.

3.3.3 Choice of available drawdown

The choice of the available drawdown is one of the most critical parameters for sustainable yield estimations. Guidelines for the choice of the available drawdown is as follow:

- Position of main water strike, if this position was reached during the constant rate test.
- Position where the drawdown graph showes a sharp increase.
- If the position of the main water strike was not reached during the test, the best option in most cases would be to take the **geometric mean** of the end drawdown value of the test and the distance to the main water strike (measured from the rest water level) as available drawdown. If the end drawdown is used as available drawdown, a conservative sustainable yield will be estimated, while using the distance to the main water strike as available drawdown could result in an over-estimate of the sustainable yield (especially in the case of a deep water strike). An easy way to estimate the geometric mean of two values is to take the square root of the product of the two values.
- Position of the first water strike if there is a possibility of clogging [e.g. due to elevated iron levels (iron bacteria and biofouling), calcite depositing, etc.] and also permanent deformation in especially weak sandstone formations.
- Also consider the recovery data to decide if the borehole has recovered completely. If the recovery data show a horizontal flattening at a late time, it indicates that the fracture system acts like a semi-closed boundary (i.e. very low formation T). If the recovery has not reached the rest water level (RWL) after an equivalent time than the duration of the pumping, the available drawdown must be corrected with the difference (e.g. if the RWL = 24 m and the recovery water level = 22 m after an equivalent recovery time as the pumping test, 2 m must be subtracted from the available drawdown value).
- For dolomitic aquifers an available drawdown of 5 m would normally be a good choice (sinkholes could form if the drawdown is more).

The FC-program recommends a value for the available drawdown, but the user has the option to overwrite this value.

3.4 IDENTIFICATION OF CHARACTERISTIC FLOW REGIMES

The identification of characteristic flow regimes is very important for both parameter and sustainable yield estimation. The following figures show some of the most important flow characteristics and how to identify them from constant rate pumping tests:

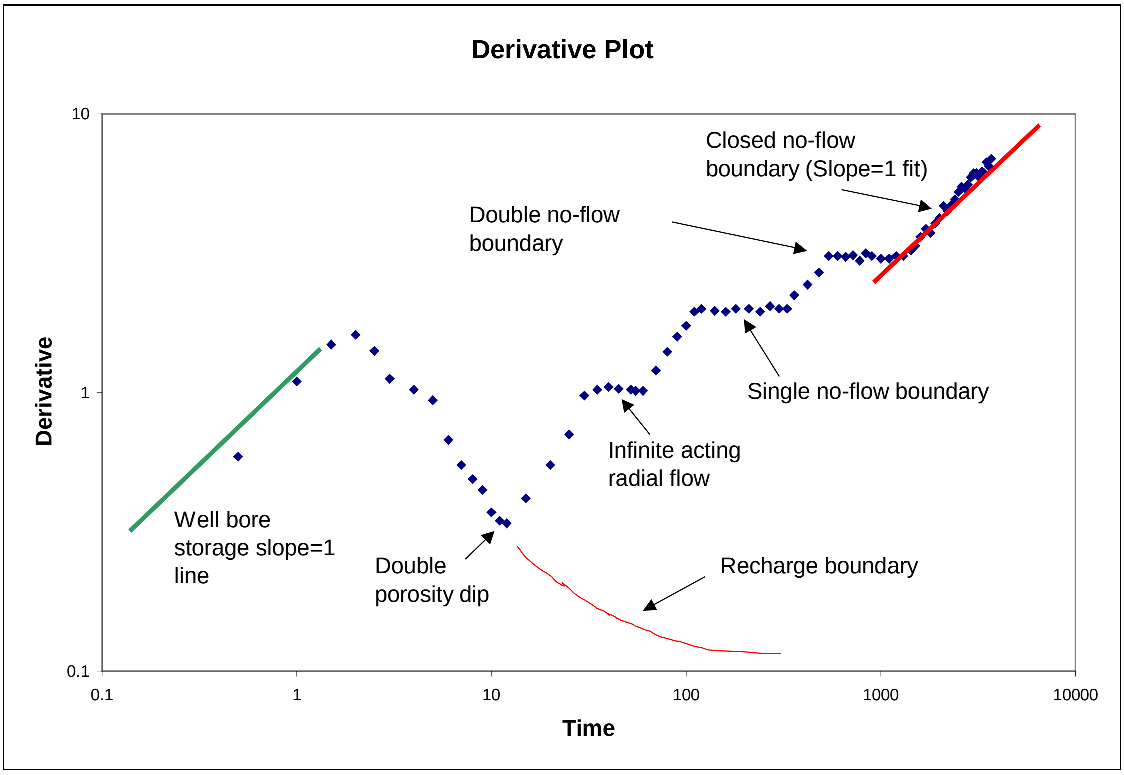


Fig. 3.1 **Flow diagnostics obtainable from the derivative plot (WBS is showing as a slope =1 line fit; radial acting flow = horizontal line; closed no-flow boundary = line with slope 1 at late time; one no-flow boundary = doubling of derivative value at most).**

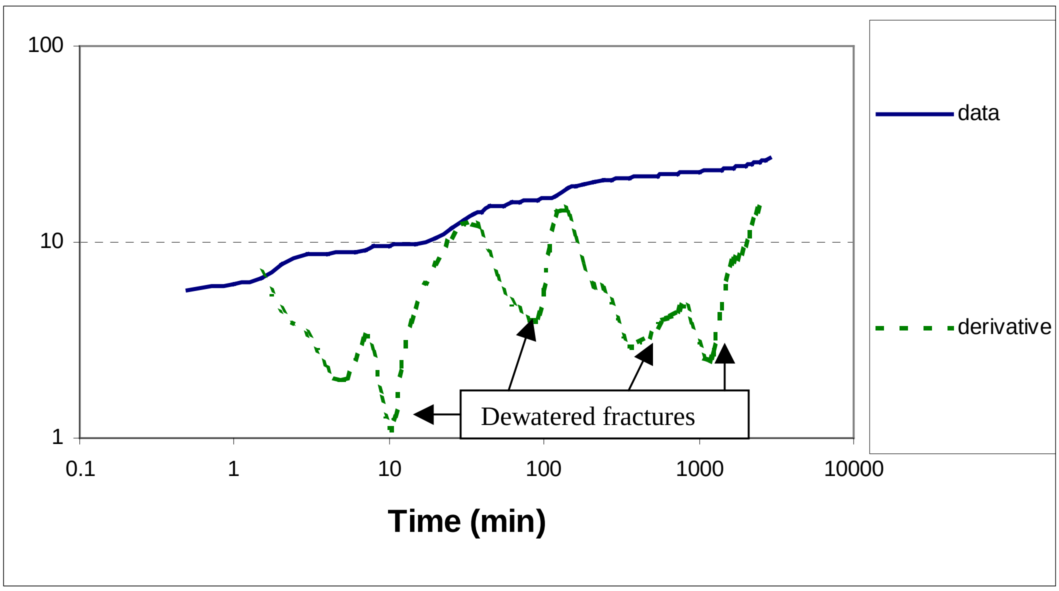


Fig. 3.2 **Derivative plot if fractures were dewatered during the test (at the position of a fracture the derivative is going down and after the fracture has been dewatered, the derivative is going up).**

Linear and bilinear flow can easily be identified from a log-log plot (linear flow = line with slope=0.5 fit and bilinear flow = line with slope =0.25 fit).

3.5 FIELD EXAMPLES

Examples, applying the different analytical methods for parameter estimation to case studies, are given in Chapter 2, Part B. This section will therefore concentrate on estimating the borehole sustainable yield for a few case studies.

3.5.1 Borehole M11 at the Meadhurst Site (Dolerite Dyke Contact)

Borehole M11 was drilled along a dolerite dyke with the main water strike 30 m below the rest water level, which is situated at 22 m below surface. The dolerite dyke was intersected at 28 m below the rest water level. At 30 m below the rest water level a water strike of 4 L/s was encountered in the dolerite. Two constant rate tests were performed on M11 - one at 3 L/s and the other at 7 L/s. Fig. 3.3 shows the pumping test results as well as the information on the water strikes.

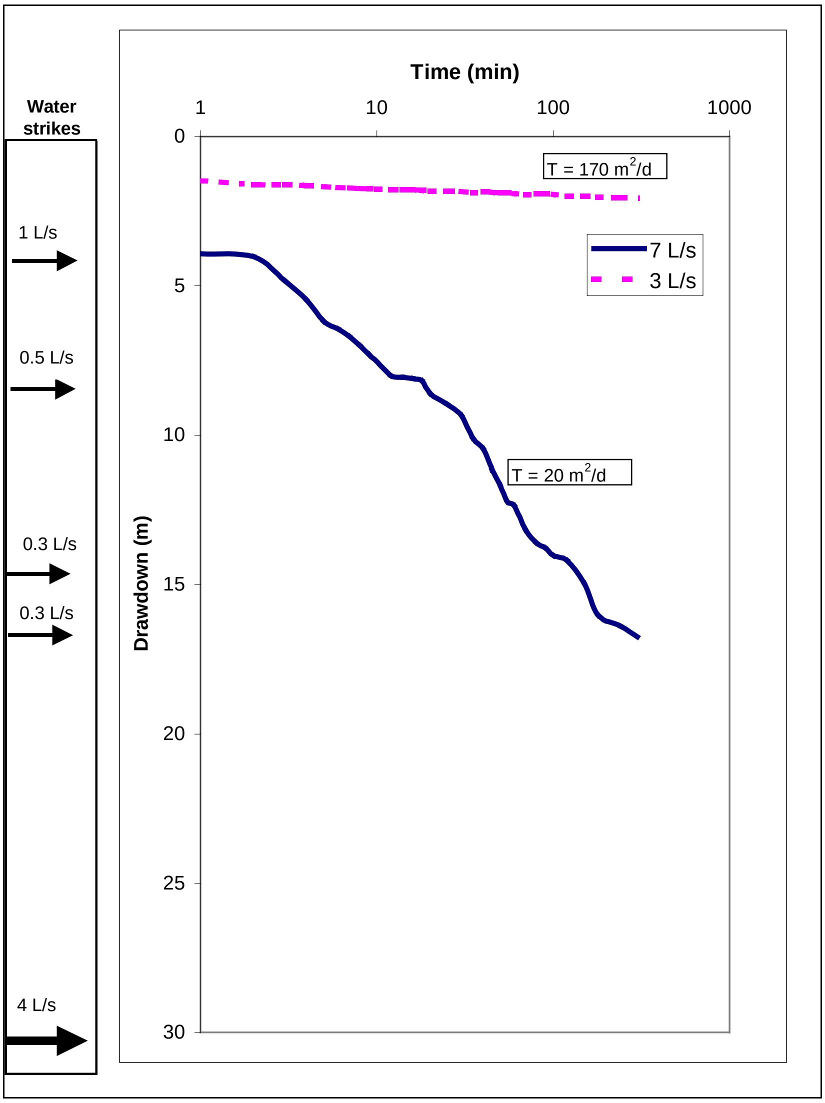


Fig. 3.3 Pumping test results and water strike information of borehole M11 on the Meadhurst Test Site

It is clear from Fig. 3.3 that the results from the two abstraction rates produced different drawdown curves. The estimated T-value with the 3 L/s abstraction rate is 170 m^2/d, while the estimated T-value for the 7 L/s second abstraction rate is 20 m^2/d. At a rate of 7 L/s, the fractures could not sustain the abstraction rate with the result that the estimated T-value of 20 m^2/d is the formation T-value, while the lower rate gives a T-value more representative of the fractures. Very interesting is the fact that a long duration pumping test performed on borehole M1, which is situated 88 m from M11 along the same dyke, also gave a formation T-value of 20 m^2/d.

The distance from the rest water level to the position of the main water strike is 30 m and this position was not reached during both tests. To investigate further, three different available drawdown values were used in estimating the sustainable yield of borehole M11 for both the low and high abstraction rate, namely: (a) the position of the main water strike, (b) the end drawdown of the test and (c) the geometric mean of (a) and (b). Fig. 3.4 shows the estimated sustainable yields with the FC-program for each of these choices as available drawdown.

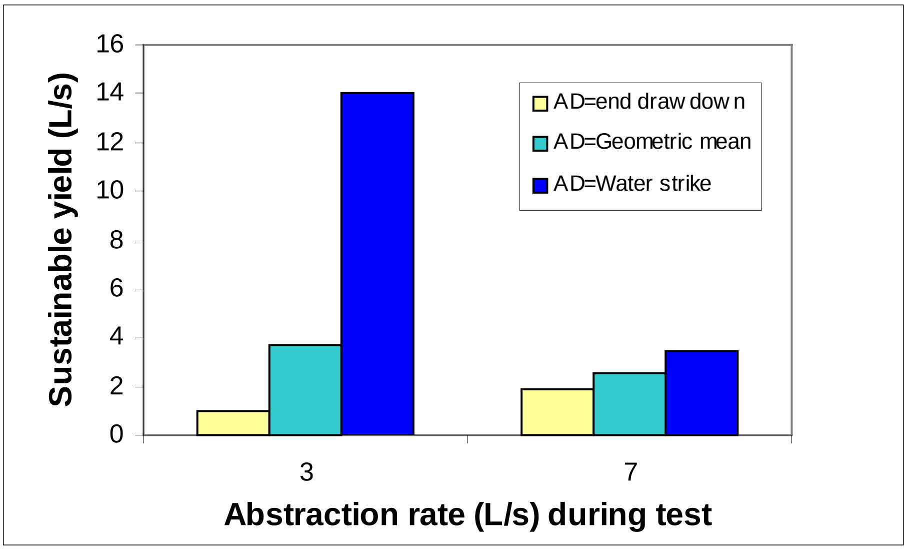


Fig. 3.4 **Estimated sustainable yields for borehole M11 by using different choices for the available drawdown**

The following is clear from Fig. 3.4:

- Using the **end drawdown** of the test as available drawdown gives a minimum "safe" yield for borehole M11. The estimated sustainable yield of 1 L/s from the low rate test is too conservative and is much lower than the yield of 1.9 L/s estimated with the higher rate test. Using the end drawdown during a test as available drawdown, will give the minimum sustainable yield of the borehole and in cases where the end drawdown is still far above the position of the main water strike, the estimated sustainable yield could be far too conservative.
- Using the "true" available drawdown (i.e**. the position of the main water strike**), the estimated sustainable yields are 14 and 3.4 L/s respectively for the low and high rate tests. The 14 L/s obtained from the low rate test, is a gross over-estimate of the sustainable yield of borehole M11. This clearly illustrates the problem encountered when using the position of the main water strike as available drawdown when the end drawdown during a test is still far above the position of the main water strike.

- If the **geometric mean** of the end drawdown of the tests and the position of the main water strike is used as available drawdown, much more realistic values for the sustainable yield are estimated, i.e. 3.7 and 2.5 L/s for the low and high rate tests respectively.

Using the information of the 7 L/s constant rate test, it can be concluded that the sustainable yield of M11 most probably lies between 1.9 and 3.4 L/s. The sustainable yield of 2.5 L/s estimated from the 7 L/s test by using the geometric mean as available drawdown is a good recommended abstraction rate for borehole M11 if no other boreholes are in operation in the vicinity of M11. Actually, two other production boreholes are in operation, and on including their effect, the FC-program estimated the sustainable yield for M11 as 1.93 L/s. The owner of borehole M11 is operating this borehole at a rate of 2.8 L/s for 8 hours per day to irrigate lucerne. Weekly monitoring of this borehole has shown that the recommended abstraction rate is a sustainable one.

This example clearly illustrates the non-uniqueness of the sustainable yield estimate if the position of the main water strike is not reached during the test. The best option in such cases is to use the geometric mean of the end drawdown and the position of main water strike as available drawdown.

3.5.2 Borehole Solo 4 in the Northern Province (fracture in Gneiss)

Fig. 3.5 shows the constant pumping test performed on borehole Solo 4 in the Northern Province at a rate of 6.7 L/s. The only water strike in this borehole was at a depth of 9 m below the rest water level and this position was reached after 3 min., which was too quick. After 400 min. the fracture was dewatered and the water level jumped to the pump inlet. No reliable sustainable yield can be estimated from this constant rate test. In a case where the main fracture position is reached within a very short time, the best option is to stop the constant rate test and repeat it at a lower rate.

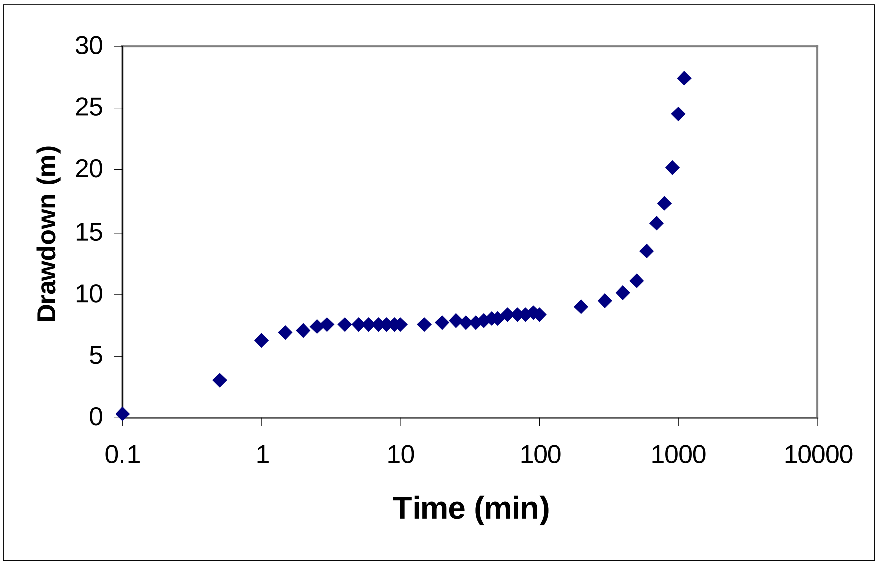


Fig. 3.5 Constant rate test data obtained for the Solo 4 borehole showing that the main fracture position was reached too quickly for a reliable sustainable yield estimate

3.5.3 Borehole B427 drilled in a fault zone

Fig. 3.6 shows the constant rate test data from borehole B427, drilled in an extensional dip slip fault at the margin of a half-graben (borehole sited and pump tested by Karim Sami). The blow yield of the borehole was more than 50 L/s and the abstraction rate during the constant rate test was 23.4 L/s (it was not possible to use a bigger pump due to the radius of the borehole of 160 mm).

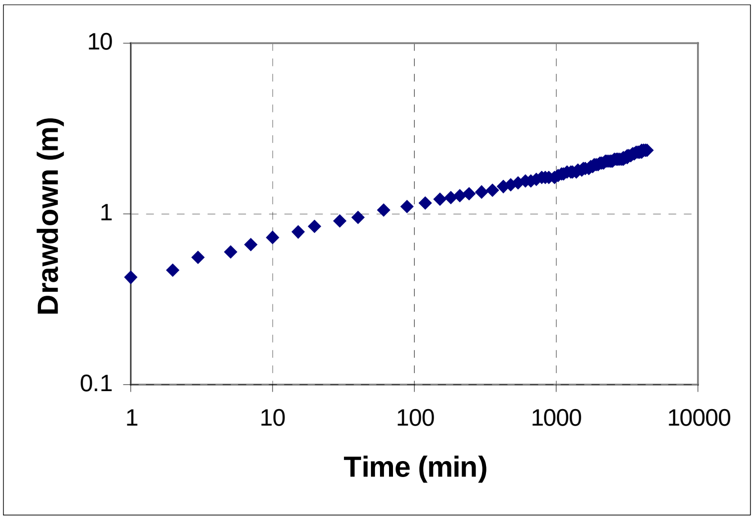


Fig. 3.6 Constant rate pumping tests data of borehole B427 drilled into a fault zone

The drawdown after 3 days of pumping was only 2.35 m (the main water strike was at 44 m below the rest water level). It is very difficult in this case to estimate the sustainable yield of the borehole because of the small drawdown reached at the end of the test. If the end drawdown of 2.35 m is used as available drawdown, the FC-program estimates the sustainable yield of the borehole as 6 L/s, which is probably too conservative. If the geometric mean (i.e. 10 m) of the end drawdown (i.e. 2.35 m) and the true available drawdown (i.e. 44 m) is used, the FC-program obtaines a value of 21 L/s as the sustainable yield for borehole B427. Sami (pers. comm.) recommended a production rate of 10 L/s for this borehole.

This example also illustrates the uncertainty in the sustainable yield estimate if the drawdown reached at the end of the test is much less than the distance to the main water strike.

3.5.4 Examples from Chapter 4, Part B

The following examples are considered (see Chapter 4, Part B):

- M1 at the Meadhurst Site.
- Zonnebloem 1 and Zonnebloem 2 at Middelburg.
- The borehole at Khorixas.

In all the above cases more than one constant rate test was conducted on the borehole. This is to illustrate that a reliable sustainable yield could still be estimated in cases where the main water strike is relatively shallow and if the geometric available drawdown is used as available drawdown in the FC-program.

Table 3.1 lists the borehole information together with the abstraction rates used during the constant rate pumping tests.

Table 3.1 Borehole information, abstraction rates used during the two tests and the end drawdown value of each test

Borehole	Geology	Main water strike (b RWL)	Q1 (L/s)	Q2 (L/s)	End drawdown during tests (m)
M1	Dolerite/ contact	14	1.5	2.0	4.24 and 6.98
Zonnebloem 1	Mudstone	13	14	19	5.9 and 9.98
Zonnebloem 2	Mudstone	13	15	35	4.1 and 12.28
Khorixas	Calcrete	10	1.66	4.16	4.7 and 15.0

By using the geometric mean of the position of the main water strike and the end drawdown value reached during each test, the FC-program estimated the sustainable yields for the boreholes as shown in Fig. 3.7.

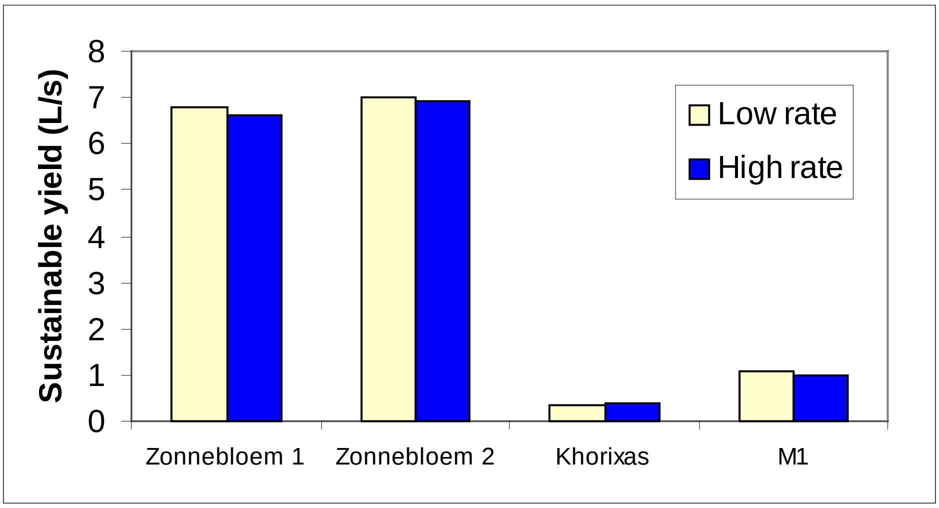


Fig. 3.7 Estimated sustainable yields for the boreholes in Table 3.1 using two abstraction rates for the constant rate tests

All boreholes in these examples have relatively shallow water strikes and for such cases, using the geometric mean value as available drawdown, would usually give a reliable sustainable yield estimate, even for a constant rate test where the position of the main water strike was not reached.

3.3.5 Borehole BK1 at Boschkloof

During January 1998 one of the strongest boreholes in SA was drilled on the farm Boschkloof near Citrusdal (borehole sited by Rowena Hay of Umvoto Africa CC). At a depth of 154 m below surface, a large fracture zone in the underlying quartzitic sandstone formation was intersected. The blow yield of the borehole was in excess of 120 L/s and the temperature of the water was 28 ^{o}C. Fig. 3.8 shows the flushing of the water from this borehole by compressed air during the drilling process.

Fig. 3.8 Groundwater flushing from borehole BK1 at Boschkloof at a rate of more than 120 L/s

A constant rate pumping test at 27 L/s was performed on BK1 and Fig. 3.9 shows the measured drawdown for a period of 4320 minutes.

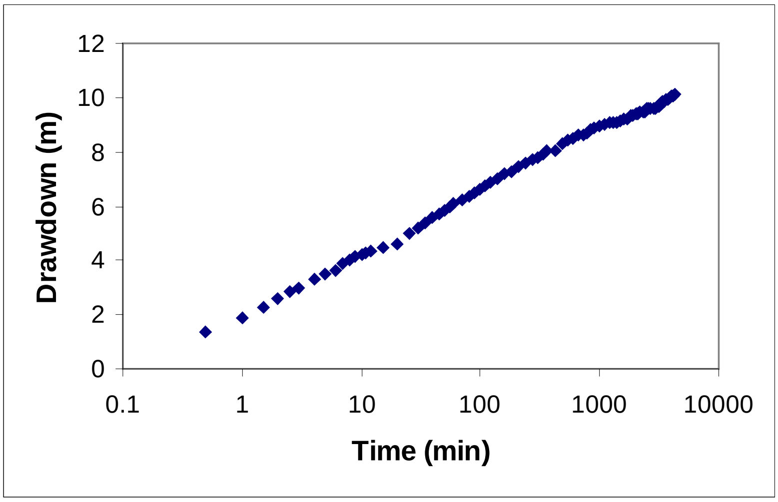


Fig. 3.9 Constant rate pumping tests data of borehole BK1 at Boschkloof

The drawdown at the end of the test was only 10.12 m, which was far above the main water strike of 154 m.

- Using the end drawdown of 10.12 m as available drawdown, the FC-program estimated the sustainable yield of BK1 as 9 L/s, which is by far an under-estimate of the "safe" yield of BK1.
- By using the position of the main water strike (154 m; borehole was artesian) as available drawdown, a far too high sustainable yield of 130 L/s was obtained.
- Using the geometric mean (i.e. 39.5 m) of the end drawdown and 154 m as available drawdown, a sustainable yield of 34 L/s was estimated.

With the information from the constant rate test, the sustainable yield of BK1 most probably lies between 9 and 34 L/s (a value closer to 34 L/s will be more correct as the 9 L/s was obtained with an available drawdown of only 10.12 m, which is much less than the "true" available drawdown of 154 m).

A step drawdown test of four steps was also conducted on BK1 and applying the FC-nonlinear method (see Chapter 4, Part B) to the step data, a relative good fit (see Fig. 3.10) was obtained by using Eq. (4.8) in Chapter 4, Part B, and the estimated sustainable yield was 30 L/s.

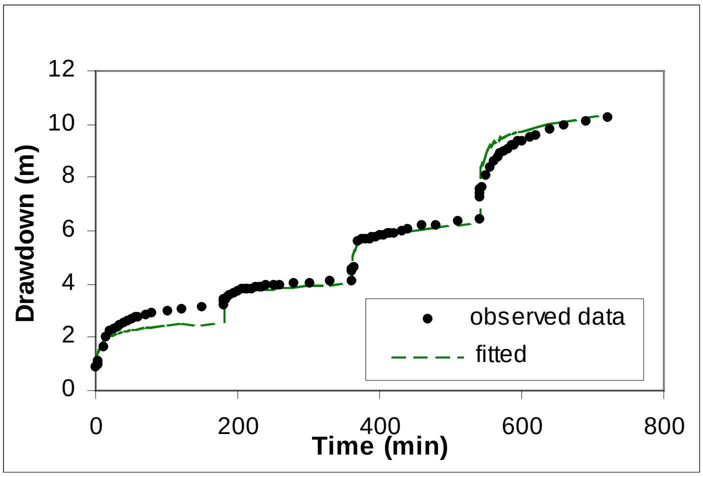


Fig. 3.10 Step drawdown data of borehole BK1 and fit obtained by using (Eq. 4.8) in Chapter 4, Part B, from which a sustainable yield of 30 L/s was estimated

The abstraction rate during the constant rate was 27 L/s and the yields estimated with the FC-program are a bit more than this value. The recovery data also show that the borehole has recovered to the original rest water level within 3 days. The best option in this case would be to use the rate of 27 L/s as production rate. Only monitoring will show if this recommended rate was correct. Four other boreholes were also drilled in the vicinity of BK1, and if it is planned to put them into operation, the influence that BK1 will have on them, must be included when a sustainable yield is estimated for each of them.

3.6 CONCLUDING REMARKS

The most reliable sustainable yield will be estimated if the position of the main water strike was reached during the constant rate test. Using the end drawdown of the constant rate test as available drawdown, a conservative sustainable yield would usually be estimated. If the position of the main water strike was not reached during the test, a good choice would be to use the geometric mean of the end drawdown and position of the main water strike in the case of relative shallow main water strikes. For very deep water strikes, and if the end drawdown during the test was still far above the main water strike, using the geometric mean, could lead to an over-estimation of the sustainable yield.

In the FC-program the user has the choice to obtain an estimate of the sustainable yield of a borehole with the following methods:

- FC-method (Basic, Advanced, Inflection point and non-linear solutions)
- Cooper-Jacob method
- Barker-method

The program also estimates the standard deviation of the estimates obtained with all the methods. In the case of the Advanced FC-method, a risked-based sustainable yield is estimated. To apply the advanced method, the user must know the possible range of the distances to no-flow boundaries and the range of T- and S-values. The program then estimates a 68 or 95 % certainty sustainable yield.

To extrapolate the drawdown measured in an abstraction borehole to a very long period (e.g. 2 years) from data of a pumping test of maximum 3 days remains uncertain. The importance of water-level monitoring during the actual operational phase of the borehole could not be stressed enough. **Monitoring must always be an integral part of aquifer management.**

MANUAL ON PUMPING TEST ANALYSIS IN FRACTURED-ROCK AQUIFERS

Part B

Theoretical Background and Details for Pumping Test Analysis

CHAPTER 1

INTRODUCTION

1.1 SOFTWARE DEVELOPED

Two software packages were developed or enhanced during the current study:

- FC (Flow Characteristic method).
- TPA (Test Pumping Analysis).

TPA is a windows program in DELPHI and was written by Ingo Bardenhagen as part of his Ph.D. study at the Institute for Groundwater Studies, while Gerrit van Tonder, Harald Kunstmann and Yongxin Xu developed the original FC_EXCEL spreadsheet for the Department of Water Affairs and Forestry (SA).

During the current project, the **FC** software was enhanced and includes the following procedures:

- Porous aquifer solutions (Theis, Cooper-Jacob I and II and Hantush methods and also a solution for water-table aquifers).
- Step drawdown and multirate analyses.
- Fractal pumping test analysis (Barker's Generalized Radial Flow Model).
- Slug test analysis (Bouwer and Rice method).
- Estimation of a risk-based sustainable yield of a borehole by using drawdown derivatives, boundary information and error propagation.
- Testing of the suitability of the water according to the Classes used by the DWAF.
- Different diagnostic plots for flow regime identification (e.g. derivatives, second derivatives, log-log (Theis)-plot, lin-log (Cooper-Jacob)-plot, square root of time plot, fourth root of time plot, spherical and recovery plot).
- Delineation of borehole protection zones in fractured aquifers.

The main emphasis of the FC program is to estimate a risk-based sustainable yield for a borehole by using different methods.

TPA was developed with the aim to fit pumping test data in fractured aquifers and include the following fractured aquifer methods:

- Double porosity aquifer (Moench method).
- Solutions for single vertical and horizontal fractures (Gringarten, Kazemi, Warren and Root and Stallman, including uniform flux, finite conductive and infinite conductive fractures as well as boundary conditions and a solution for a dyke aquifer).
- Porous solutions.
- Diagnostic plots.

TPA was specially developed as a curve fitting procedure for pumping tests performed in fractured-rock aquifers to estimate aquifer parameters.

It is well recognised that on a theoretical basis the best method to obtain fractured-rock aquifer parameters is by the use of a 3D-numerical model, like MODFLOW. However, the data required for such a numerical model may not always be available and the application of the model also requires an experience user and the construction of the correct conceptual model for the geological set up. The emphasis of this document will thus be on the application of analytical procedures to analyse pumping test data.

1.2 WAY TO USE PART B

There are mainly two objectives in doing pumping tests:

- Estimation of aquifer parameters.
- Recommendation of a sustainable yield for a borehole.

Fig. 1.1 shows a flow chart of how to use Part B of the document.

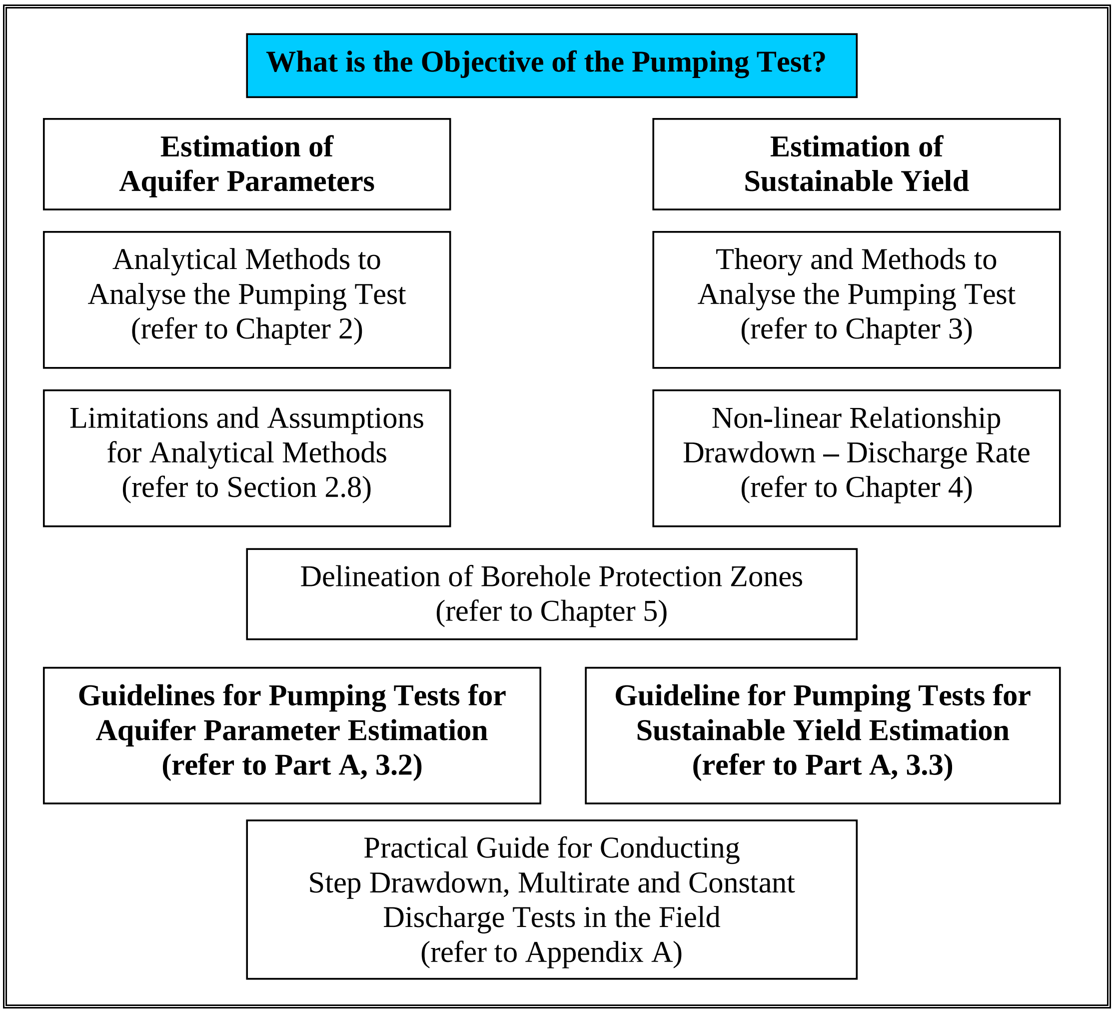


Fig. 1.1 Flow chart for the different sections of Part B

Readers that are interested in all the analytical methods and the different flow diagnostics are referred to Chapter 2. Chapter 3 focuses on the estimation of the sustainable yield of a borehole, while Chapter 4 is devoted to variable rate tests and non-linear relationships between drawdown and abstraction rate. Chapter 5 deals with the delineation of borehole protection zones in fractured aquifers and in Chapter 6 the reader can find recommendations concluded from this manual.

CHAPTER 2

FLOW DIAGNOSTICS AND ANALYTICAL MODELS FOR PUMPING TESTS IN FRACTURED AQUIFERS

2.1 INTRODUCTION

Pumping tests are the most important experiments for the aquifer investigation in the groundwater industry. They are the only method that provides simultaneous information on the hydraulic behaviour of the well, the reservoir and the reservoir boundaries, which are essential for an efficient aquifer and well field management.

The complex situation in fractured aquifers requires a decent understanding of the drawdown behaviour if reliable reservoir information is desired. This can be achieved by a detailed diagnosis of drawdown and recovery data in combination with a conceptual model of the geological set-up. The computer programs FC (Flow Characteristic) and TPA (Test Pumping Analysis) were designed for this purpose. They provide powerful tools for the detailed diagnostic and analysis of the pumping test data, and a simulator that can be used for a forward modelling of test curves in the case of TPA. The simulator offers solutions for primary aquifers like confined, leaky, delayed response and two aquifers including the influences from reservoir boundaries, well bore storage and partial penetration. In the case of fractured aquifers, solutions for double porosity, single vertical and horizontal fractures with finite and infinite conductivity and generalised radial flow, along with the influences of reservoir boundaries, well bore storage and partial penetration are supplied.

This chapter will concentrate only on the special diagnosis and analysis techniques for fractured aquifers because the standard methods, derived for porous media (i.e. Theis and Cooper-Jacob), cannot be applied correctly in many cases, as discussed in Section 2.8. It will demonstrate the diagnostic tools and methodologies available to identify the different flow phases that can occur during a pumping test in a fractured environment. Theoretical examples and field data will be presented to illustrate and discuss the limits of each derived solution.

2.2 FRACTURE NETWORK PROPERTIES

Characteristic for fractured aquifers is the fact that most of the water flows along fractures. Those fractures are usually embedded in porous matrix blocks (sandstone) or micro fissured blocks (quartzite), which are low permeable compared to the fracture conductivity, but capable to store water in the uncountable pores or micro fractures. In extreme cases, the blocks between the fractures are so low permeable (granite) that very little water can be exchanged between fracture network and matrix, which is then called ‘inert’.

If fractures are densely interconnected, they conform a ‘fracture network continuum’ characterised by a large storage capacity that contributes substantially to the volume extracted by a pumped well. Whether a fracture network can be considered as continuum or not is determined by the following three properties:

- Representative elementary volume (REV).
- Fracture connectivity.
- Conductivity contrast between fracture and matrix.

The fracture connectivity describes the interconnection between fractures in a given volume of rock, which is a function of the fracture length and fracture density. Generally, the fracture network continuity of a rock volume increases with increasing fracture length and fracture density (Long and Witherspoon, 1985).

The conductivity contrast between fracture and matrix can diminish or increase the continuous behaviour of a fracture network. Wei *et al.* (1998), by means of numerical modelling, observed linear flow in a well situated in a parallel fracture system, embedded in a matrix with a high conductivity contrast between fracture (K_f) and matrix (K) conductivities ($K_f/K = 10000$). The same fracture distribution with a lower contrast ($K_f/K = 100$) resulted in a long bilinear flow phase followed by a radial flow phase. A similar situation was observed in a perpendicular two-dimensional fracture network with low contrast, whereas using a high contrast, the system behaved a homogeneous media alike.

The storage of a single fracture or a fracture cluster is very limited, which can be demonstrated by the following calculation:

$$V_f = l_f \cdot h_f \cdot b = 2000m \cdot 200m \cdot 0.002m = 800m^3$$

where
V_f = fracture volume [L^3]
l_f = fracture length [L]
h = fracture height [L]
b = fracture aperture [L]

A well located in such a fracture that pumps at a rate of 10 m^3/h would empty it within 80 hours. Under real world conditions, this is usually not the case because the matrix, where the fracture is embedded, is drained by the fracture which, in this instance, acts as a conduit. However, in both extremes the continuum and the single fracture case have very characteristic flow and drawdown behaviour that can be observed during pumping tests and will be presented in the following section.

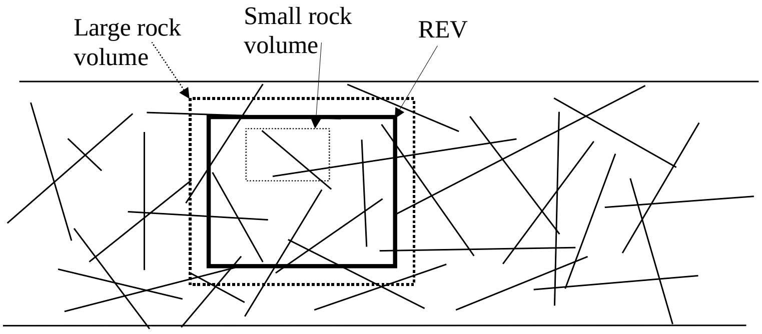


Fig. 2.1 **The representative elementary volume REV of a fractured rock is considered as hydraulically homogeneous (continuously fractured). A volume of rock larger than the REV would maintain the same hydraulic properties, but not a smaller volume**

2.3 GOVERNING EQUATION FOR FLOW IN FRACTURED AQUIFERS

The Poiseuille equation or ‘cubic law’ governs the laminar flow within a single fracture (Witherspoon *et al.*, 1979). This law is a special case of the ‘Darcian Law’, which is written as:

$$Q = K \cdot \frac{\Delta h}{\Delta l} \cdot A \qquad (2.1)$$

where
Q = flow through the area A [L^3T^{-1}]
A = through-flow area [L^2]
Δh = potential or head difference over the length of interest l [L]
Δl = length of interest [L]
K = hydraulic conductivity [LT^{-1}]

The hydraulic conductivity is defined as $K = k\rho g / \mu$, where
ρ = density of the fluid [ML^{-3}]
g = acceleration of the gravity [LT^{-2}]
μ = dynamic viscosity [$ML^{-1}T^{-1}$]
k = permeability [L^2]

In the ‘cubic law’, the hydraulic conductivity is defined as $K = (2b)^2 \rho g / 12\mu$ [L/T] and A = bh [L^2]. Replacing K and A in Eq. (2.1):

$$Q = \frac{b^3 \cdot \rho \cdot g \cdot h}{3 \cdot \mu} \cdot \frac{\Delta h}{\Delta l} \qquad (2.2)$$

where
b = aperture or width of the fracture [L]
h = height of the fracture [L]

Eq. (2.2) represents the Poiseuille equation, which is valid for laminar flow or Reynold numbers smaller than 2300 (Wendland, 1996). It shows that the rate Q is a function of the cube of the fracture aperture, hence the name ‘cubic law’.

Taking into consideration Eq. (2.2), the cone of depression produced by a pumped well at a certain point P(r,z) in a fracture continuum, can be described by the following diffusivity equation in cylindrical co-ordinates (Moench and Ogata, 1984):

$$K \frac{1}{r} \frac{\partial}{\partial r}\left[r \frac{\partial h(r,t)}{\partial r} \right] + K_z \frac{\partial^2 h(r,t)}{\partial z^2} = S_s \frac{\partial h(r,t)}{\partial t} + q_b \qquad (2.3)$$

where
h = hydraulic head [L]
r = distance from P to the well [L], with $r \geq r_w$
r_w = drilled radius [L]
z = vertical position of P [L]
K = conductivity of the continuum fracture network [LT^{-1}]
K_v = vertical conductivity of the fracture network [LT^{-1}]
S_s = specific storage coefficient of the reservoir [L^{-1}]
q_b = additional source function

Eq. (2.3) is valid under the following conditions:
- Negligible change in the gravity acceleration.
- Constant fluid properties.
- Laminar flow.

In a fully penetrating well, the hydraulic head does not vary with depth. Therefore, the second term on the left-hand side in Eq. (2.3) becomes zero and the equation reduces to an ordinary linear inhomogeneous differential equation.

The solutions of Eq. (2.3) that will be discussed in this article were derived by several authors, using either the Laplace transformation or Green's functions under different boundary conditions.

The Laplace transformation L, applied to the hydraulic head function h(t), is often used to solve radial symmetric boundary conditions. It reads

$$L\{h(t)\} = \bar{h}(p) = \int_0^\infty e^{-p \cdot t} \cdot h(t) \cdot dt \qquad (2.4)$$

The advantage of the Laplace transformation lies in the elimination of one of the integration variables, which, in many cases, results in an ordinary arithmetic function. The inversion of this function can be done either analytically or numerically. Mavor and Cinco-Ley (1979) and Moench and Ogata (1984) showed that the Stehfest (1970) algorithm for the numerical inversion of the Laplace transform is extremely fast and usually accurate enough to be used in these cases. The Stehfest algorithm reads:

$$h(t) \approx \left(\frac{\ln 2}{t}\right) \cdot \sum_{i=1}^{N} V_i \cdot \bar{h}(p) \cdot \left[\frac{i \cdot \ln 2}{t}\right] \qquad (2.5)$$

where V_i are weighting factors calculated as

$$V_i = -1^{(\frac{N}{2})+i} \cdot \sum_{k=\frac{i+1}{2}}^{\min(i,\frac{N}{2})} \frac{k^{\frac{N}{2}+1} \cdot (2k)!}{(\frac{N}{2}-k)! \cdot k! \cdot (k-1)! \cdot (i-k)! \cdot (2k-i)!}$$

with
N = even number
i,k = integer values

The advantage of the algorithm lies in the fact that V_i is calculated only once for a given even number N, becoming hence very fast. Stehfest (1970) and Walton (1996) report that the quality of the results decreases with increasing number of N due to rounding errors. For this reason solutions derived in TPA use a range of N from 4 to 26 depending on the time interval calculated.

Green's functions were first applied to boundary flow problems in fractured aquifers by Gringarten and Ramey (1973) and have the advantage that two source functions can be combined by simply multiplication, which is known as the Newman product. Using this technique, Gringarten *et al.* (1974) and Gringarten and Ramey (1974) derived solutions for the drawdown in wells situated in single vertical and horizontal fractures. The drawdown solutions for pumping wells located in vertical fractures with uniform flux and infinite flux are generally analytically derived, whereas in most cases the drawdown in observation wells within the matrix is numerically determined.

2.4 FLOW BEHAVIOUR IN FRACTURED MEDIA

The following flow types can occur during pumping tests in fractured reservoirs (Barker, 1988):

- Linear flow.
- Radial flow.
- Spherical flow.

2.4.1 Linear flow

The name 'linear flow' derives from the way in which the pressure drops along fractures: linear-proportional to the extraction rate. Linear flow is also described as 'parallel flow' (Kruseman and De Ridder, 1991) because of the parallelism between the streamlines.

The typical geological features where linear flow is observed are subvertical fractures, faults, or dykes. The different flow phases that can be distinguished during pumping tests in those features are listed below (Fig 2.2):

- Linear fracture flow is observed when the feature has a finite conductivity and is either embedded in an inert formation (matrix) or in a low conductive formation (Boehmer and Boonstra, 1986; Cinco-Ley and Samaniego, 1981).
- If the matrix is permeable enough, the linear flow in the fracture is superposed by a perpendicular linear flow from the formation to the fracture. This flow situation is described as the 'bilinear flow' (Cinco-Ley and Samaniego, 1978).
- Linear flow from the formation to the fracture in the case of infinite conductive single features with negligible storage (Gringarten *et al.*, 1974).

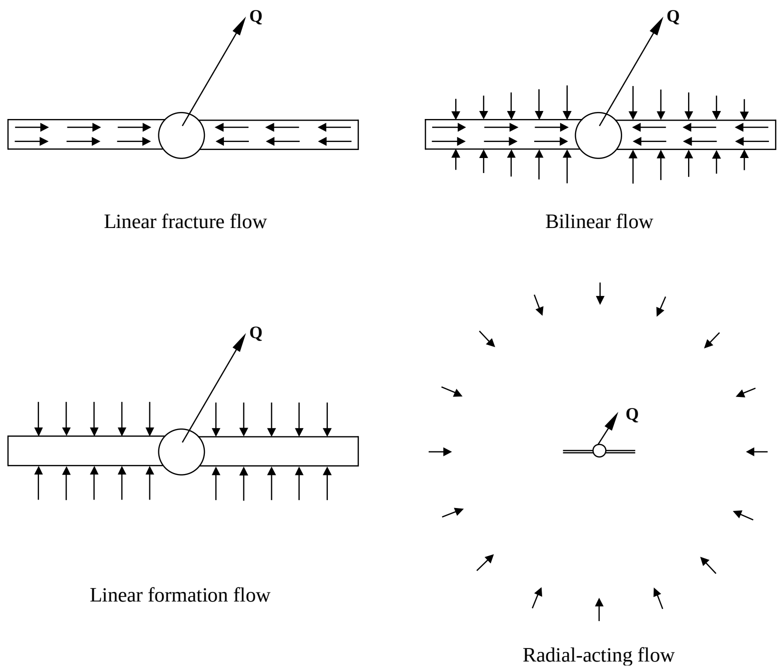


Fig. 2.2 Different flow phases observed in a single fracture of finite extension embedded in an infinite formation (adapted from Horne, 1997)

- A special case of bilinear flow occurs in reservoirs that consist of a continuous fracture network embedded in porous matrix blocks (Fig. 2.3), which is known as double porosity reservoir (Barenblatt *et al.*, 1960) or naturally fractured reservoir (Mavor and Cinco, 1979).

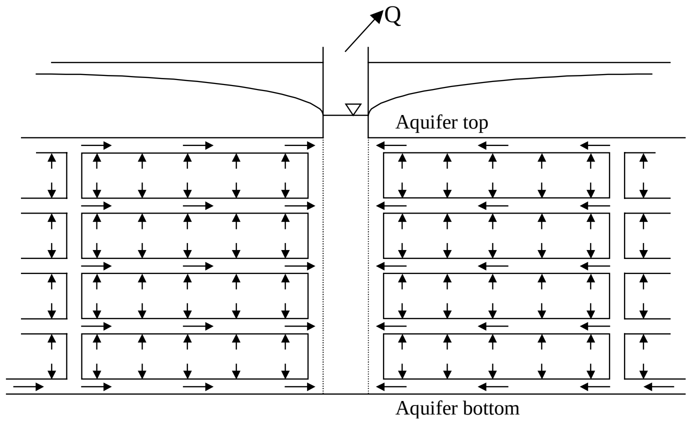


Fig. 2.3 Groundwater flow in an idealised double porosity aquifer

2.4.2 Radial Flow

Radial flow (also known as pseudo-radial flow or radial-acting flow) appears when the cone of depression is approximately circular. It is generally observed in a fully penetrating well (line source) located in homogeneous reservoirs, but also in a well in any fractured reservoir that can be considered as continuum. The start of the radial flow indicates the time at which the fractured reservoir behaves as homogeneous. The distance from the pumped well at which the radial flow starts determines the dimension of the REV, as demonstrated in Fig. 2.4.

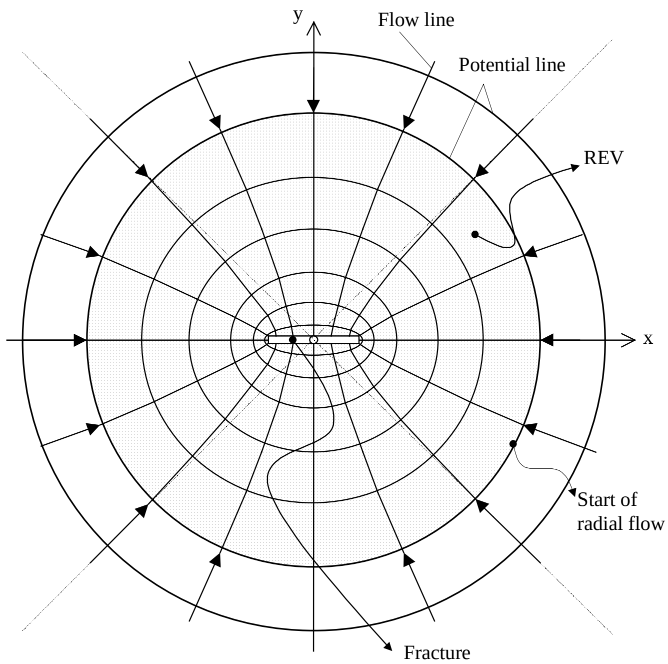


Fig. 2.4 REV for a single vertical fracture with infinite conductivity. An observation point beyond the grey area would show only radial-acting flow behaviour

The characteristic distance or dimension of the REV for a single fracture embedded in an infinite matrix equals 5 times the fracture's half-length x_f (Fig. 2.4). An observation well located outside of the REV will show only radial-acting flow as the characteristic flow behaviour (Fig. 2.5). In instances where the REV coincides with the drilled radius r_w, any observation well will show radial-acting flow. Such observation data can then be analysed with methods usually applied to primary aquifers. For observations within the REV, the influence of the fracture network must be considered.

(a) (b)

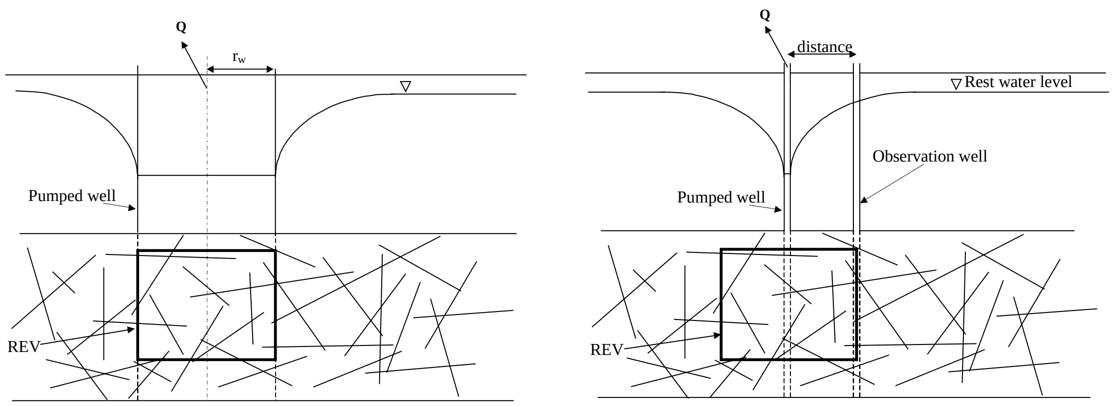


Fig. 2.5 Whether the influences of a fracture network can be observed or not depends on the location of the observation point. Is the REV smaller than the drilled radius (a) or is the observation well distance (b) equal or further apart than the REV, the influence of the fracture network cannot be observed and the drawdown curve will follow a Theis curve. Therefore, in the sketch of the right side only, the observation in the pumped well can show the influence of the fracture network

2.4.3 Spherical Flow

In cases where the extraction source is a point in an isotropic medium, the cone of depression becomes a sphere (Gringarten and Ramey, 1973). In real world, spherical flow will be observed only within small dimensions and over a short time period, because the spherical cone of depression will reach the bottom of the aquifer and the cone will become an ordinary radial flow (Fig. 2.6). Furthermore, due to anisotropy effects in the aquifer the sphere will become an ellipsoid. Therefore, the spherical flow can be considered as a special case of a partial penetrating well in a formation with isotropic conductivity ($K_x = K_z = K_z$ or $K_r = K_v$).

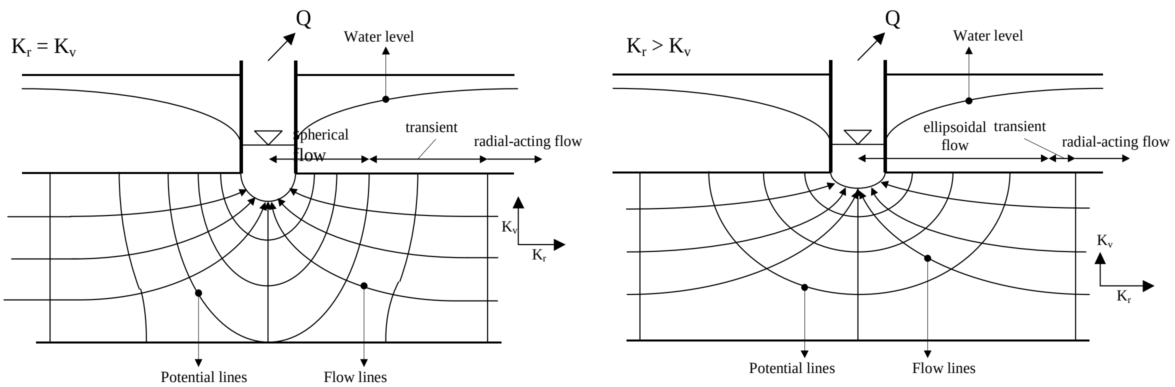


Fig. 2.6 Spherical flow behaviour in a bounded aquifer under isotropic ($K_r = K_v$) and anisotropic ($K_r > K_v$) conditions

2.5 DIAGNOSTIC TOOLS

2.5.1 Straight lines

The flow phases that appear during a pumping test in a fractured aquifer show characteristic straight lines either in a double logarithm (log-log) or in a linear-logarithm (lin-log) plot.

A log-log plot will provide information at early time as follows:

- Well bore storage shows a slope of 1.
- Linear flow shows a slope of 0.5.
- Bilinear flow shows a slope of 0.25 (= flow from fracture and formation).
- Fracture storage shows a slope between 0.5 and 1 in the linear flow case and between 0.25 and 1 in the bilinear flow case.
- On late time, the log-log plot will provide information as follows:
- Two parallel no-flow boundaries show a slope of 0.5 (Ehlig-Economides and Economides, 1985).
- Three equidistant no-flow boundaries show a slope of 0.5.
- Limited reservoir (four closed boundaries) shows a slope of 1.

In cases of a single vertical or horizontal fracture, the log-log plot can also be used to determine linear formation flow (slope 0.5 at intermediate time) and limited extent of the fracture (slope 1 at intermediate time). For more details, see Section 2.7.

A lin-log plot provides the following information:

- Radial-acting flow appears as a straight line.
- One no-flow boundary doubles the slope of the radial-acting flow straight line.
- Two perpendicular no-flow boundaries quadruple the slope of the radial-acting flow straight line.

Only one straight line will be observed when all the boundaries are located at equidistant to the pumping well. If this is not the case, each boundary will increase the slope of the previous straight line reached.

2.5.2 Special plots

Besides the lin-log and log-log plots, the following three additional plots are very useful for the diagnosis of pumping test data in fractured aquifers:

- Linear drawdown versus square root of time.
- Linear drawdown versus fourth square root of time.
- Linear drawdown versus one divided by square root of time.

The first plot is useful for the determination of linear flow behaviour, as the drawdown data will plot on a straight line that starts in the origin of the diagram. In the second plot, the drawdown data of the bilinear flow phase plot on a straight line starting in the origin.

The drawdown data of a spherical flow plot on a straight line that starts in the origin in the third plot.

Cinco-Ley and Samaniego (1981) demonstrated that the first two diagrams are very useful to determine skin effects in drawdown data of wells in single fractures with either linear or bilinear drawdown behaviours. Such skin effects cause an additional drawdown that increases clogging phenomena and, in extreme cases, can even destroy the stimulation effect of drilling in a fracture zone (Economides, 1989).

Cinco-Ley and Samaniego (1981) found that due to skin effects, the early time linear flow data in a log-log plot graphs as an almost horizontal line that develops into the radial-acting phase (bilinear flow would plot similarly). Plotting the same data in a linear drawdown versus square root of time diagram will also show a straight line but shifted downwards from the origin. Both authors stated further that data only from the pumped well do not provide a unique solution for the determination of the skin location, which could be located at the well, between fracture and formation, or at both. Bardenhagen (1999) showed a unique evaluation method for the skin location in a single vertical fault by using the linear drawdown versus square root of time plot for drawdown data of a pumped well and an observation well located in the same fault.

These considerations are also valid in the case of horizontal fractures, as they are only related to the flow regime.

2.5.3 Recovery

The same plots can be applied to recovery data after correction of the measured time. In most practical cases, the quality of the recovery data is better than that of the drawdown, as they are not influenced by fluctuations in the pumping rate. This holds especially for the application of derivatives that are commonly used as diagnosis tools of drawdown phases. On the recovery t/t' plot, a limited closed reservoir will result in a horizontal flattening at a late time.

2.5.4 Curve derivatives

The use of derivatives is of great advantage due to their sensitive reaction to small changes in the drawdown or recovery, while they are independent of skin effects. Usually, the first derivative is plotted as ($\Delta s/\Delta t \cdot t$) (Economides and Nolte, 1989), which provides the following advantages:

- All characteristics of the straight line slopes remain the same.
- Well bore storage shows a line with slope 1 at early times.
- The radial-acting flow phase is plotted as a horizontal line, which eases the identification for the human eye.
- A closed boundary shows a line with slope 1 at late times.
- A dip in the first derivative after the well bore storage is an indicator of a double porosity aquifer.
- At the position of a fracture, the derivative shows a decrease and after the fracture is dewatered, the derivative will go up again.
- A recharge boundary or fixed head boundary is seen on the derivative graph as a strong downward trend at late times.

- While reaching a no-flow boundary, the horizontal line of a radial-flow period will shift to a doubled value.

  Using the second derivative, the following information will be provided:
- A second derivative of 1 shows a closed boundary.
- During radial flow, the second derivative is equal to zero and is therefore not visible in a log-log plot.

Unfortunately, derivatives applied to real data often show noisy results. Smoothing of the derivatives would overcome this problem, but it cannot be ensured that the applied mathematical algorithm would not produce misleading artifacts. The TPA program is using the original derivatives, while the FC program plots smoothed derivatives. Nevertheless, with some experience even noisy derivatives can be interpreted.

2.6 WELL AND RESERVOIR EFFECTS

The following well and reservoir effects can affect the drawdown and recovery data within fractured aquifers:

- Well bore storage.
- Well bore skin.
- Partial penetration skin.
- Fracture skin.
- Pseudo-skin.
- Fracture dewatering.
- Reservoir boundaries.

2.6.1 Well bore storage

Well bore storage effects occur due to changes in the water level or compressibility of the water-well system (Ramey and Agarwal, 1972). These effects are generally important at the beginning of the test but disappear with time (Streltsova, 1988). The dimensionless well bore storage coefficient W_d is defined as (Moench, 1984):

$$W_d = \frac{r_c}{2 \cdot r_w^2 \cdot S} \qquad (2.6)$$

where
rc = casing radius [L], where the water-level change occur
r_w = drilled radius [L]
S = specific storage coefficient of the reservoir [-]

Eq. (2.6) is valid if the compressibility of the water well system is negligible (Moench, 1984).

Immediately after commencement of extraction, all water is pumped from the storage volume of the well, as the gradient within the reservoir is still small; hence the enormous well bore storage effects at the beginning of the test. With time, the gradient within the reservoir increases gradually until all extracted water is provided by the reservoir and consequently the well bore storage effects disappear (Fig. 2.7).

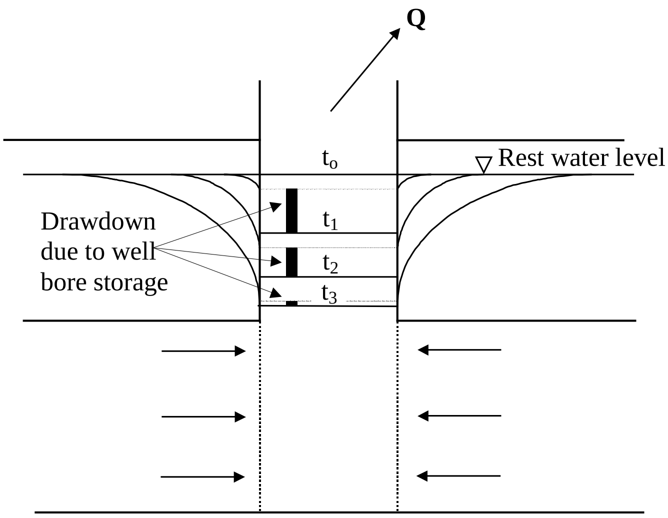


Fig. 2.7 Relationship between gradient changes in the reservoir and well bore storage

Graphed in a log-log plot, the well bore storage effects in a pumped well show a unit slope. In observation wells, this slope decreases with increasing distance of the observation well to the pumped well (Fig. 2.8).

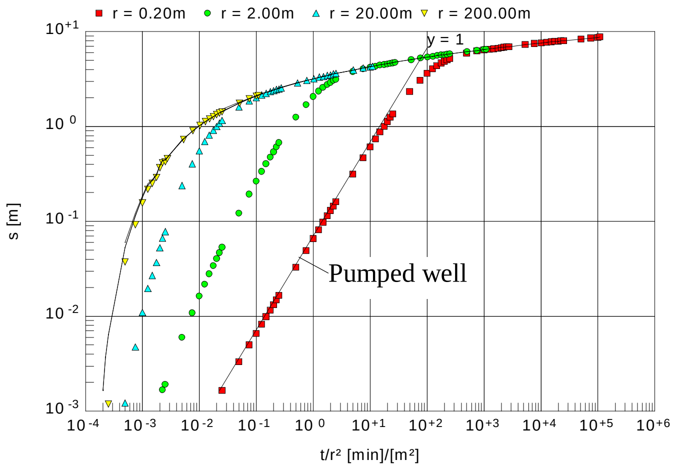
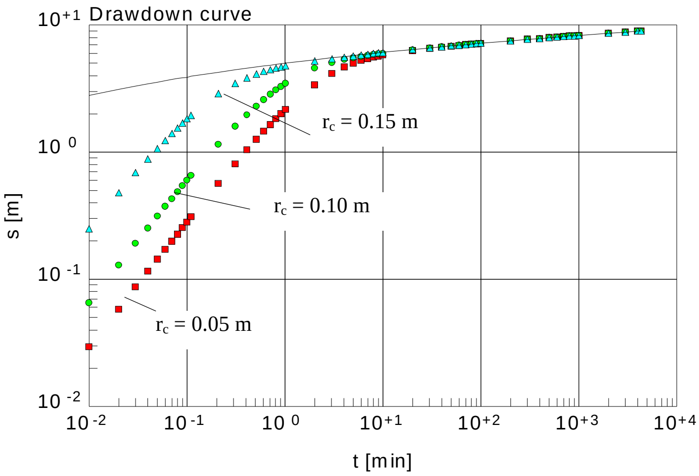


Fig. 2.8 Well bore storage effect in a pumped well and observation wells at various distances. Straight line slope 1 indicates the well bore storage in the pumped well. The solid curve shows the drawdown in the four wells without well bore storage effect. Aquifer type: confined, infinite extended; Discharge Q = 12.5 m^3/h; Transmissivity T = 50 m^2/d; Storage coefficient S = 10^{-4}; Drilled radius r_w = 0.2 m

A unit slope is also typical for the drawdown in a well situated in a closed reservoir; therefore the same physical interpretation can be adopted for the well bore storage. The well can be considered as a reservoir with an almost infinite conductivity in which the 'cone of depression' reaches the 'well bore boundaries' instantaneously, resulting in a drawdown behaviour similar to that of a limited reservoir at early times. However, the period of time for which the well bore storage influence is observable is a function of the transmissivity of the reservoir and the stored volume in the well. Given two equally designed wells situated in two reservoirs of different transmissivities, the well bore storage effect will last longer in the reservoir with the lower transmissivity. Given three wells with different casing diameter r_c drilled into the same reservoir, the well with the larger diameter will show a longer period of well bore storage (Fig. 2.9a). If all wells are pumped at the same rate, the well with a smaller casing radius shows the larger drawdown. This results in a larger gradient between the water level in the well and the aquifer, which forces a deeper cone of depression in the aquifer. Therefore, after any time, t_i, is the portion of the discharge rate from the aquifer depletion in the well with the smaller casing radius bigger than in the well with a larger casing radius, during the phase where the drawdown is affected by well bore storage (Fig. 2.9b).

(a)



(b)

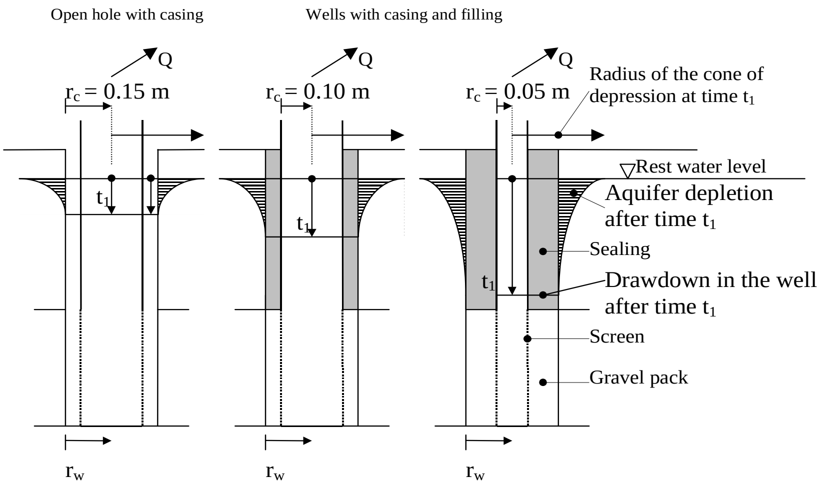


Fig. 2.9 **Well bore storage effect, illustrated as drawdown (a) and sketch (b), in three wells with different casing radius r_c. Aquifer type: confined, infinite extended; Discharge rate Q = 12.5 m^3/h; Transmissivity T = 50 m^2/d; Storage coefficient S = 10^{-4}; Drilled radius r_w = 0.15 m. Solid curve in (a) indicates the drawdown without well bore storage effect**

If the water level in a telescoped casing drops from a bigger diameter into a smaller diameter, the drawdown increases suddenly (Fig. 2.10). On the contrary, if the water level drops from a smaller diameter into a bigger diameter, the drawdown decreases until the well bore storage effect is vanished (Fig. 2.10) (Earlougher, 1977). The same effects will appear if the diameter of the drilled radius changes relatively to the casing radius.

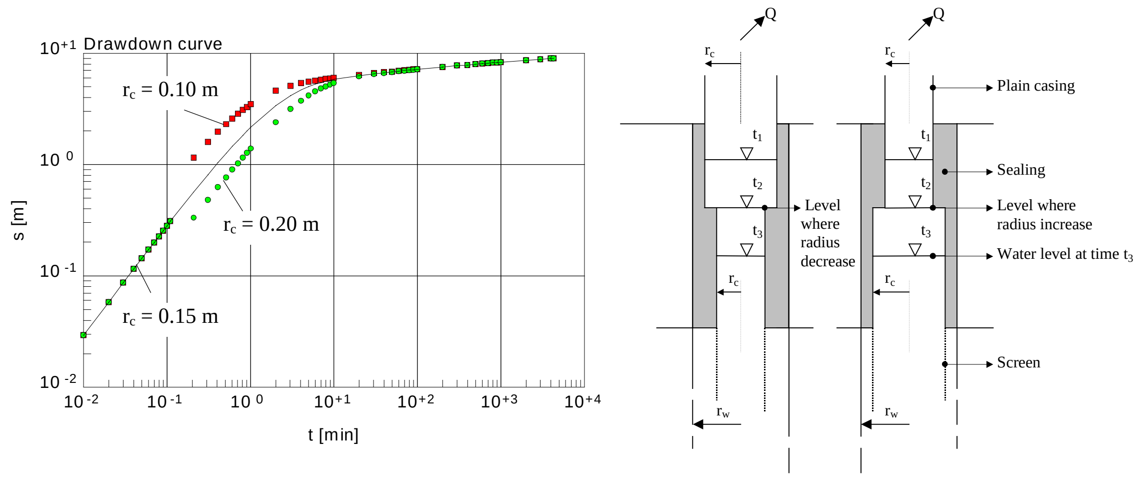


Fig. 2.10 **Drawdown during the well bore storage phase due to changes of the casing radius. The solid curve indicates the drawdown without changes in the casing radius. Aquifer type: confined, infinite extended; Discharge Q = 12.5 m^3/h; Transmissivity T = 50 m^2/d; Storage coefficient S = 10^{-4}; Drilled radius r_w = 0.15 m**

2.6.2 Well bore skin

Well bore skin is a thin layer with a very small storage capacity located between the borehole wall and aquifer that restricts the inflow to a pumped well. It averages the effects of various sources as clogged screens, gravel pack, too small open area of the screens and mineral precipitation between the well wall and formation. In the presence of well bore skin, an additional drawdown is observed within the well (Fig. 2.11). This effect is also known as well losses or skin effect. Mathematically, these losses are described by a linear and a non-linear term (Jacob, 1947). Both terms are respectively constant as long as the discharge rate is constant (Kawecki, 1995). The sum of both well loss components can be represented by a constant total well skin factor ξ, which is simply added to a given well function F (Everdingen, 1952) to calculate the total drawdown within the pumped well:

$$F(u,\xi) = F(u) + \xi \qquad (2.7)$$

where u is the argument that describes the relation between the aquifer parameters T and S as well as the geometry of the abstraction source over the extraction period. The drawdown affected by a skin is a curve parallel to that without skin effects, whereas no effects appear during the recovery phase, except during the well bore storage period (Fig. 2.12).

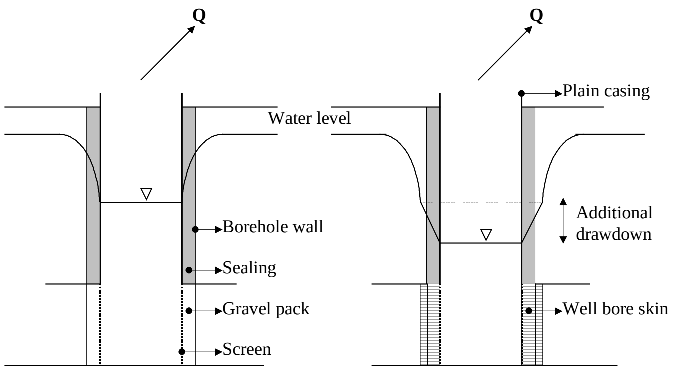


Fig. 2.11 Well bore skin and its effect on the drawdown in a pumped well

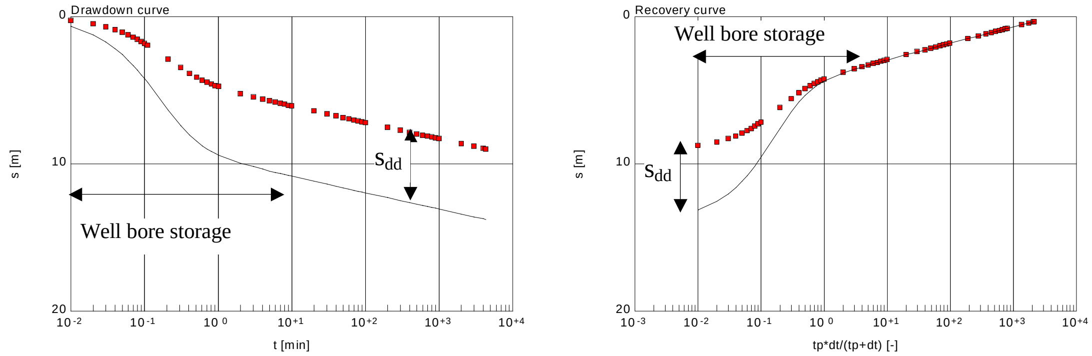


Fig. 2.12 Drawdown and recovery curve with and without additional drawdown cause by a skin

Considering the skin effect, the drawdown s, for a fully penetrating well in a homogeneous confined aquifer, pumped at a constant discharge rate and negligible well bore storage writes (Theis, 1935)

$$s = \frac{Q}{4 \cdot \pi \cdot T} \cdot [Ei(u) + 2\xi] \qquad (2.8)$$

where

$$u = \frac{S \cdot r_w^2}{4 \cdot T \cdot t}$$

$$F(u) = Ei(u), \quad with \quad Ei = \int_u^\infty \frac{e^x}{x} dx$$

Q = discharge rate [L^3T^{-1}]
T = transmissivity [L^2T^{-1}]
t = time [T]
r_w = drilled radius [L]
S = storage coefficient [-]

The dimensionless well skin factor ξ derived from Eq. (2.8) reads:

$$\xi = \frac{2 \cdot \pi \cdot T \cdot s}{Q} - 0.5 \cdot Ei(u) \qquad (2.9)$$

If u ≤ 0.03, the exponential integral Ei(u) is satisfied by the Cooper and Jacob (1946) approximation: Ei(u) ≈ -ln(1/u)-0.5772 within 1% error. The corresponding additional drawdown s_{add} in metre can be calculated from following relationship (Kruseman and DDe Ridder, 1991):

$$s_{add} = \frac{\xi \cdot Q}{2 \cdot \pi \cdot T} \qquad (2.10)$$

In homogeneous aquifers and an ideal well, ξ is zero. Physically, this would mean that the effective radius r_{eff} is equal to the drilled radius r_w ,because ξ can be related to drilled radius as follows (Sabet, 1991):

$$r_{eff} = r_w \cdot e^{-\xi} \qquad (2.11)$$

In case of restricted inflow, ξ becomes positive, which according to Eq. (2.11) results in an effective radius smaller than the drilled radius. In cases where the permeability of the formation around the well is improved, for example with well development, a negative ξ will be observed (Gustafson and Anderson, 1997), which results in an enlarged effective radius. However, for practical purposes, it is unlikely that the development can produce a positive skin larger than 0.5 (Fig. 2.13). An increased effective radius will be observed in a well situated in a single fracture that acts as a conduit (Horne, 1995).

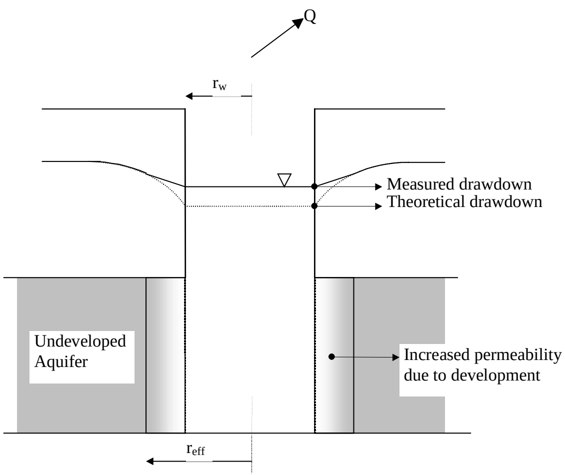


Fig. 2.13 Increased effective radius due to increased permeability zone by development or fracture influence

The additional drawdown effect is almost instantaneous due to the limited storage capacity of the skin layer. During the radial-acting flow phase, the calculated skin effect using Eq. (2.9) will always plot as a horizontal line in a lin-log plot, but not for those parts of the curve affected by well bore storage or other reservoir effects (Fig. 2.14). This effect can be used for identification of the radial-acting flow phase.

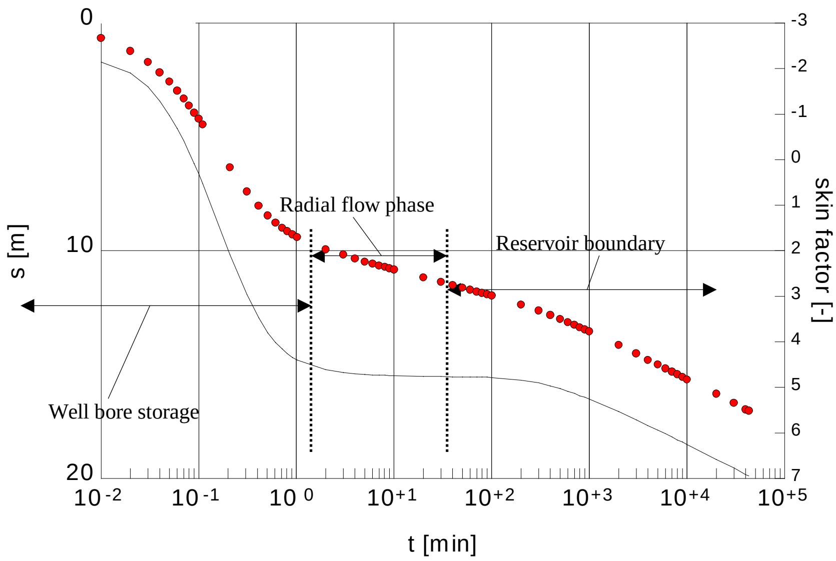


Fig. 2.14 **Drawdown data (dots) and corresponding skin factor ξ (solid line) for a confined homogeneous aquifer with well bore storage and one no-flow boundary. The skin factor plots as a horizontal line during the radial-acting flow phase**

2.6.3 Partial penetration skin

The reduced entrance area in a partial penetrating well causes an additional drawdown due to high velocity losses at the bottom of the well and anisotropy effects of the aquifer in the area close to the well (Fig. 2.15).

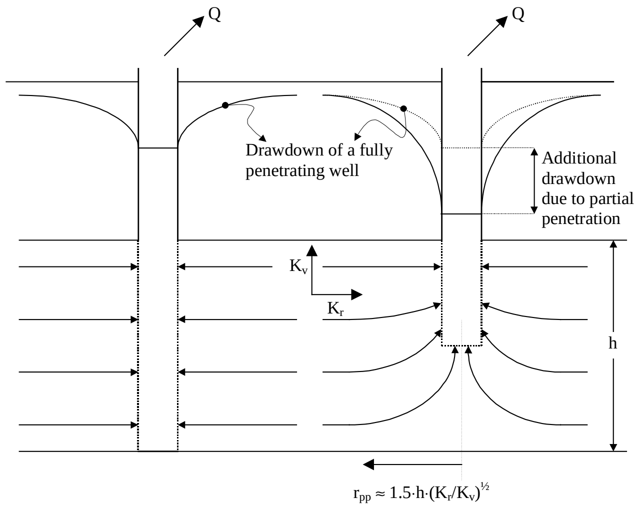


Fig. 2.15 **Flow to a fully penetrating (left) and a partial penetrating well (right)**

The slope of the drawdown in the early time data in the pumped and observation wells within the critical distance r_{pp} is increased and not only shifted as in the case of well bore skin (Fig. 2.16). This effect can lead to an underestimation of the reservoir transmissivity, which might not be dangerous in the design of a water-supply scheme, but it certainly is in the design of a dewatering scheme for mining or engineering purposes. For late time data only an additional drawdown, shown as a parallel shift, is observed (Fig. 2.16). Moench and Ogata (1984) give the following equation for the calculation of the partial penetration skin in the Laplace space:

$$F = \frac{2}{(x_l - x_d)(x_l^{'} - x_d^{'})} \sum_{n=1}^{\infty} \frac{1}{n^2} [\sin(x_l n) - \sin(x_d n)] \cdot [\sin(x_l^{'} n) - \sin(x_d^{'} n)] \cdot K_o(\varphi) \qquad (2.12)$$

where

$$\varphi = \left[r^2 x + \Omega \left(\frac{n\pi r}{h} \right)^2 \right]^{\frac{1}{2}}$$

$x^{½}$ = function that describes the reservoir properties. For a line source in a homogenous confined aquifer $x^{½} = (p \cdot S/T)^{½}$

K_o = modified Bessel function of second kind and order zero
T = transmissivity $[L^2T^{-1}]$
S = storage coefficient [-]
p = Laplace transform variable
Ω = K_v/K [-]
K_v = vertical reservoir conductivity $[LT^{-1}]$
K = horizontal reservoir conductivity $[LT^{-1}]$
x_l = top of the screen related to the top aquifer in the pumped well [L]
x_l' = top of the screen related to the top aquifer in the observation well [L]
x_d = bottom of the screen related to the top aquifer in the pumped well [L]
x_d' = bottom of the screen related to the top aquifer in the observation well [L]
h = aquifer thickness [L]
n = integer value from 1 to infinite, for practical purposes n = 30 is sufficient

Applying the numerical inversion algorithm of Stehfest (1970) to Eq. (2.12) expressed as

$$\bar{h}(p) = \frac{F}{p} \qquad (2.13)$$

gives the additional drawdown due to the partial penetration skin ξ_{pp}.

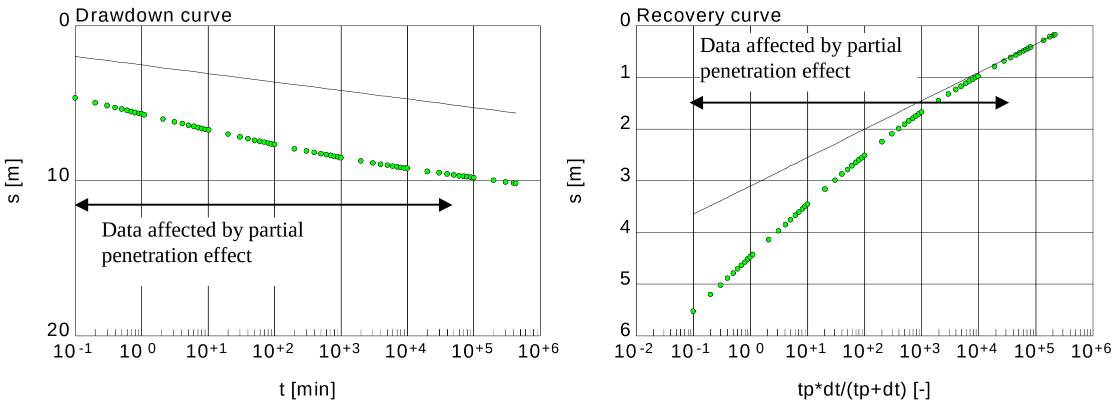


Fig. 2.16 **Increased drawdown and recovery slope at early time due to partial penetration skin. The solid line indicates the drawdown and recovery for a fully penetrating well. Aquifer type: confined, infinite extended; Transmissivity T = 100 m²/d; Storativity S = 7·10⁻⁴; Vertical conductivity 1 m/d; Aquifer thickness h = 100 m; Partial penetration depth = 50 m. The partial penetration effect is negligible after 10⁴ minutes (~7 days)**

2.6.4 Fracture skin

The fracture skin is a thin layer between fracture and matrix with reduced conductivity and very small storage capacity. Such a skin can be created by mineral precipitation (Moench, 1984) or by clay minerals as a result of weathering. Fracture skin in a single fracture causes an additional drawdown similar to that of a well bore skin (Fig. 2.17) (Cinco-Ley and Samaniego, 1977), whereas in fractured rock with double porosity behaviour, it results in a pseudo-steady flow exchange between fracture and matrix blocks (Fig. 2.18) (Moench, 1984). Cinco-Ley and Samaniego (1977) defined the fracture skin factor ξ_f as follows:

$$\xi_f = \frac{\pi \cdot b_s}{2 \cdot x_f} \cdot \left(\frac{k}{k_s} - 1 \right) \qquad (2.14)$$

where
b_s = thickness of the skin [L]
x_f = fracture half-length [L]
k = conductivity of the matrix or formation [LT^{-1}]
k_s = conductivity of the skin [LT^{-1}]

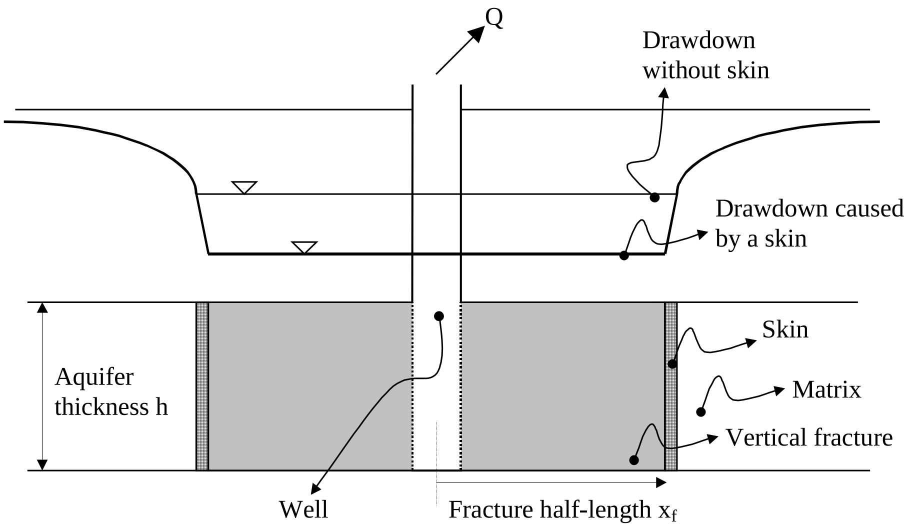


Fig. 2.17 Drawdown in a single vertical fracture caused by a skin between fracture and matrix

Moench (1984) defined the fracture skin factor for the double porosity solution to

$$\xi_f = \frac{k \cdot b_s}{k_s \cdot b} \qquad (2.15)$$

where
b = average half-aperture of the fracture [L]

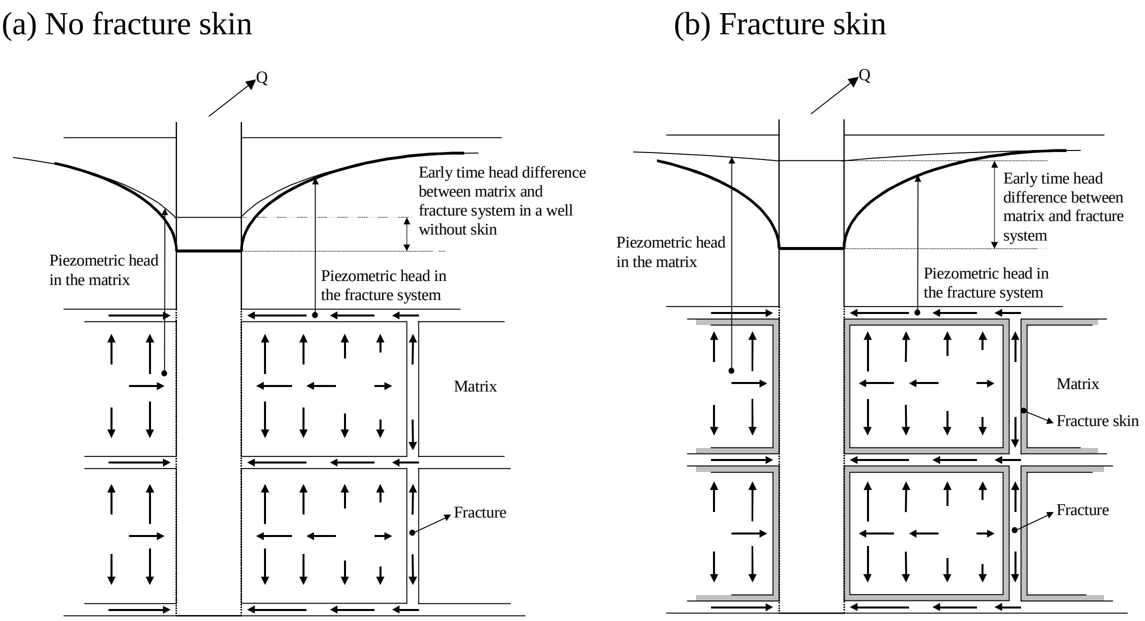


Fig. 2.18 Effect of a fracture skin on the drawdown of the matrix and fracture system in a double porosity aquifer

2.6.5 Pseudo-skin

A well located within or in the proximity of a fracture that acts as a conduit shows less drawdown than that expected for wells in a homogeneous formation within the REV (Fig. 2.19). This effect is known as pseudo-skin (Gringarten and Ramey, 1974). The determination of the skin using Eq. (2.9) would lead to a negative skin factor ξ [-] which, after Eq. (2.10), would result in a larger effective radius.

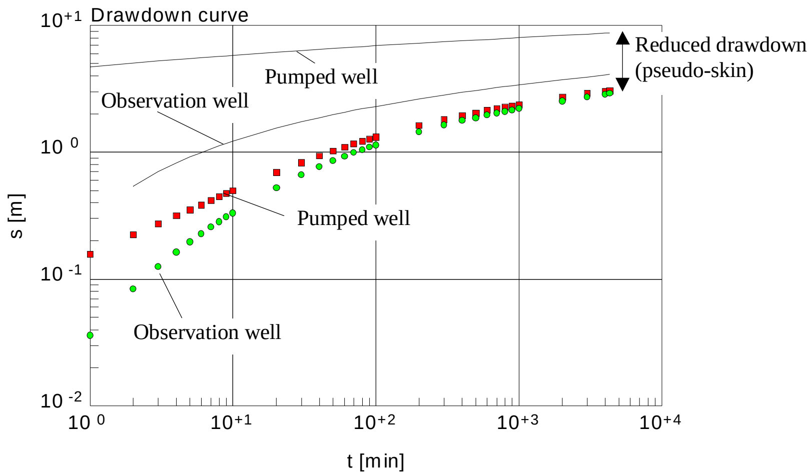


Fig. 2.19 Drawdown in a pumped well situated in a homogeneous aquifer and an observation well 25 m apart (solid curves). Drawdown in a pumped well (squares) situated in a single fracture (fracture half-length x_f = 200m) with infinite conductivity and an observation well (dots) located in the matrix at a distance of 25 m perpendicular to the fracture strike direction. Transmissivity of the matrix T = 50 m^2/d; Storage coefficient S = 10^{-4}. The difference in the drawdown is known as pseudo-skin effect

This effect can be used to determine whether a well is located in a fracture zone, as, in principle, no negative skin factor or enlarged effective radius is observed in a continuous fractured aquifer. If the REV is equal or smaller than the drilled radius r_w (Fig. 2.4), ξ will be zero. An exception might be a zone of higher permeability due to caving processes during the drilling works.

2.6.6 Fracture dewatering

Fracture dewatering should be avoided, whenever feasible, because of the danger of mineral precipitation that can cause fracture and well clogging. These effects are directly related to the water chemistry. Precipitation occurs especially when oxygenation of waters with high manganese, iron or bicarbonate contents is possible.

If continuous fracture network (homogenous aquifer) is dewatered, the physical conditions change gradually with time due to the reduction of the down-hole influx area. Under these circumstances, the dewatering phenomena can be approached, applying the Jacob correction $s' = s - s^2/2h$ to the drawdown data, as in an unconfined aquifer.

If a discontinuous fracture network is dewatered (Fig. 2.20), a sudden drop of the water level in the borehole is observed when it reaches the fracture (Van Tonder *et al.*, 1998). This effect is characteristic of discrete down-hole water strikes. In these cases the physical conditions in the vertical direction change instantaneously due to following reasons:

- The aquifer above the dewatered fracture becomes a purged aquifer that releases water into the fracture and borehole.
- Unconfined conditions in the dewatered fracture.
- Turbulent flow in the dewatered fracture and along the borehole wall.
- Reduced influx area.

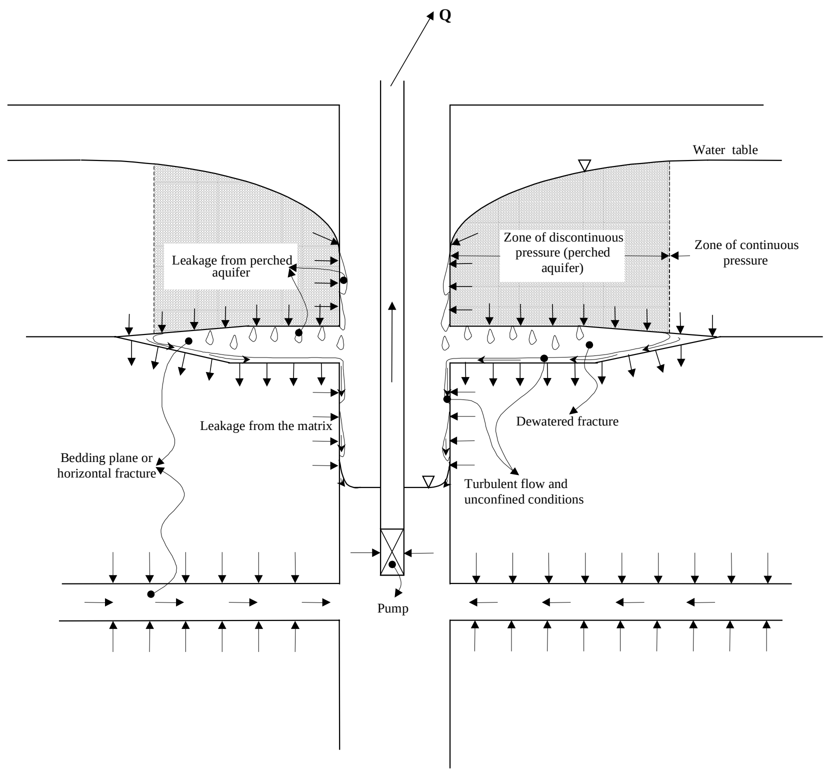


Fig. 2.20 **Effects after the dewatering of a bedding plane or horizontal fracture**

The drawdown scenario can be described as follows:

- As soon the water level in the borehole reaches the water strike, e.g. a bedding plane, the flow conditions in the dewatered fracture change from confined to unconfined.
- If the storage capacity of the fracture is small compared to the discharge rate, the drawdown will drop continuously below the water strike at the cost of the well bore storage (this part of the curve in a log-log plot shows usually a slope of 1), until a new pressure difference between the water level in the borehole and the matrix builds up to cover the discharge rate.
- If radial flow is observed in both before and after the dewatering of the fracture, the drawdown curve after the dewatering e.g. the fracture in the lin-log plot, will show an increased slope compared to the initial one (Fig. 2.21.)

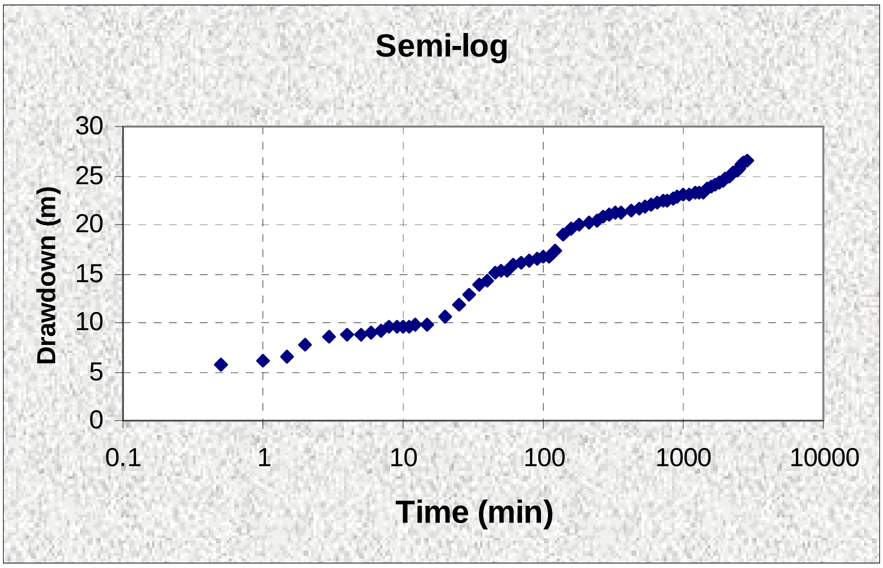


Fig. 2.21 Typical drawdown behaviour during dewatering of discrete fractures

The determination of aquifer parameters, using such disturbed drawdown curves, is sometimes possible when applying conventional methods for parts of the curve. Generally, the evaluation is extremely complicated and, in the case of parameter estimating, it is rather recommended to repeat the drawdown test using a smaller pumping rate to avoid the dewatering of the fractures (see Chapter 6). Step or multirate tests are usually very helpful for the proper adjustment of the pumping rate. The rate to be chosen should lead to a drawdown that does not reach the water strikes. Some fracture dewatering effects on drawdown curves will be demonstrated later using field examples.

2.6.7 Reservoir boundaries

All groundwater reservoirs are limited. Whether the influence of reservoir boundaries is seen in a pumping test curve is a function of the pumping time, the transmissivity, the storage coefficient and the distance of the boundaries, but not from the discharge rate. This can be demonstrated by the calculation of the distance at which the cone of depression is zero (drawdown $s = 0$). The Cooper-Jacob equation which is the solution for the differential Eq. (2.3) for long time ($u < 0.03$), gives:

$$s = \frac{0.183 \cdot Q}{T} \cdot \log\left(\frac{2.25 \cdot T \cdot t}{S \cdot r^2}\right) \qquad (2.16)$$

If a well is discharged, Q > 0 and the term 0.183·Q/T cannot become zero (0). Therefore, the logarithmic should be set equal to zero:

$$\log\left(\frac{2.25 \cdot T \cdot t}{S \cdot r^2}\right) = 0 = s \qquad (2.17)$$

Applying exponential:

$$\frac{2.25 \cdot T \cdot t}{S \cdot r^2} = 1 \qquad (2.18)$$

The radius at which the drawdown disappears is given by the positive result of the square root:

$$r = \sqrt{\frac{2.25 \cdot T \cdot t}{S}} \qquad (2.19)$$

This equation is useful to estimate the extension of a cone of depression or, knowing the distance from the well to the boundary, to determine at which time t, the cone of depression, will reach the boundary assuming that T/S (Diffusivity) remains constant. Eq. (2.19) shows that the larger T and the smaller S, the bigger the cone of depression for a given time. Ferris *et al.* (1960) introduced the widely used concept of mirror wells to consider the effect of positive (recharge) boundaries or negative (no-flow) boundaries. Earlougher (1977), Streltsova (1988) and Kruseman and De Ridder (1991) present a detailed overview on how the mirror well concept can be used. The effects of positive and negative boundaries on the drawdown curve are shown in Fig. 2.22 to 2.25. Basically, recharge boundaries show a flattening of the curve, whereas no-flow boundaries show an increase of the drawdown when the cone of depression reaches the boundary.

(a) (b)

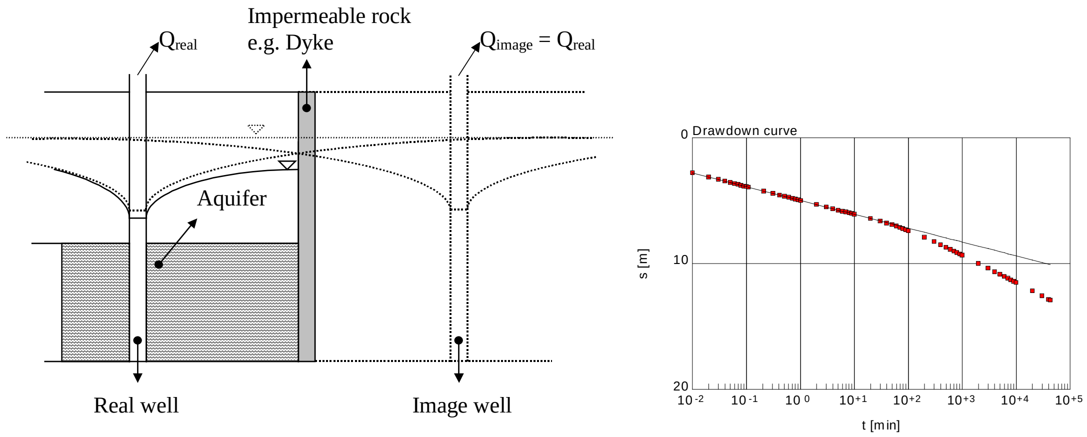


Fig. 2.22 (a) One no-flow boundary and its representation as superposed image well. (b) Example of a drawdown curve affected by one no-flow boundary

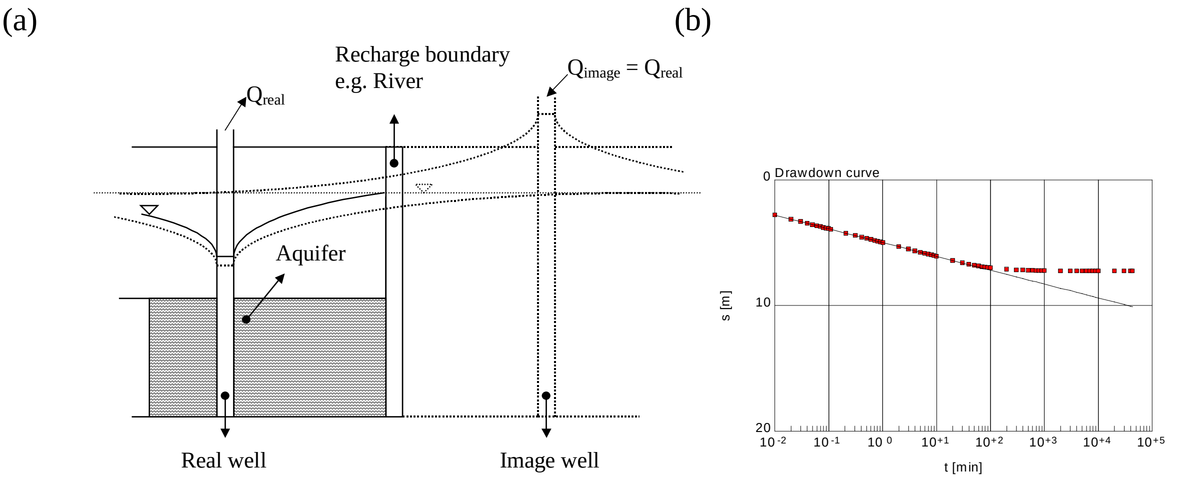


Fig. 2.2 **(a) One recharge boundary and its representation as superposed image well. (b) Example of a drawdown curve affected by one recharge boundary**

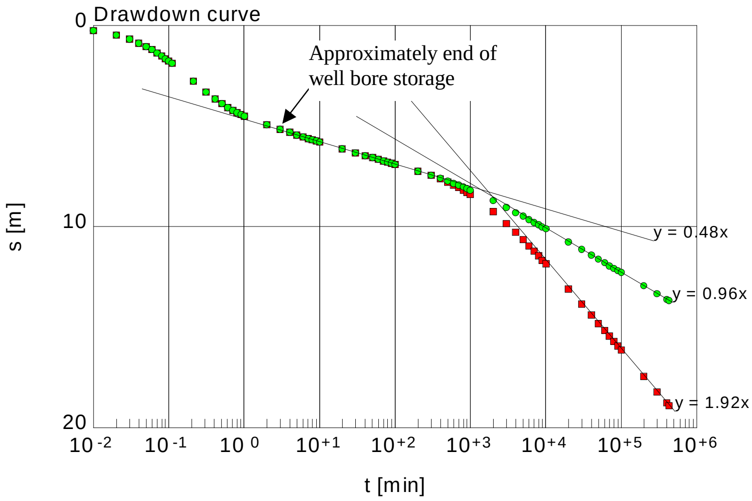


Fig. 2.24 **Diagnostic straight lines in a lin-log plot for the identification of reservoir boundaries. The slope of 0.48 indicates radial flow not affected by boundaries. The double slope 0.96 indicates one no-flow boundary and the slope of 1.92 indicates two perpendicular no-flow boundaries**

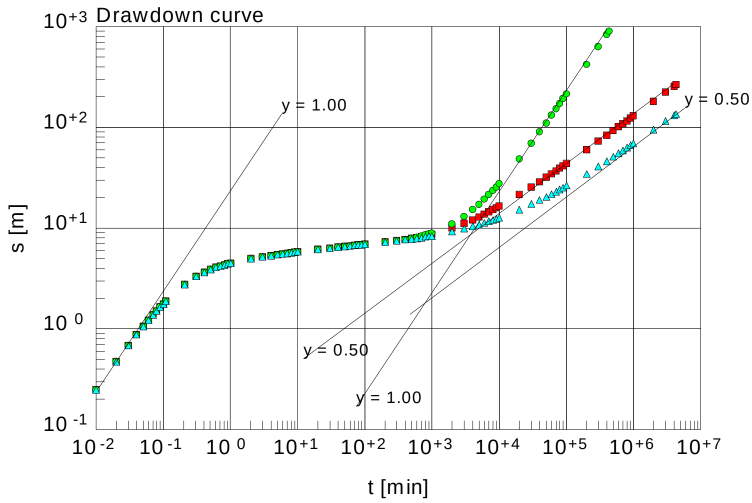


Fig. 2.25 **Diagnostic straight lines in a log-log plot for the identification of various reservoir boundaries. The slope 1 at early times indicates well bore storage effects. The slope 1 at late times indicates a closed reservoir (four boundaries). The slope 0.5 at late times indicates two parallel boundaries (triangles) or three boundaries perpendicular to each other (squares)**

2.7 FLOW MODELS

2.7.1 Double porosity model (Moench, 1984)

2.7.1.1 Theory

The concept of double porosity was introduced by Barenblatt *et al.* (1960), considering homogenous distributed conductive fractures embedded in a homogenous distributed matrix. For both fracture and matrix, different conductivity and storage coefficients can be adopted. Two matrix types are generally discussed; the spherical block matrix (Warren and Root, 1964) used to represent aquifers like quartzite and the layered matrix (Kazemi, 1969) adopted, for example, for sandstones with bedding planes (Fig. 2.26). Olarewaju (1997) introduced a solution that explains the flow exchange between matrix and a fractal fracture system, which is a more realistic approach for the description of double porosity behaviour, but requires detailed knowledge of the fracture system and therefore will not be further considered.

The concept of double porosity was extended to a triple porosity by Abdassah and Ershaghi (1984) which was recently reinterpreted by Al-Ghamdi and Ershaghi (1996) with the introduction of a dual fracture model connected to matrix blocks. Both models concentrate on the interpretation of the behaviour of the very early time data, which are unfortunately often masked by well bore storage effects. Therefore the practical use of these interpretations is questionable and will not be further discussed.

Warren and Root (1964) introduced a pseudo-steady state block-to-fracture flow solution, which seems to adequately represent field data, but Kazemi (1969) and Wei *et al.* (1998) using numerical models, found that the flow is of transient block-to-fracture nature. However, Moench (1984) shows that the pseudo-steady flow case is, in fact, a special case of the transient flow restricted by a skin between fracture and matrix (Fig. 2.17, Page 17) caused by mineral precipitation on the matrix blocks' surfaces.

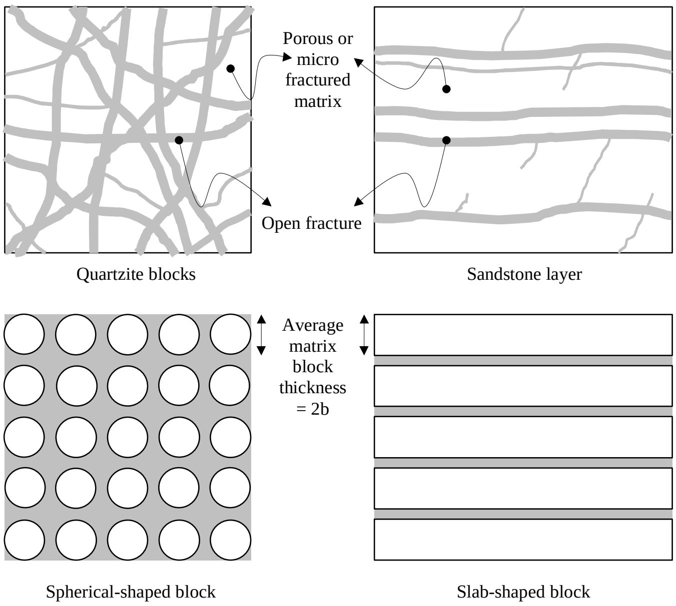


Fig. 2.26 Natural fracture systems and their simplification into spherical-shaped block and slab-shaped blocks

For the mathematical description of the drawdown behaviour, Moench (1984) considers two representative elementary volumes (REV); one for the fractures and one for the matrix. However, the classic concept of double porosity cannot be applied if the cone of depression is not larger than both REV, otherwise the influence of the individual fractures is not negligible as demonstrated by Wei *et al.* (1998). Furthermore, Moench (1984) assumes an infinite extent of the matrix and fracture system under confined conditions. To be able to deal with well bore storage, well losses (skin at the well) and partial penetration, he derived a solution in the Laplace space using the Stehfest (1970) algorithm for the numerical inversion. The solution infers that only water from the fracture network reaches the well and that the contribution of the matrix is negligible. Combining the findings from Mavor and Cinco-Ley (1979), Moench and Ogata (1984) and Moench (1984), the drawdown in a pumped well with wellbore storage, skin effect and partial penetration in the Laplace space can be written as:

$$\bar{h}(p) = \frac{2\left[K_0(x) + x \cdot \xi \cdot K_1(x) + F\right]}{p\left\{p \cdot W_d\left[K_0(x) + x \cdot \xi \cdot K_1(x)\right] + x \cdot K_1(x)\right\}} \qquad (2.20)$$

and in an observation well as

$$\bar{h}(p) = \frac{2\left[K_0(r_d \cdot x) + F\right]}{p\left\{p \cdot W_d\left[K_0(x) + x \cdot \xi \cdot K_1(x)\right] + x \cdot K_1(x)\right\}} \qquad (2.21)$$

where
$\bar{h}(p)$ = dimensionless drawdown in the Laplace space [-]
K_o = modified Bessel function of second kind and zero order [-]
K_1 = Bessel function of second kind and first order [-]
F = partial penetration skin function (Section 6.3)
ξ = dimensionless skin factor at the well (well bore skin factor) [-]
W_d = dimensionless well bore storage coefficient [-]
p = Laplace transform variable [-]
x = $(p+\bar{q}d)^{1/2}$
$\bar{q}$ = dimensionless block-to-fracture flow [-]
r_d = dimensionless radius defined as r/r_w [-]
r = distance of the observation well to the pumped well [L]
r_w = drilled radius of the pumped well [L]

The dimensionless transient block-to-fracture flow $\bar{q}$ for sphere-shaped blocks is given by Moench (1984) in the Laplace space as

$$\bar{q} = \frac{3 \cdot \gamma^2\left[m \cdot \coth(m) - 1\right]}{\left\{1 + \xi_f\left[m \cdot \coth(m) - 1\right]\right\}} \qquad (2.22)$$

and for slab-shaped blocks

$$\bar{q} = \frac{\gamma^2 \cdot m \cdot \tanh(m)}{\left[1 + \xi_f \cdot m \cdot \tanh(m)\right]} \qquad (2.23)$$

Moench (1984) also gives solutions for the pseudo-steady flow exchange, which seems to be contradictive to the general findings of his paper. The solution produces an almost horizontal flattening of the transient phase for small dimensionless skin value ξ_f, which in some instances may fit data better than the transient solution. The flow $\bar{q}$ for sphere blocks under a pseudo-steady state flow situation reads:

$$\bar{q} = \frac{3 \cdot \gamma^2 \cdot m^2}{(3 + \xi_f \cdot m)} \qquad (2.24)$$

and for slab-shaped blocks

$$\bar{q} = \frac{\gamma^2 \cdot m^2}{(1 + \xi_f \cdot m)} \qquad (2.25)$$

where:

ξ_f = dimensionless fracture skin
γ = $r_w/b\ (K/K_f)^{1/2}$ [-]
m = $(S/S_f\ p)^{1/2}/\ \gamma$ [-]
r_w = drilled radius [L]
b = average half thickness of the block [L]
K = conductivity of the matrix block [LT^{-1}]
K_f = conductivity of the fracture system [LT^{-1}]
S = storage coefficient of the matrix block [-]
S_f = storage coefficient of the fracture system [-]

Unfortunately, if no skin is present, the matrix responds immediately to pressure changes, which results in an almost instantaneous water release from the matrix into the fracture network masking the flattening effects (Fig. 2.27).

The double porosity model, presented by Moench (1984), was implemented in TPA because it considers a fracture skin thus being a more realistic approach, which produces a restricted flow exchange between matrix and fracture.

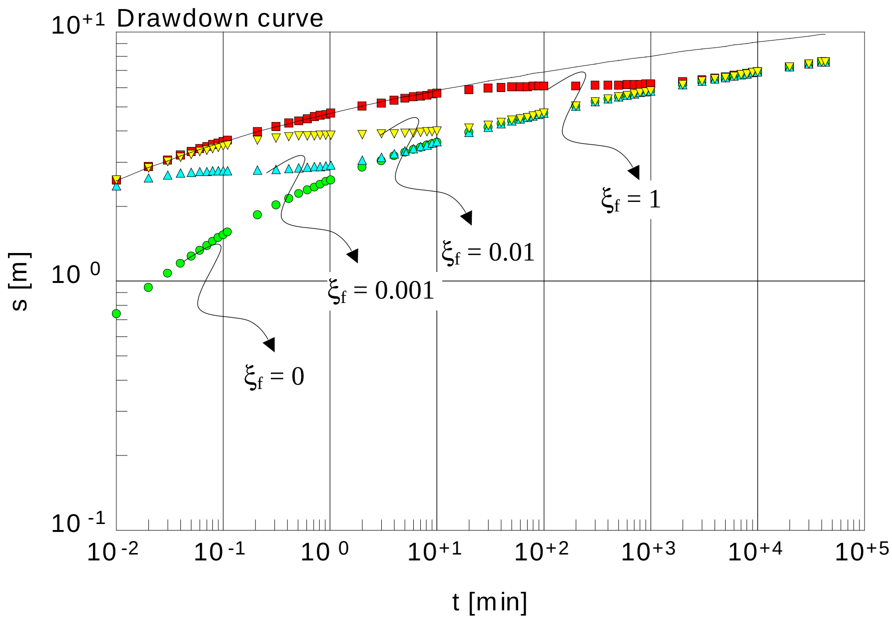


Fig. 2.27 **Comparison of the drawdown behaviour with and without skin in the pseudo-steady case. The solid curve represents the flow in a confined infinite aquifer**

2.7.1.2 Diagnosis

Initially, mainly the fracture network of a double porosity aquifer releases water when pumping from a well starts and the drawdown are characterised by a straight line in a lin-log plot that is proportional to the transmissivity of the fracture system. The flattening of the curve is originated by the ever-increasing additional contribution from the matrix storage. The later drawdown is the response of both the matrix and the fracture storage, and is also proportional to the transmissivity of the fracture system and plots as by a straight line parallel to the initial one in the lin-log plot. In a log-log plot, both the initial and the late drawdown are characterised by Theis curves shifted horizontally from each other. The distance between the two parallels or two Theis curves depends on the fracture/matrix storage coefficient ratio (Fig. 2.28).

Both the pseudo-steady state and the transient model with fracture skin show an almost horizontal flattening (Fig. 2.29). The flattening in the transient model without block to fracture skin is not horizontal; it has a slope of half the slope of the two parallels (Fig. 2.30). If well bore storage masks the first straight line, this behaviour can be mistaken as a drawdown in a confined aquifer with one no-flow boundary (Fig. 2.31). In fact, there is no unique diagnosis method available, if data from observation wells are not available. Fig. 2.32 shows that in the case of double porosity data plotted in a log-log plot, the extension of the late time data joins the distance depending time axis = t/r^2, whereas in case of a boundary effect, the medium time data join this axis, but not the late time data. However, this diagnosis method works for a pumped well and one observation well if the well losses or skin at the pumped well are negligible, or if data of at least two observation wells are available (Fig 2.32).

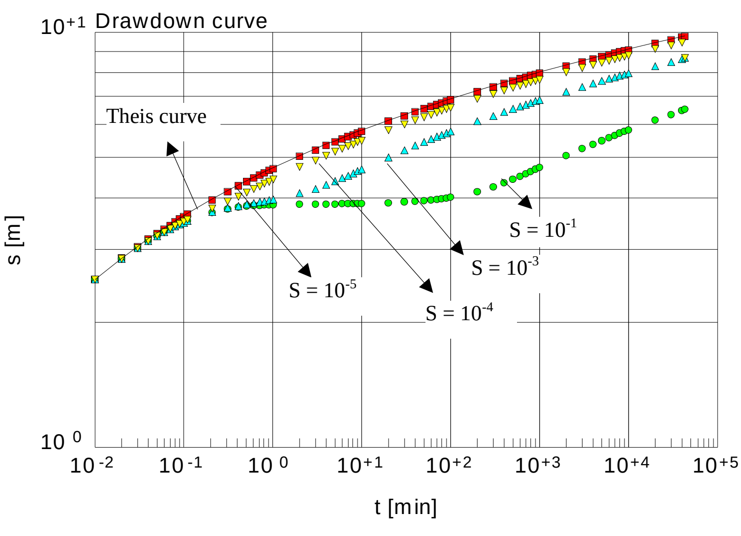


Fig. 2.28 **Drawdown curves for various matrix storage coefficients S and storage coefficient of the fracture $S_f = 10^{-4}$**

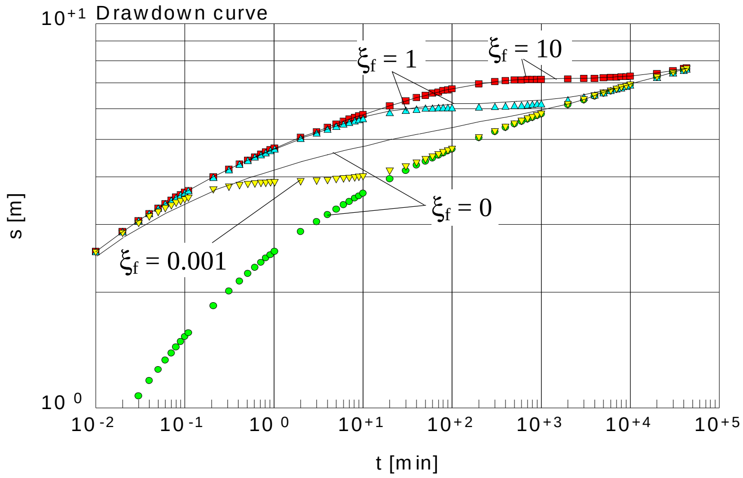


Fig. 2.29 Comparison between pseudo-steady state flow (marker) and transient flow (solid curves) for various fracture skins

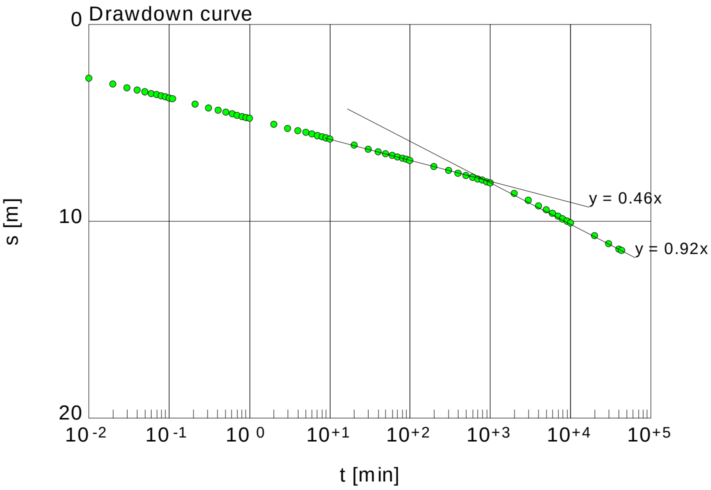


Fig. 2.30 Drawdown in a double porosity aquifer with transient block to fracture flow and no fracture skin

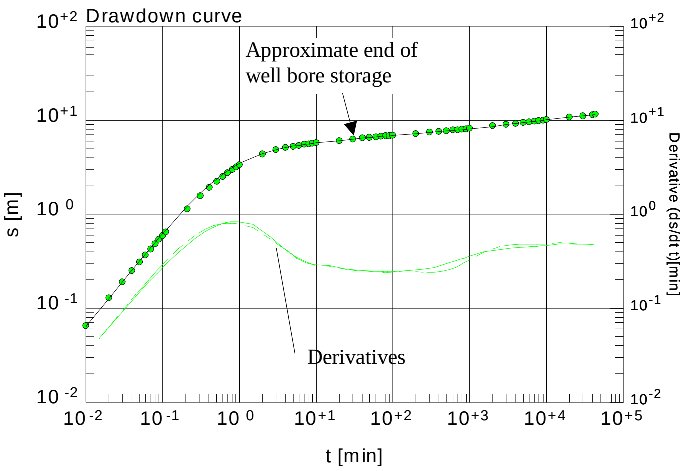


Fig. 2.31 Comparison between the drawdown in a confined aquifer with one no-flow boundary (marker) and in a double porosity aquifer (solid line) and well bore storage. For all practical purposes, it is not possible to distinguish between both cases

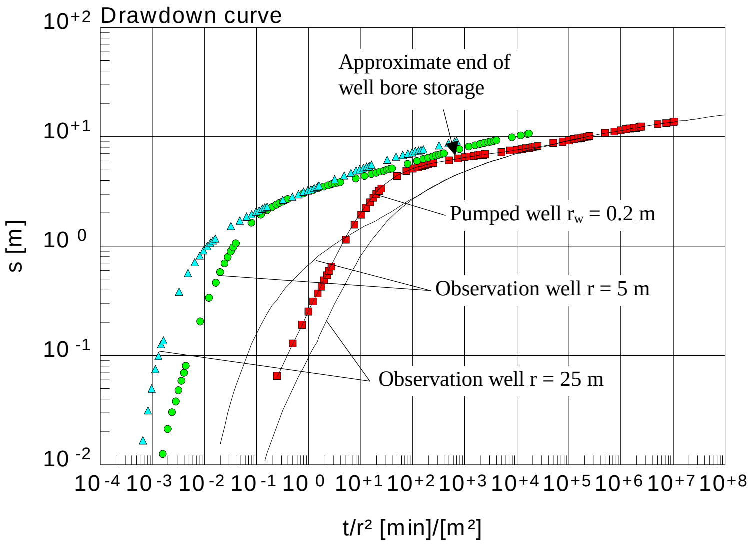


Fig. 2.32 Comparison between the drawdown of a pumped well and two observation wells in a confined aquifer with one no-flow boundary (marker) and in a double porosity aquifer (solid curves) and well bore storage. It is clearly seen that the double porosity curves merge the time dependent axis at late time, whereas in the confined case the merge occurs at medium time, while the boundary is not affecting the drawdown

2.7.1.3 Method of analysis

Moench (1984) proposed a type curve approach for the analysis of double porosity data, which includes well bore storage, well skin and fracture skin and is therefore more advanced than the simple straight line approach proposed by Warren and Root (1963) and Kazemi (1969). However, due to the tremendous increase of computer calculation power, a combined approach of straight line and forward modelling is nowadays possible. This approach will be described in this work using TPA. The advantage of this proposed method lies in the ability to calculate different drawdown scenarios after model calibration to find the optimal abstraction rate for a particular well.

2.7.1.3.1 Application of straight line methods

Once a double porosity case is diagnosed and it has been determined that the influence of the well bore storage is negligible, the Warren and Root straight line method can be applied to the pumped well data to determine the transmissivity T of the fracture system and the storage coefficients S_f and S for the fracture system and the matrix, respectively (Fig. 2.33).

Whether the method can be applied or not depends on the following conditions:

- $u < 0.03$ for the first straight line, $u = S_f\, r^2/(4t\, T_f)$.
- $1/u > 100$ for the second straight line, $u = (S_f+\beta S)r^2/(4t\, T_f)$.

These limits should be cross-checked with the first derivative of the data curve, which must plot as a horizontal line within the limit area (Fig. 2.33). The steps to follow in the application of the Warren .and .Root method for the pumped well data using a lin-log plot are:

- One straight line must be fitted to the early time data (first branch) of the curve.
- A second straight line must be fitted to the late time data (second branch).

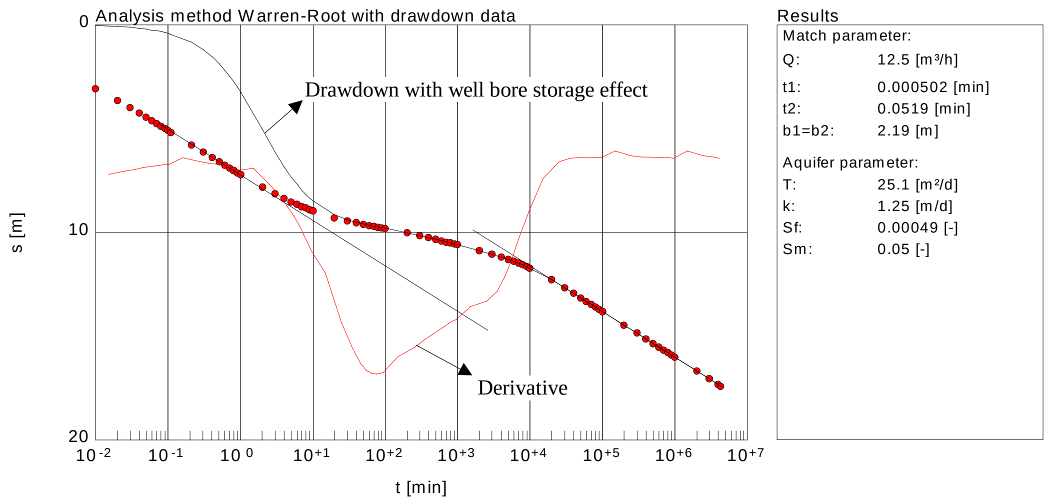


Fig. 2.33 Application of the Warren and Root method to a pumped well that shows double porosity behaviour. Solid curve indicates drawdown affected by well bore storage, which does not allow fitting of a straight line at early times

Both straight lines must have the same slope, as they reflect the transmissivity of the fracture system. In the presence of skin at the well, the Warren and Root method gives wrong results for the two storage coefficients, as demonstrated in Fig. 2.34. A simple method for the skin determination will be presented in the following section. The Kazemi straight line method should be applied to observation data. Both methods are in fact, similar to the Cooper-Jacob straight line method. The aquifer parameters for both methods are calculated as follows:

$$T_f = \frac{0.183 \cdot Q}{d} \qquad (2.26)$$

$$S_f = \frac{2.25 \cdot t_{01} \cdot T_f}{r^2} \qquad (2.27)$$

$$S = \left[\frac{2.25 \cdot t_{02} \cdot T_f}{r^2} - S_f \right] \cdot \beta \qquad (2.28)$$

where

d = drawdown of the straight lines over one log cycle [-]

r = effective well radius (Warren and Root method) or distance to the pumped well (Kazemi method) [L]

T_f = transmissivity of the fracture system [L^2T^{-1}]

S_f = storage coefficient of the fracture system [-]

S = storage coefficient of the matrix [-]

β = shape factor: 1/3 for spherical blocks; 1 for slab blocks [-]. This parameter implies that the conceptual model of the geological situation is known

t_{01} = time at which the first straight line intercepts the time axis [T]

t_{02} = time at which the second straight line intercepts the time axis [T]

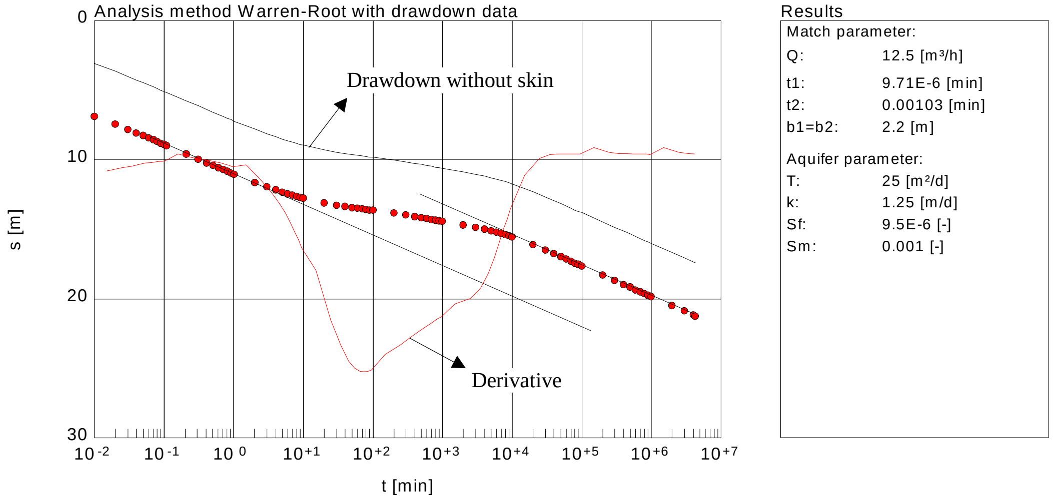


Fig. 2.34 The solid curve is drawn using the same aquifer parameter as in Fig. 31. An additional drawdown of 2 m due to well bore skin still gives the same transmissivity T = 25 m^2/d, but both storage coefficients S and S_f are much smaller due to the fact that the extrapolated time value for Eqs. 2.27 and 2.28 is wrongly determined

The method can be applied if the following conditions are true:

- Aquifer is infinite.
- Aquifer is confined.
- Darcian flow prevails in fracture network and matrix.
- Fracture network is considered as continuum during the whole abstraction period.
- Matrix is considered as continuum during the whole abstraction period.
- Well penetrates the aquifer fully.
- Negligible well bore storage.
- Negligible well bore skin.
- First straight line can be applied if u < 0.03, where u is defined as $S_f\, r^2/(4t\, T_f)$ with t = time since extraction started. Crosscheck, if the time is equal to the time at which the first derivative becomes horizontal.
- Second straight line can be applied if 1/u > 100, where u is defined as $(S_f+\beta S)r^2/(4t\, T_f)$ and t = the time at which the first derivative for late time data becomes horizontal.

2.7.1.3.2 Determination of the well bore skin

Eq. (2.9) can be applied for the determination of the well bore skin in a double porosity aquifer, if the transmissivity T is set equal to the fracture transmissivity T_f. If data of an observation well are available, the Kazemi method can be applied to determine the correct values for the storage coefficient of the fracture system S_f and the matrix S. If observation data are not available, the storage coefficient must be estimated. A coefficient $S_f = 10^{-4}$ for a fracture system is considered adequate as it assumes confined conditions at early time. The value for the matrix storage coefficient is more variable and therefore difficult to estimate.

The storage coefficient for the fracture system S_f can be exchanged for S in Eq. (2.9), which will give a zero skin factor ξ for the early time data, whereas the transient and late time data result in a negative skin value (Fig. 2.35a). Alternatively, the sum of $S_f + \beta S$ can be exchanged for S, which results in a positive skin for the early times and transient data and a zero value for the late time data in the absence of a skin (Fig. 2.35b).

a) b)

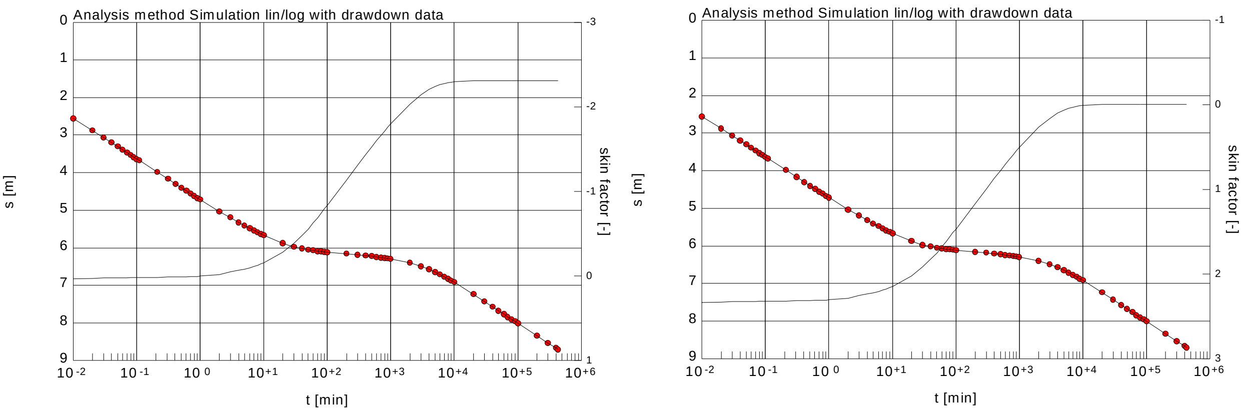


Fig. 2.35 Determination of the skin factor for early and late times

An easier approach is shown in Fig. 2.36, where the offset between the late time data of the pumped well data and an observation well indicates the skin effect in meter of additional drawdown. Eq. (2.10) can be used to solve for the dimensionless skin factor ξ.

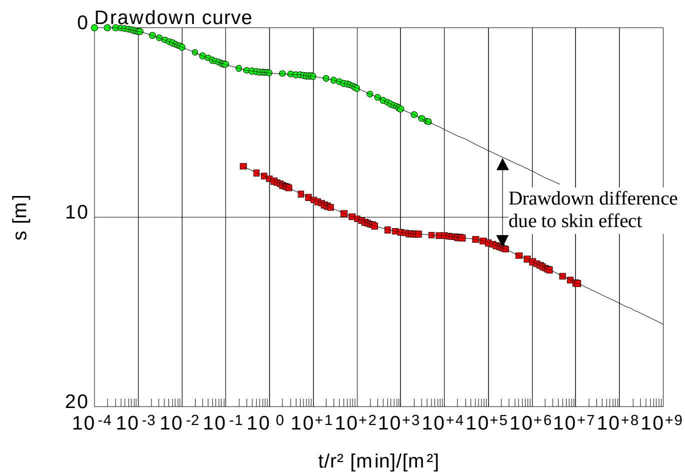


Fig. 2.36 The offset between the late time data in a pumped well (dots) and an observation well (squares) indicates the additional drawdown in metres that can be used as an initial approximation of the skin factor ξ

2.7.1.3.3 Application of the forward modelling

The aquifer values from the straight line approach, the well bore skin determination and the conceptual model of the geology in the well constitute the 'known aquifer parameter' for the forward modelling. These are:

- Transmissivity of the fracture network T_f $[L^2T^{-1}]$.
- Storage coefficient of the fracture network S_f [-].
- Storage coefficient of the matrix block S [-].
- Well bore skin factor ξ [-].
- Principle geometry of the matrix block.
- Characteristic thickness of the matrix block b [L].

The unknown parameters for the Moench model are:

- Conductivity of the matrix block K $[LT^{-1}]$.
- Fracture skin ξ_f [-].

These two unknowns must be estimated via trial and error to fit the measured data until the simulated curve and its derivative satisfactorily fit the measured data and its derivative. This procedure is very fast for the experienced user. Usually, 3 to 5 runs are enough to get a good fit.

2.7.1.4 Field examples

<u>First example: Effects of well bore skin and well bore storage</u>

Fig. 2.37 shows the forward modelling results for the drawdown and its derivatives using TPA. The data correspond to drawdown data of a pumped well (UE-25b) and an observation well (UE-25a) situated in an aquifer of layered volcanic rocks that show double porosity behaviour from a drilling site in Nevada, USA, published by Moench (1984). The curve fit was achieved using the same aquifer parameters determined by Moench:

Discharge rate $[m^3/h]$	Q	= 129
Fracture transmissivity $[m^2/d]$	T_f	= 345.6
Fracture storage coefficient [-]	S_f	= 0.0006
Matrix storage coefficient [-]	S	= 0.12
Matrix conductivity [m/d]	K	= 0.7
Well bore skin factor [-]	ξ	= 0
Fracture skin factor [-]	ξ_f	= 1
Aquifer thickness [m]	h	= 400
Half slab thickness [m]	2b	= 80
Drilled well bore radius [m]	r_w	= 0.11
Distance of the observation well [m]	r	= 110

Both curves do not show horizontal derivatives, therefore the straight line methods of Warren and Root (1964) and Kazemi (1969) cannot be applied. The solution was instead obtained using forward modeling. Although the results for both wells are acceptable, the fit for the pumped well can be improved if a negative well bore skin factor ξ = -0.5 is applied.

This indicates that either the drilled radius is larger than published or the REV for the fracture network is larger than the drilled radius.

The drawdown data of the pumped well at early times show an increase of the casing radius to approximately 0.14 m, thus data are still affected by well bore storage (Fig. 2.38). An increase of the matrix storage coefficient S to 0.25 leads to a better fit of the late time data from the observation well (Fig. 2.38).

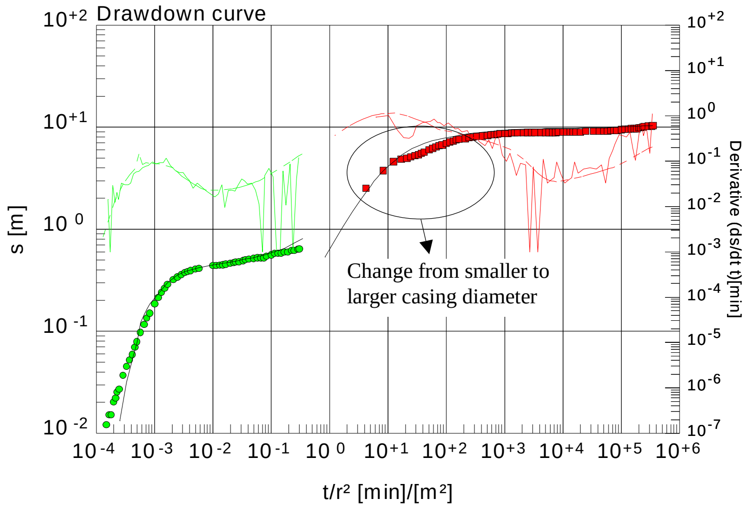


Fig. 2.37 Forward modelling results for the data published by Moench. Both curves do not show horizontal derivatives, therefore the straight line methods of Warren and Root and Kazemi cannot be applied

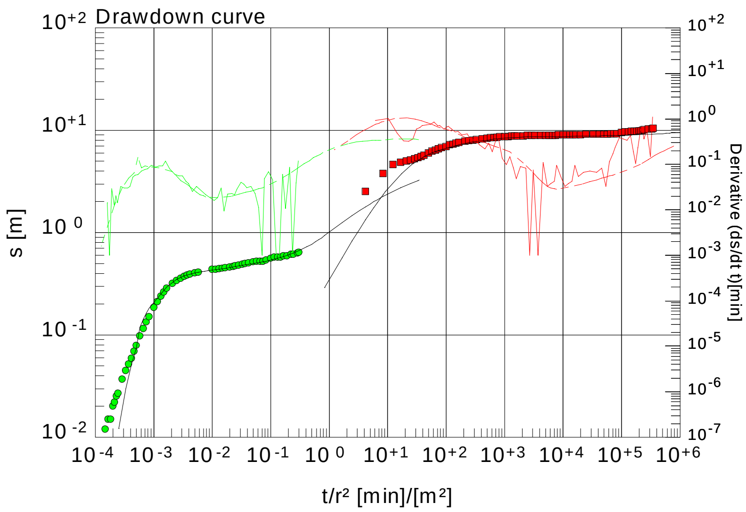


Fig. 2.38 Forward modelling with matrix storage coefficient S increased to 0.25 and casing radius of the pumped well increased to 0.14 m

Second example: Effect of dewatered water strike

Fig. 2.39 presents data of a pumped well and an observation well situated in a layered Karoo sandstone aquifer in Botswana. Both curves do not show horizontal derivatives, therefore the straight line methods of Warren and Root (1964) and Kazemi (1969) cannot be applied. The solution was obtained using forward modelling instead. The aquifer parameters are determined as:

Discharge rate [m^3/h]	Q	= 19.2
Fracture transmissivity [m^2/d]	T_f	= 35
Fracture storage coefficient [-]	S_f	= 0.0003
Matrix storage coefficient [-]	S	= 0.015
Matrix conductivity [m/d]	K	= 0.011
Well bore skin factor [-]	ξ	= 3.7
Fracture skin factor [-]	ξ_f	= 0.3
Aquifer thickness [m]	h	= 70
Slab thickness [m]	$2b$	= 11
Drilled well bore radius [m]	r_w	= 0.155
Distance of the observation well [m]	r	= 14.1

Using the above listed parameters, it is possible to perfectly fit the observation well data, but not the pumped well data due to the dewatering of the main water strike, which was recorded at 9 m below the rest water level. This example illustrates that in the case of water strike dewatering, it is not possible to determine the aquifer parameters if only data from the pumped well are available. The early time data in the pumped well can be fitted with a well bore skin factor of $\xi = 3.7$, but not the medium and late time data (Fig. 2.39).
The Jacob's correction ($s = s - s^2/2h$) applied to the drawdown data of the pumped well is not sufficient to overcome the additional losses due to the dewatering of a water strike. The correction leads to a reduction of the well bore skin factor to $\xi = 2.4$, but the medium and late time data still cannot be fitted (Fig. 2.40).

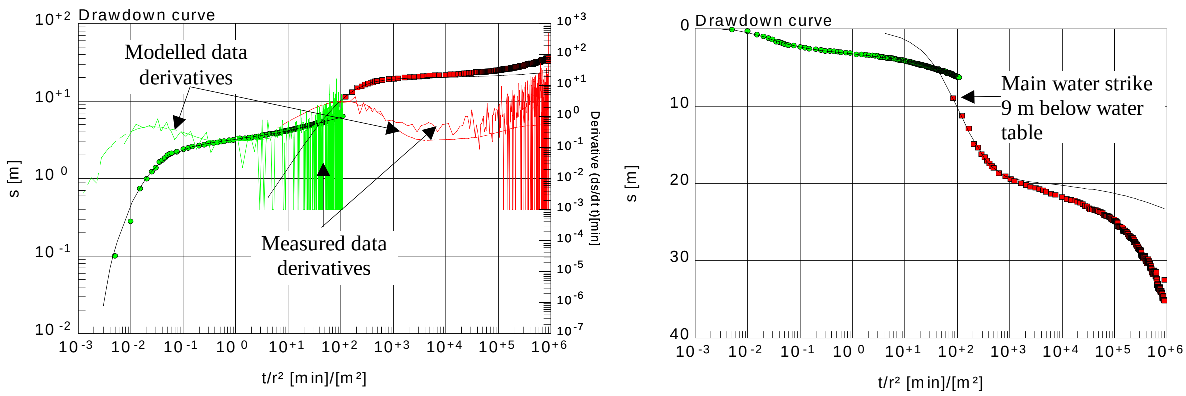


Fig. 2.39 **The simulated curve (solid line) fits the observation well data (dots) very well, but not the pumped well data (squares) due to additional well losses caused by dewatering of the main water strike. Early time data of the pumped well are fitted using a well bore skin factor of $\xi = 3.7$**

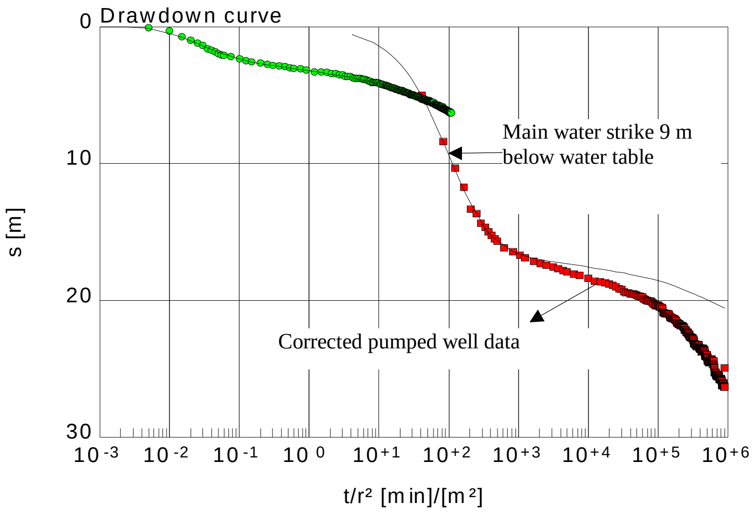


Fig. 2.40 **Jacobs correction ($s = s - s^2/2h$) applied to the drawdown data of the pumped well is not sufficient to overcome the additional losses due to the dewatering of a water strike. Early time data of the pumped well can be fitted using a well bore skin factor of $\xi = 2.4$**

2.7.2 Single vertical fracture with infinite conductivity and finite extend (Gringarten *et al.*, 1974)

2.7.2.1 Theory

The drawdown behaviour in wells connected to a single fracture became a focus of interest in the petroleum industry after the introduction of hydraulic fracturing, which is used to enhance the yield of production boreholes in low permeable formations. Stober (1986), Merton (1987) and other researchers showed that the solutions applied to hydro fracturing could be utilised for natural vertical fracture cases.

Prat (1959) investigated the drawdown behaviour of single vertical fractures with finite extent using an electrical analogue model. Based on Green's source function and applying the Neuman product, Gringarten *et al.* (1974) produced the following general solutions for the dimensionless drawdown p_d in a vertical fracture with infinite conductivity and finite length:

$$p_d(x_d, y_d, t_d) = \int_0^{t_d} e^{-\frac{y_d^2}{4(t-t')_d}} \sum_{m=1}^{M} Q_m(t_d) \cdot F(x_d, t_d) \cdot \frac{dt_d'}{4\sqrt{(t-t')_d}} \qquad (2.29)$$

where

$$Q_m(t_d) = \frac{2q_m(t_d') \cdot h \cdot x_f}{Q}$$

q_m = Influx rate per fracture segment [L^2T^{-1}]
Q = Influx rate over the entire length of the fracture equivalent to the discharge at the well [L^3T^{-1}]
h = Fracture height [L]
x_f = Fracture half-length [L]

$$F(x_d, t_d) = erf\left(\frac{x_d + \frac{m}{M}}{2\sqrt{(t-t')_d}}\right) - erf\left(\frac{x_d + \frac{m-1}{M}}{2\sqrt{(t-t')_d}}\right) - erf\left(\frac{x_d - \frac{m}{M}}{2\sqrt{(t-t')_d}}\right) + erf\left(\frac{x_d - \frac{m-1}{M}}{2\sqrt{(t-t')_d}}\right)$$

x_d = Cartesian dimensionless distance, x/x_f [-]
y_d = Cartesian dimensionless distance, y/x_f [-]
t_d = Dimensionless time,

$$t_d = \frac{T \cdot t}{S \cdot x_f^{\ 2}} [-]$$

T = Formation transmissivity [L^2T^{-1}]
t = Time [T]
S = Formation storage coefficient [-]
M = Number of fracture segments
m = Integer number
dt = Integration variable [-]
t_d' = Integration parameter [-]
e = Exponential function
erf = Error function,

$$erf(x) = \frac{2}{\sqrt{\pi}} \int_0^x e^{-u^2} du$$

The transformation of Eq. (2.34) in real world space is given by

$$s = \frac{Q}{2 \cdot \pi \cdot T} \cdot p_d(x_d, y_d, t_d) \qquad (2.30)$$

Gringarten *et al.* (1974) presented solutions for two different boundaries along the fracture surface:

- Uniform flux, where the flux distribution is homogeneous along the fracture and constant in time.
- Infinite flux, where the flux distribution is uniform along the fracture during the linear flow phase, but not in the transient and radial-acting flow phases. Simultaneously, it varies with time until the radial-acting flow phase is reached.

In the uniform flux case the flux Q_m is independent of time and can be taken out of the integral in Eq. (2.29). The solution reduces to

$$p_d(x_d, y_d, t_d) = \int_0^{t_d} e^{-\frac{y_d^2}{4t_d'}} \cdot \left[erf\left(\frac{1 - x_d}{2\sqrt{t_d'}} \right) + erf\left(\frac{1 + x_d}{2\sqrt{t_d'}} \right) \right] \frac{dt'}{4\sqrt{\frac{t_d'}{\pi}}} \qquad (2.31)$$

Generally Eq. (2.31) must be solved by numerical integration. However, for the pumped well and observation wells located along the fracture analytical solutions exist. For the pump well the analytical solution reads

$$p_d(0,0,t_d) = \sqrt{\pi \cdot t_d} \cdot erf\left(\frac{1}{2\sqrt{t_d}} \right) - \frac{1}{2} Ei\left(-\frac{1}{4 \cdot t_d} \right) \qquad (2.32)$$

For observation wells along the fracture, the solution is given by

$$p_d(x_d,0,t_d) = \frac{1}{2}\sqrt{\pi \cdot t_d} \cdot \left[erf\left(\frac{1 - x_d}{2\sqrt{t_d}} \right) + erf\left(\frac{1 + x_d}{2\sqrt{t_d}} \right) \right] - \frac{(1 - x_d)}{4} \cdot Ei\left[-\frac{1 - x_d}{4 \cdot t_d} \right] - \frac{(1 + x_d)}{4} \cdot Ei\left[-\frac{1 + x_d}{4 \cdot t_d} \right] \qquad (2.33)$$

Gringarten *et al.* (1974) found that the drawdown at a dimensionless distance $x_d = 0.732$ is equal for both the uniform flux and the infinite flux cases (Fig. 2.41). Substituting this x_d value in Eq. (2.33), the drawdown reads:

$$p_d(0,0,t_d) = \frac{1}{2}\sqrt{\pi \cdot t_d} \cdot \left[erf\left(\frac{0.134}{\sqrt{t_d}} \right) + erf\left(\frac{0.866}{\sqrt{t_d}} \right) \right] - 0.067 \cdot Ei\left(-\frac{0.018}{t_d} \right) - 0.433 \cdot Ei\left(-\frac{0.75}{t_d} \right) \quad (2.34)$$

Although the drawdown for both cases does not differ much, measured data can usually be described by only one of them (Fig. 2.42).

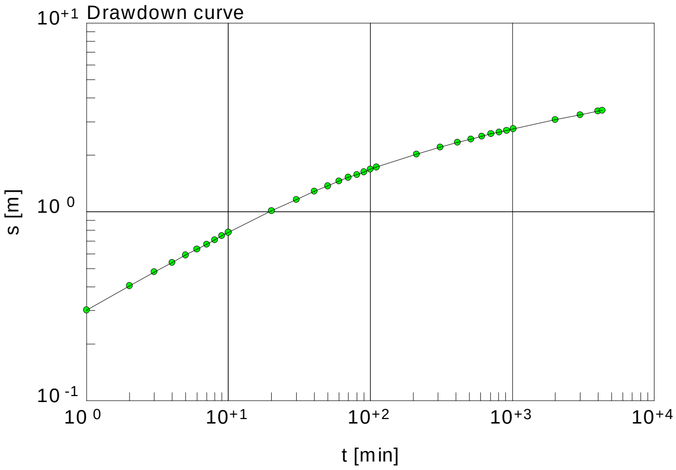


Fig. 2.41 Comparison of the drawdown in an infinite conductive vertical fracture for uniform flux boundary at x_d = 0.732 (dots) and infinite flux boundary (solid line)

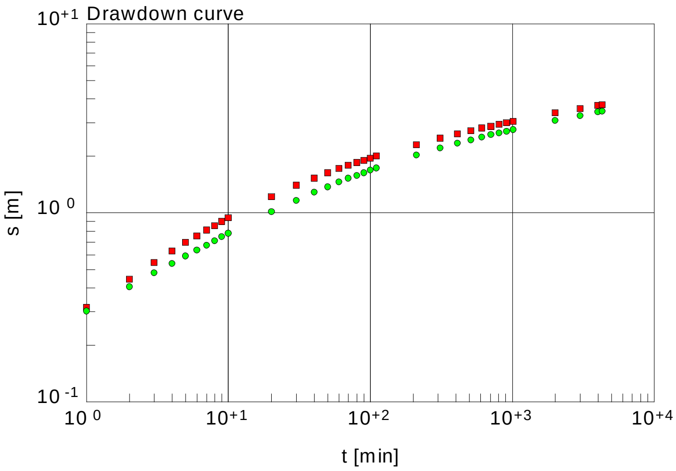


Fig. 2.42 Comparison of drawdown in an infinite conductive vertical fracture with uniform flux boundary (squares) and infinite flux boundary (dots)

The drawdown s in a fracture at early times is given by Gringarten *et al.* (1974) for both the uniform and infinite flux cases and reads:

$$s = \frac{Q}{2 \cdot \pi \cdot T} \cdot \sqrt{\pi \cdot t_d} \qquad (2.35)$$

with
$t_d < 10^{-2}$ for uniform flux
$t_d < 2 \cdot 10^{-1}$ for infinite flux

Eq. (2.35) describes the drawdown during the linear flow phase, which implies uniformity of flux along the fracture for both cases. Due to the fact that the fracture has infinite conductivity, there is an infinite small pressure gradient along the fracture that can be neglected ($\Delta p/\Delta x_f = 0$). Therefore, at early time the same drawdown is observed along the entire length of the fracture and an observation well located in the fracture will show the same curve as the pumped well.

The long term solution describes the radial-acting flow phase ($t_d > 5$) on the fracture as

$$s = \frac{Q}{2 \cdot \pi \cdot T} \cdot \left[\frac{1}{2}\ln(t_d) + 1.1\right] \qquad (2.36)$$

Eq. (2.36) plots as a straight line in a lin-log plot. Analogue to a homogeneous aquifer the transmissivity value for the formation can be obtained from the Cooper-Jacob approach, which will be demonstrated later.

The concept of relative storage capacity CD_f was introduced by Ramey and Gringarten (1976) to describe the influence of the fracture storage on the drawdown behaviour at early time and is defined as:

$$CD_f = \frac{S_f \cdot w}{S \cdot x_f} \qquad (2.37)$$

where
S = Storage coefficient of the matrix [-]
S_f = Storage coefficient of the fracture [-]
w = Fracture width or aperture [L]
x_f = Fracture half-length [L]

2.7.2.2 Diagnosis

The drawdown in a pumped well situated in a vertical fracture with infinite conductivity and finite extent is dominated by two different flow phases:

- Linear flow at early times, which shows a typical slope of 0.5 in a log-log plot (Fig. 2.43a) or a straight line in a lin $t^{½}$ plot (Fig. 2.43b).
- Radial flow after a transition period, which plots as a straight line in a lin-log plot (Fig. 2.43c).

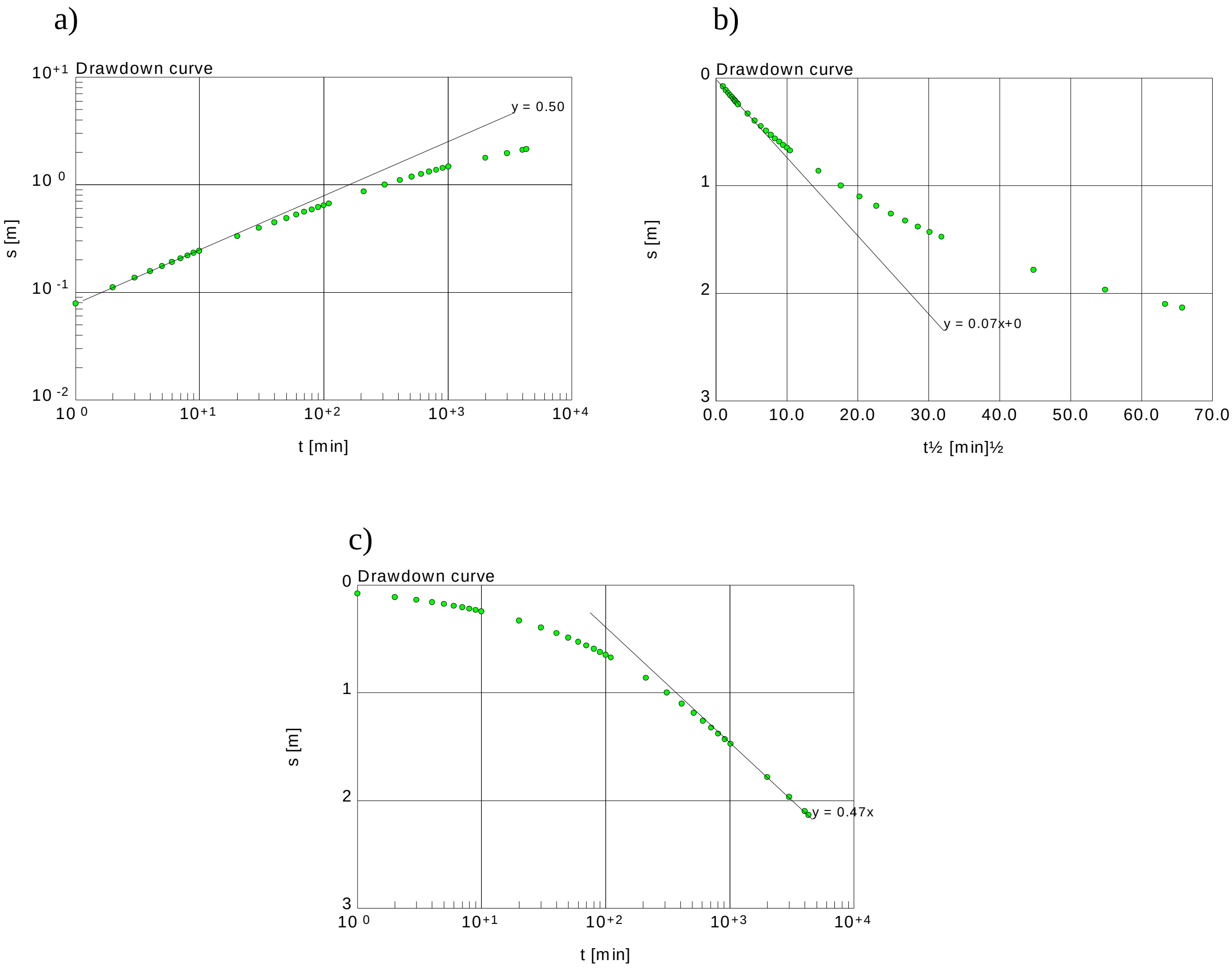


Fig. 2.43 Flow phases in various diagnosis plots

Unfortunately, drawdown data often plot on an almost horizontal line at early times due to skin effects at the well (well bore skin; Section 2.6.2), at the fracture interface (fracture skin; Section 2.6.4), or at both (Cinco-Ley and Samaniego, 1977). In such cases, the data plot as a straight line in a lin $t^{½}$ plot with a positive shift from the origin (Fig 2.44). The same plot can be used to uniquely determine the skin location (Bardenhagen, 1999), as shown in Fig. 2.45. Alternatively, the derivative of the curve can be used to determine the linear flow phase, due to the fact that it is not affected by the skin effects. The derivative will plot as a straight line with a slope of 0.5 in a log-log plot (Fig 2.46a) or as a straight line from the origin in a lin $t^{½}$ plot (Fig 2.46b).

Whether the radial-acting flow phase is present, can be determined using the derivative of a log-log plot, as it graphs horizontal once the radial-acting flow was reached. The data beyond the point where the radial-acting flow starts can be used for the estimation of the transmissivity, using common methods e.g. the Cooper-Jacob straight line method. If the radial-acting flow phase is not fully developed, the Gringarten type curve method should be applied, whose handling is basically the same as the common Theis type curve approach.

It must be borne in mind that no unique evaluation of the aquifer parameters is possible if only the linear flow phase is observed, as demonstrated in various recovery curves in Fig. 2.47.

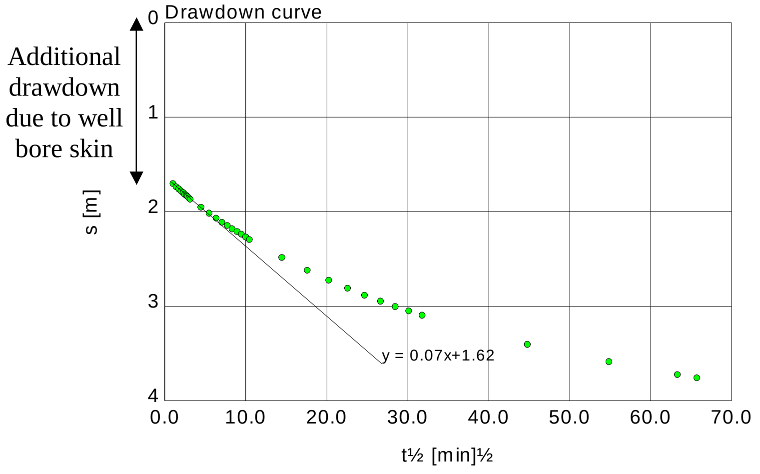


Fig. 2.44 Drawdown in an infinite conductive vertical fracture affected by skin indicated by a positive shift of the drawdown curve from the origin

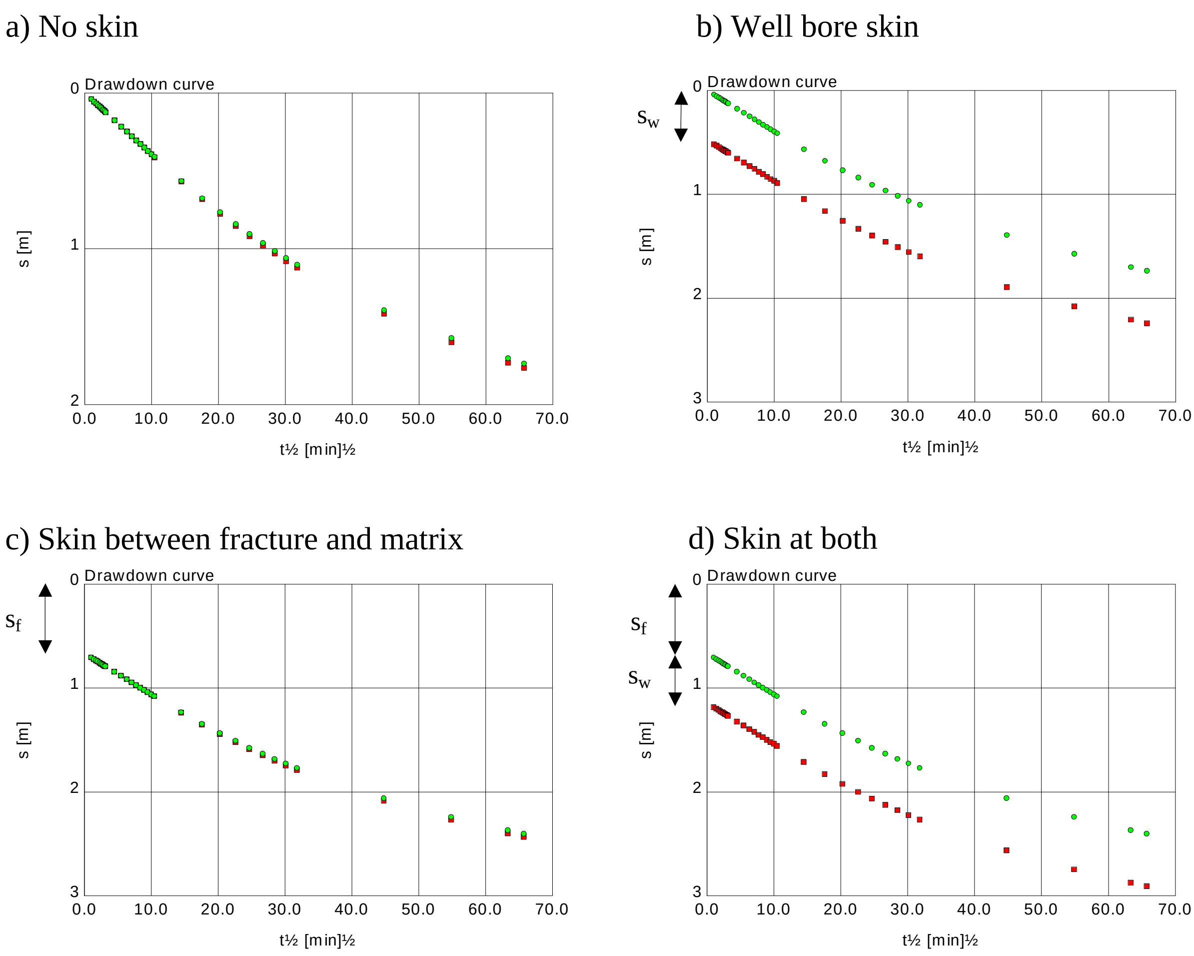


Fig. 2.45 Skin effects on drawdown curves from pumped well (squares) and observation well (dots) both located in the same fracture

a) b)

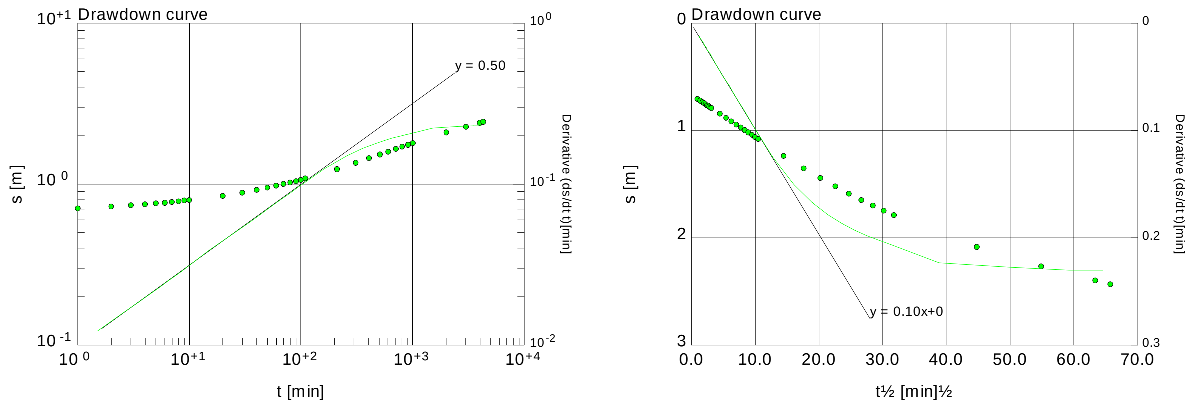


Fig. 2.46 The derivative is not affected by the skin effects and can therefore be used to determine the linear flow phase at early times

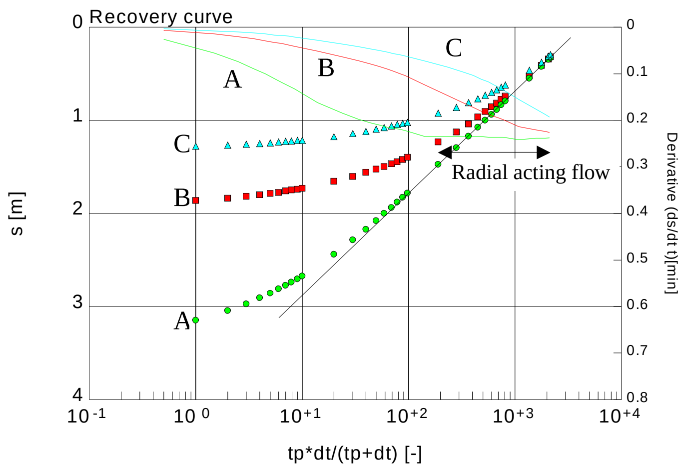


Fig. 2.47 The Agarwal straight line method can only be used for the determination of the aquifer transmissivity if the pumping time is long enough to allow for radial-acting flow as indicated by the horizontal derivative in curve A (solid line A). The pumping time in curves B and C was not long enough to reach the radial-acting flow and therefore the derivatives (solid lines B and C) are not horizontal

The influence of the fracture storage on the drawdown is described by the relative fracture storage coefficient CD_f. At early times the drawdown curves differ significantly according to the CD_f value. The early time drawdown for $CD_f = 10^{-4}$ does not plot on a straight line with slope 0.5 (Ramey and Gringarten, 1976). Indeed, between $10^{-4} \leq CDf \leq 10^{-2}$, the drawdown curves are not characterised by straight lines for times of interest. If $CD_f > 10^{-2}$ the data will plot as a straight line with slope 1, which indicates that the cone of depression has reached the edges of the very high conductive fracture (Fig. 2.48). Therefore the drawdown at early times is similar to that of pumping from a limited reservoir (closed boundaries). As soon as the gradient between matrix and fracture builds up, the influx from the matrix to the fracture increases and the slope of 1 vanishes, as in the well bore storage effect case.

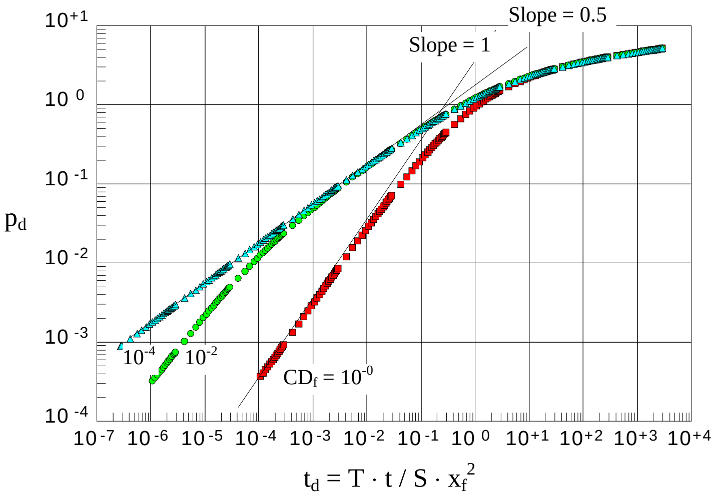


Fig. 2.48 Various dimensionless drawdown curves and their derivatives for different relative fracture storage capacity CD_f ($pd = 2 \cdot \pi \cdot T \cdot s/Q$)

2.7.2.3 Method of analysis

Basically, two methods of analysis for the drawdown data are used to determine the formation or matrix transmissivity T: a straight line method using a lin-log plot and a type curve method using a log-log plot. If recovery (build up) data are available, the recovery method of Theis (1935) or Agarwal (1979) can be used. Alternatively, a forward modelling using TPA can be applied to determine the aquifer parameter for both the drawdown and recovery phase. Note that the determination of the actual fracture transmissivity T_f is not possible because it is a priori considered infinite!

The methods can be applied if the following conditions are true:

- Matrix is infinite.
- Aquifer (fracture and matrix) is confined.
- Darcian flow prevails in fracture and matrix.
- Well and fracture penetrate the aquifer fully.
- Negligible well bore storage and fracture storage.
- Negligible well bore skin and fracture skin.
- Straight line can be applied if td > 5. Cross-check, where the first derivative becomes horizontal.

2.7.2.3.1 Straight line application

Straight line methods can be applied to both pumped well and observation well data, but only if a significant part of the curve shows radial-acting flow (see previous section). The handling is similar to that of the Cooper-Jacob (1947) method. The transmissivity T of the formation can be determined by the following equation:

$$T = \frac{0.183 \cdot Q}{s} \qquad (2.38)$$

where
T = Transmissivity of the formation or matrix [L^2T^{-1}]
Q = Discharge rate [L^3T^{-1}]
s = Drawdown over one logarithmic cycle [L]

It must be borne in mind that the common Cooper-Jacob approach for the determination of the storage coefficient is only applicable if the distance of the observation well to the pumped well is more than 5 times that of the fracture half-length x_f.

The Theis (1935) and Agarwal (1980) recovery methods are applicable for the determination of the formation transmissivity T if a significant portion of the recovery curve show radial-acting flow behaviour as demonstrated in Fig. 2.47.

The handling of the straight line recovery method is similar to the drawdown approach. The transmissivity of the formation can be determined by the following equation:

$$T = \frac{0.183 \cdot Q}{s'} \qquad (2.39)$$

where
s' = residual drawdown over one logarithmic cycle [L]

2.7.2.3.2 Type curve application

The advantage of the Gringarten type curve approach lies in the fact that only data of the transient phase from linear flow to radial-acting flow is needed (Fig. 2.49). In other words, the method can be used if a test was run too short to fully reach the radial-acting flow phase.

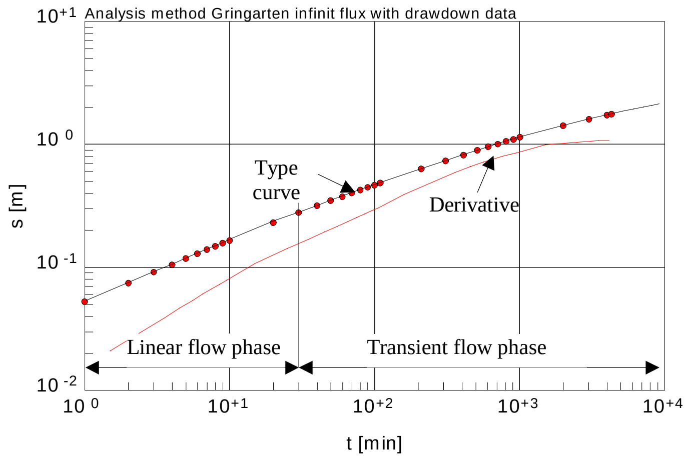


Fig. 2.49 Example of the Gringarten type curve method for a data set (dots) that does not reach the radial-acting flow

The application of Gringarten's type curve method for pumped wells is similar to that of the Theis type curve method (Fig. 2.49). After matching the data curve with the type curve, the transmissivity T and storage coefficient S can be determined by substituting the values for the match point co-ordinates as follows:

$$T = \frac{Q}{2 \cdot \pi \cdot s} \cdot p_d(0,0,t_d) \qquad (2.40)$$

$$S = \frac{T \cdot t}{t_d \cdot x_f^{\,2}} \qquad (2.41)$$

The fracture half-length x_f is usually an unknown parameter, which can only be determined if data from an observation well are available and its relative location to the fracture is known. Unfortunately, for each location of an observation well a set of type curves have to be drawn, which is not very effective. In such cases, the forward modelling represents a more appropriate approach (Fig. 2.50).

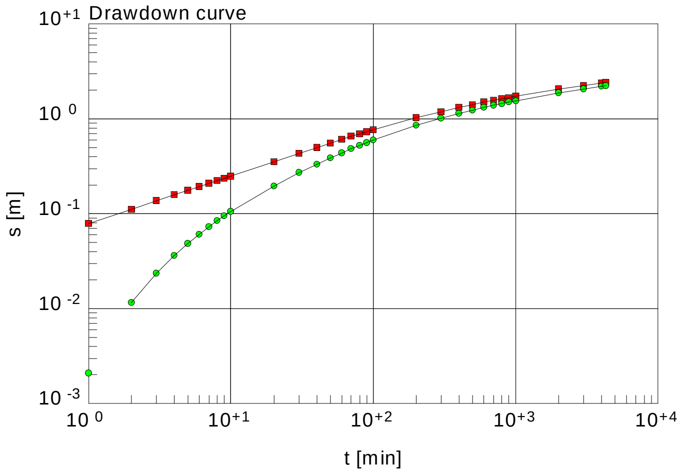


Fig. 2.50 **Example of Gringarten forward simulation (solid lines) for a pumped well (squares) and an observation well (dots). Simulation parameters are: T = 50 m^2/d; S = 0.0001; x_f = 400 m; distance between pumped and observation wells d = 50 m (perpendicular to the fracture)**

2.7.2.3.3 Determination of skin effects

Skin effects can appear at the well (well bore skin), between fracture and matrix (fracture skin) or on both as described in Bardenhagen (1999). If drawdown data from wells situated in a vertical fracture with infinite conductivity are obscured by skin effects, the curve shows an almost horizontal drawdown at the early time data followed by the radial-acting flow period after a transition zone. Fig. 2.51 shows the various possible drawdown shapes graphed in log-log plots. The total skin factor ξ_t can be graphically determined using the pumped well data represented in a lin $t^{½}$ plot (Fig. 2.45d) and the following equation:

$$\xi_t = \frac{s_{add} \cdot 2 \cdot \pi \cdot T}{Q} \qquad (2.42)$$

where

s_{add} = additional drawdown = $s_w + s_f$ [L]
T = transmissivity of the matrix [L^2T^{-1}]
Q = discharge rate [L^3T^{-1}]

In the presence of an observation well located in the same fracture, the additional drawdown caused by fracture skin ξ_f can be determined after Eq. (2.42) similarly to ξ_t. The well bore skin ξ is then calculated as

$$\xi = \xi_t - \xi_f \qquad (2.43)$$

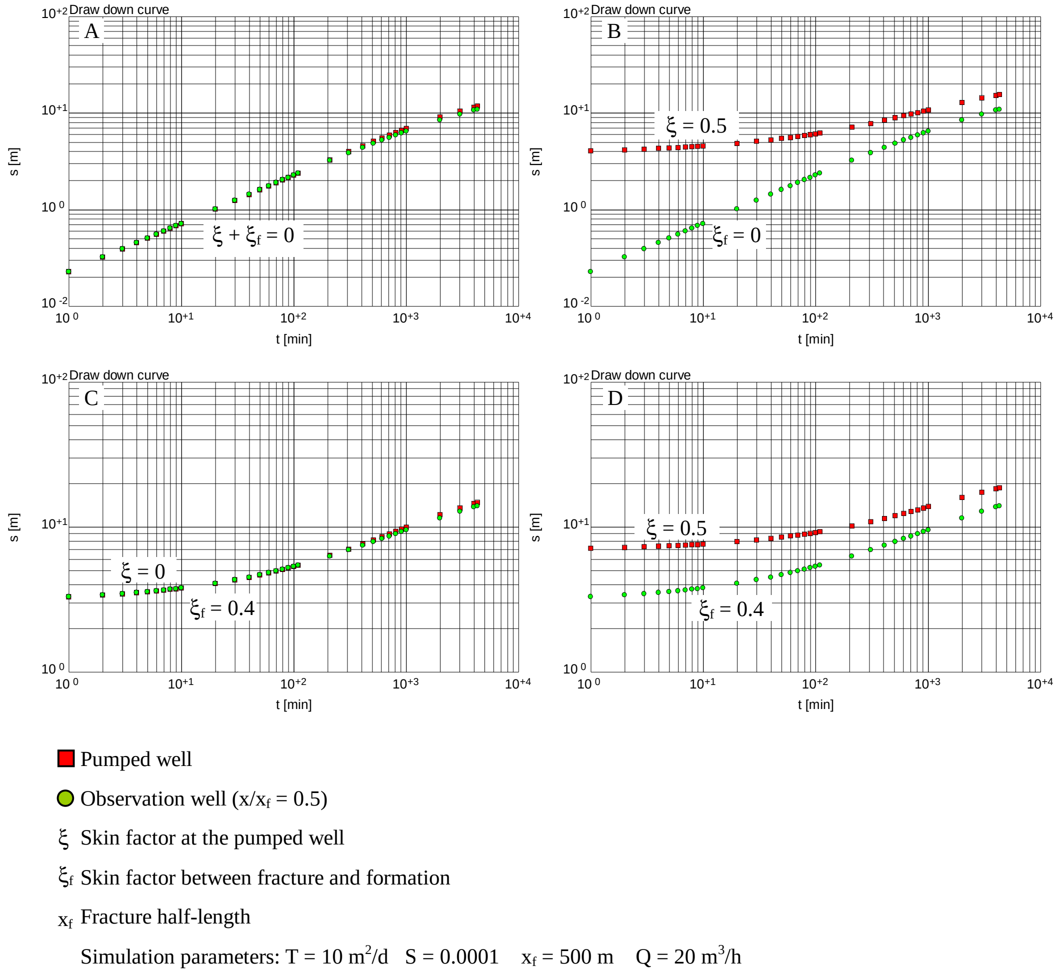


Fig. 2.51 Skin effect at pumped well and observation well located in the same infinite conductive fracture

2.7.2.3.4 Forward modelling application

If the transmissivity value is known either from the straight line approach or the type curve approach, this value should be used as a known parameter for the forward modelling to shorten the time necessary to fit the unknown parameter. In the worst case the known parameters are:

- Hydrogeological concept of a single vertical fracture with infinite transmissivity.
- Transmissivity T of the matrix.
- Skin factor $\xi_{f.}$

The unknown parameters are

- Storage coefficient S of the matrix.
- Fracture half-length $x_{f.}$

However, these parameters cannot be uniquely determined without data of an observation well located in the matrix.

2.7.2.4 Field example

<u>First example: Skin effect in well bore</u>

Two boreholes located 133.5 m apart were sited on a 15 km long, subvertical (77°S) fault zone crossing the Fish River in the southern part of Namibia, which might be a potential recharge source. The fault partly separates two low-yielding formations composed by horizontal intercalated layers of claystone, siltstone and sandstone. Both boreholes intersect the fault at 27 m below the surface. The water level in both boreholes rose immediately after the fault was struck at a level of 906.1 mamsl (8.3 m below surface in BH1 and 5.3 m below surface in BH2). The airlift yield was estimated at more than 100 m^3/h in each borehole. Screens with 0.5 mm slots were installed to avoid borehole collapse. Fig. 2.52 shows the drawdown measured during one of the constant discharge tests. Only the drawdown in the observation well shows a slope of 0.5 indicating linear formation flow. However, the drawdown in the pumped well starts almost horizontal and develops at late time to radial-acting flow. This behaviour is typical for a skin that is located at the well.

Discharge rate [m^3/h]	Q	= 67
Fracture transmissivity [m^2/d]	T_f	= 200
Fracture storage coefficient [-]	S_f	= 0.0007
Fracture half-length [m]	x_f	= 460
Well bore skin factor [-]	ξ	= 1.78
Fracture skin factor [-]	ξ_f	= 0
Drilled well bore radius [m]	r_w	= 0.11
Distance of the observation well [m]	r	= 133.5

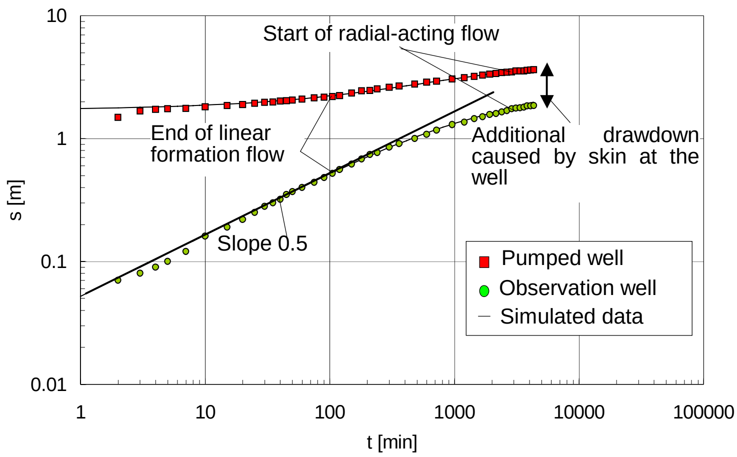


Fig. 2.52 Example for a restricted drawdown in a pumped well. The slope of 0.5 in the drawdown data of the observation well indicates linear formation flow and simulation for a vertical infinite conductive fracture with uniform flux

Second example: Well bore skin and fracture dewatering effects

A borehole was drilled into the dolomites of the Tsumeb Karst area, Namibia. The constant discharge test indicated a short period of linear flux to a fracture in the early stages. Water strikes were recorded at depths between 14 m and 27 m below the water table. As soon as the water level in the pumped well dropped to the level of the first water strike, a significant increase in the drawdown was measured with a further increase as the water level dropped below to the second strike (Fig. 2.53). This behaviour is a clear indication for over-abstraction at a rate of 20 m^3/h. However, due to unknown reasons, an additional drawdown of 7 m can be determined from the special plot. The determination of the aquifer parameters (T and S) for this test is not possible with any of the above-mentioned methods because the transient and radial-acting flow phases are masked by the effects of fracture dewatering.

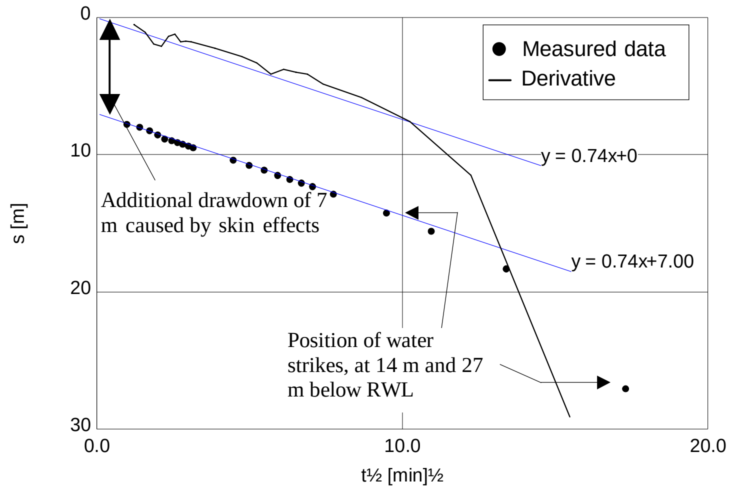


Fig. 2.53 Graphical skin evaluation using the linear flow period of drawdown curve

2.7.3 Single vertical fracture with finite conductivity and finite extent (Cinco-Ley and Samaniego, 1978)

2.7.3.1 Theory

Cinco-Ley *et al.* (1978) introduced a semi-analytical model that describes the drawdown in a single vertical fracture with finite conductivity and length, which is embedded in an infinite, isotropic, homogeneous, horizontal matrix limited by upper and lower impermeable boundaries. It considers bilinear flow in the system and is therefore a generalised solution for this type of aquifer's geometry. The Gringarten *et al.* (1974) infinite flux solution is a special case of this model.

The gradient along the fracture cannot be neglected due to its finite conductivity and the solution requires the knowledge of the flux distribution along the fracture in time. However, the flux distribution stabilises after a certain period of time that coincides with the start of the radial-acting flow phase (Fig. 2.54). This stable distribution is known as stabilised flux distribution.

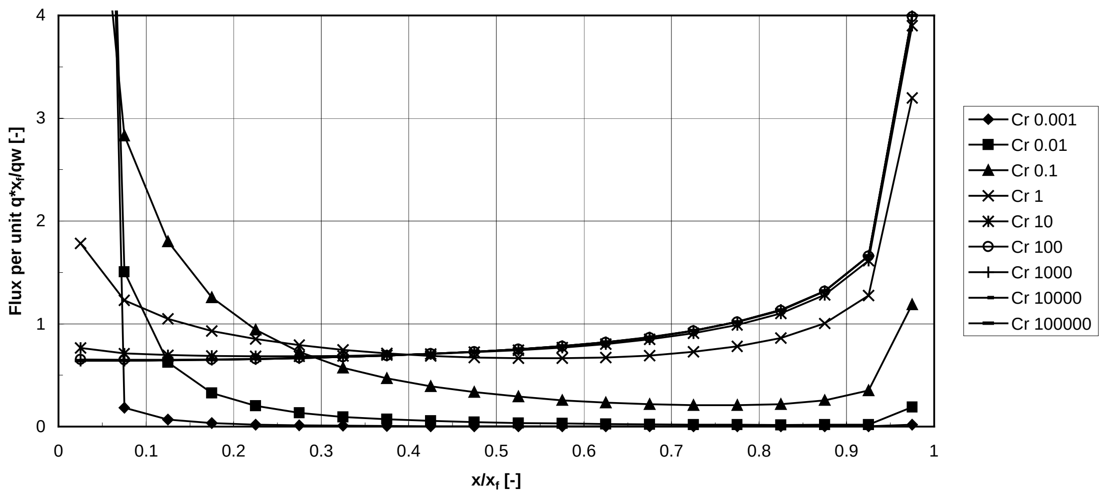


Fig. 2.54 Stabilized flux distribution for different relative conductivities Cr. The flux distribution does not change for values of Cr ≥ 100

Assuming that the drawdown distribution in the fracture (Eq. 2.44) coincides with the drawdown distribution in the matrix-fracture interface (Eq. 2.45), the matrix to fracture flux distribution can be obtained by simultaneously solving both equations. The model developed by Cinco-Ley *et al.* (1978) uses a special form of the finite difference method to obtain this distribution. The drawdown is obtained using the calculated fluxes in either Eq. (2.44) or (2.45). The equations for the fracture and reservoir are derived using the Green and source functions and the Newman product method. They read:

$$p_{fd}(x_d,t_d) = \frac{x_f}{w}\sqrt{\frac{\pi \cdot S \cdot T \cdot}{S_f \cdot T_f}} \cdot \sum_{n=1}^{\infty}\int_0^{t_d}\left\{\frac{e^{-\left[\frac{(x_d-2n)^2}{4(T_f S/TS_f)\tau}\right]}}{\sqrt{\tau}} - \int_{2n-1}^{2n+1} q_{fd}(x',\tau)\frac{e^{-\left[\frac{(x_d-x')^2}{4(T_f S/TS_f)\tau}\right]}}{2\sqrt{t_d-\tau}}dx'\right\}d\tau \qquad (2.44)$$

$$p_d\left(x_d, y_d, t_d\right) = \frac{1}{4}\int_0^{t_d}\int_{-1}^{1} q_d\left(x', \tau\right)\frac{e^{-\left[\frac{(x_d - x')^2 + y_d^2}{4(t_d - \tau)}\right]}}{t_d - \tau}\,dx'd\tau \qquad (2.45)$$

where
p_{fd} = dimensionless drawdown in the fracture [L]
x_d = x/x_f [-]
y_d = y/x_f [-]
t_d = $T \cdot t/S \cdot x_f^2$ [-]
w = fracture width or aperture [L]
T = matrix transmissivity [L^2T^{-1}]
T_f = fracture transmissivity [L^2T^{-1}]
S = matrix storage coefficient [-]
S_f = fracture storage coefficient [-]
q_d, q_{fd} = dimensionless flux from matrix to the fracture [-]
n = integer variable
τ, x' = integration variable

The solution uses the concept of relative conductivity Cr to relate the conductivities of fracture and matrix as follows:

$$Cr = \frac{T_f \cdot w}{\pi \cdot T \cdot x_f} \qquad (2.46)$$

where
T_f = fracture transmissivity [L^2T^{-1}]
w = fracture width or aperture [L]
T = matrix transmissivity [L^2T^{-1}]
x_f = fracture half-length [L]

Cinco-Ley *et al.* (1978) presented a series of type curves for the pumped well for Cr values in the range of 0.1 to 100 and dimensionless time t_d between 10^{-3} and 10^{3}. The curve that corresponds to Cr ≥ 100 practically coincides with the infinite flux solution from Gringarten *et al.* (1974). Agarwal *et al.* (1979) extended these type curves to smaller dimensionless time values ($t_d = 10^{-5}$) based on numerical results obtained with the finite difference method.

In general, the influence of the fracture storage capacity on the drawdown behaviour can be described by the relative fracture storage capacity CD_f (Eq. (2.37), Section 2.7.2.1) as shown by Cinco-Ley *et al.* (1978). However, they proved that this influence could be neglected for the practical values of dimensionless time t_d and relative fracture storage capacity CD_f (Guppy *et al.*, 1982).

2.7.3.2 Diagnosis

The drawdown in a pumped well situated in a vertical fracture with finite conductivity and finite length is characterized by the relative conductivity Cr (Cinco-Ley and Samaniego, 1981a) as follows:

- Linear flow at very early time, which shows a typical slope of 0.5 in a log-log plot or a straight line in a lin $t^{½}$ plot. It appears for Cr < 100, but is usually masked by well bore storage.

- Bilinear flow at early times, which shows a typical slope of 0.25 in a log-log plot (Fig. 2.55a) or a straight line in a lin $t^{1/4}$. It is observed for Cr < 100.
- Linear formation flow at intermediate time, which shows a typical slope of 0.5 in a log-log plot (Fig. 2.55a) or a straight line in a lin $t^{1/2}$ plot. It develops if Cr ≥ 100.
- Radial flow at late time, which plots as a straight line in a lin-log plot (Fig. 2.55b). It appears for all Cr, if the discharge time is sufficiently long (t_d >5).
  The drawdown curves show transition zones between all the different flow phases.

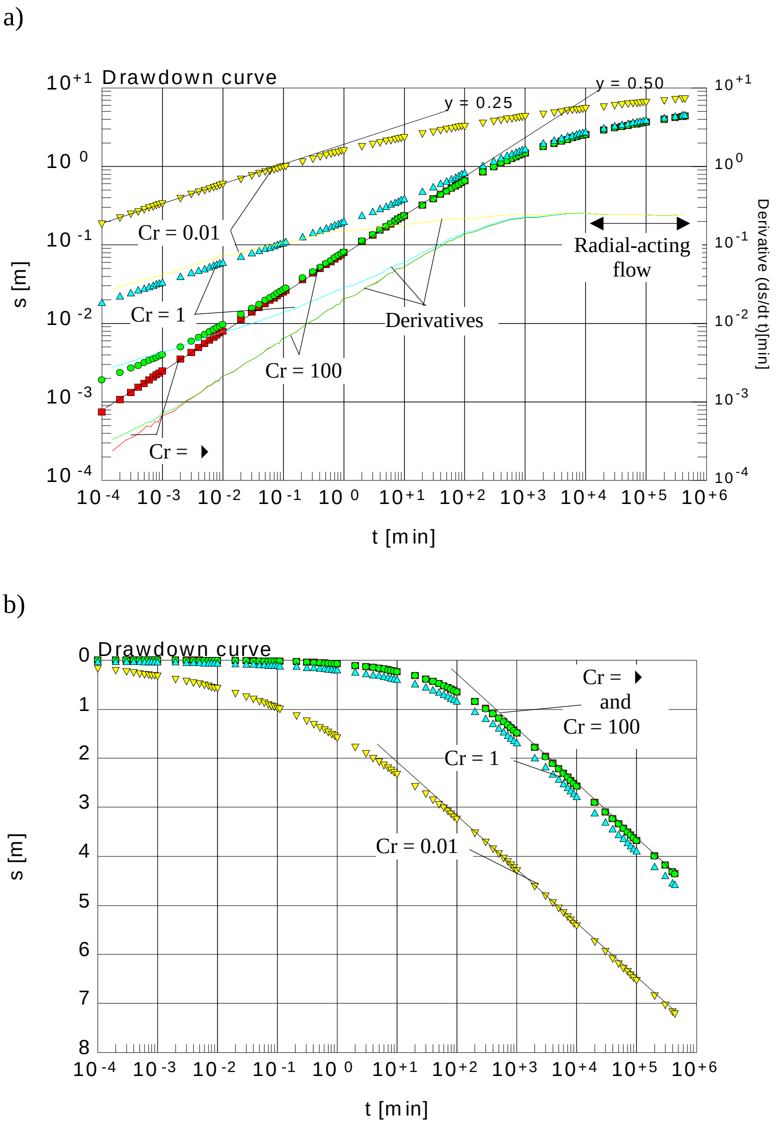


Fig. 2.55 **Drawdown curves for various relative conductivities Cr from wells located in a vertical finite conductive fracture**

The fracture has a finite conductivity and therefore the reaction in an observation well located in the same fracture as the pumped well is not instantaneous, like in the infinite conductive fracture case (Gringarten *et al.*, 1974). As a result, at early times the shape of the drawdown in the observation well is not similar to that of the pumped well (Fig. 2.56).

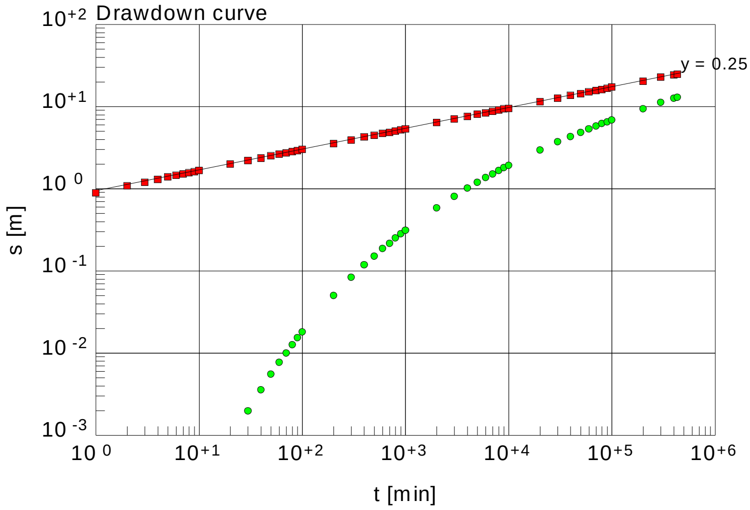


Fig. 2.56 **Drawdown in a pumped well (squares) and an observation well (dots) both located in the same vertical finite conductive fracture. The curves differ at early time, due to the finite conductivity of the fracture**

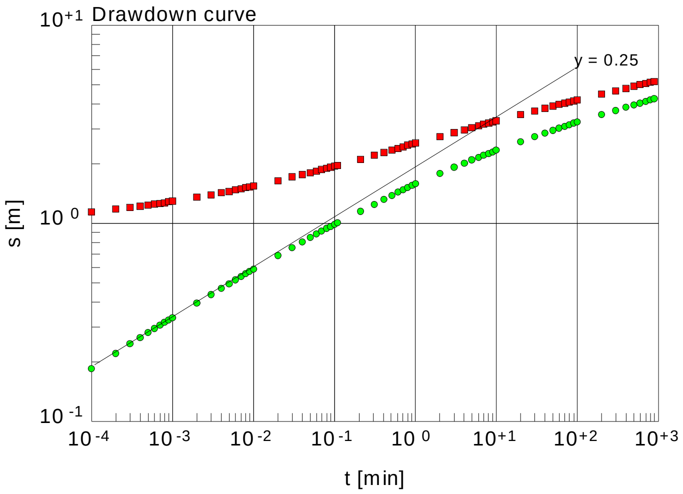
Whenever affected by skin, the drawdown at the pumped well develops similarly to the infinite conductive fracture case. In a log-log plot, it graphs initially as a horizontal straight line and after a transition period, it shows the normal radial flow shape (Fig. 2.57a). In a lin $t^{1/4}$ plot, the early time bilinear flow plots as a straight line with a positive shift from the origin (Fig. 2.57b). Additionally, the derivative of the drawdown curve can be used to determine the different flow phases, as it is not affected by skin.

Due to the finite conductivity of the fracture it is not possible to graphically determine the location of the skin as described in Section 2.7.2.2 for the infinite conductive fracture case.

In cases where the storage capacity of the fracture cannot be neglected, the influence of this parameter on the drawdown behaviour can be described by the relative fracture storage capacity CD_f (Eq. (2.37), Section 2.7.2.1). In log-log plots the effects (Fig. 2.58) are as follows:

- For values of $CD_f > 10^{-4}$,the bilinear flow at early time data does not plot on a straight line with slope 0.25.
- For values between $10^{-4} \leq CDf \leq 10^{-2}$,the drawdown curves cannot be characterised by any straight line as they only show transient flow behaviour.
- For values of $CD_f > 10^{-2}$,the data plot as a straight line with slope 0.5.

a)



b)

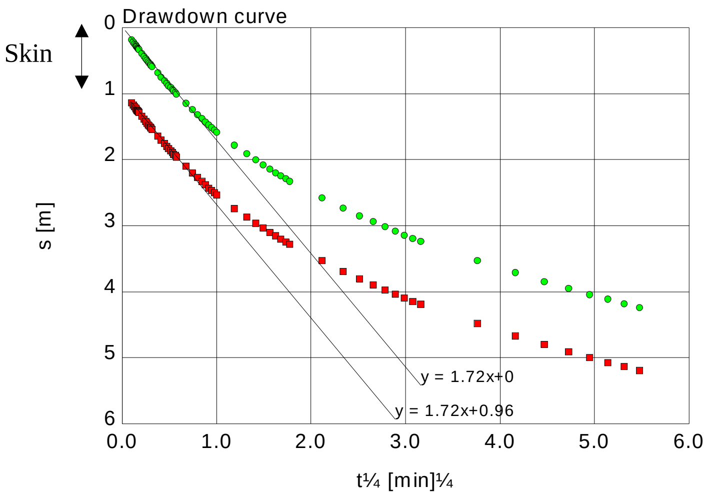


Fig. 2.57 **Skin effects on drawdown curves from the pumped well (squares) and observation well (dots) both located in the same finite conductive fracture**

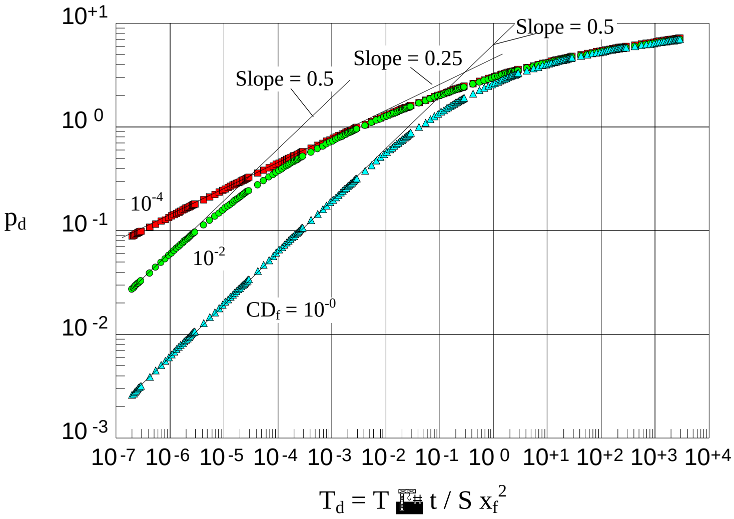


Fig. 2.58 Various dimensionless drawdown curves and their derivatives for different relative fracture storage capacity CD_f ($pd = 2 \cdot \pi \cdot T \cdot s/Q$)

2.7.3.3 Method of analysis

Using the Cinco-Ley and Samaniego model (1978) in those cases where bilinear flow and radial-acting flow are present (Cr < 100), the following parameters can be determined:

- Reservoir transmissivity.
- Reservoir storage coefficient.
- Fracture half-length.
- Fracture transmissivity.
- Fracture storage coefficient.

Basically, two methods of analysis for the drawdown data are used to determine these parameters: a straight line method using a lin-log plot and a type curve method using a log-log plot. If recovery (build up) data are available, the recovery method of Theis (1935) or Agarwal (1980) can be utilised. Alternatively, a forward modelling using TPA can be applied to determine the aquifer parameter for both the drawdown and recovery phase in the pumped well.

The methods can be applied if the following conditions are true:

- Matrix is infinite.
- Aquifer (fracture and matrix) is confined.
- Darcian flow prevails in fracture and matrix.
- Well and fracture penetrate the aquifer fully.
- Negligible well bore storage and fracture storage.
- Negligible well bore skin and fracture skin.
- Straight line can be applied if td > 5. Crosscheck, where the first derivative becomes horizontal.

2.7.3.3.1 Straight line application

The straight line application allows the determination of the reservoir transmissivity using the radial-acting flow of drawdown or recovery data in a lin-log plot for pumped well and/or observation well data. The approach is similar to the methods of Cooper-Jacob (1947) and Theis (1935). The transmissivity T of the formation can be determined using Eq. (2.38) (Section 2.7.2.3.1).

It must be borne in mind that the common Cooper-Jacob approach for the determination of the storage coefficient is only applicable, if the distance of the observation well to the pumped well is bigger than 5 times that of the fracture half-length x_f .

The Theis (1935) and Agarwal (1980) recovery methods are applicable for the determination of the formation transmissivity T if a significant portion of the recovery curve shows radial-acting flow behaviour (Section 2.7.2.2). The handling of the straight line recovery method is similar to the drawdown approach. The transmissivity of the formation can be determined using Eq. 2.39 (Section 2.7.2.3.1).

2.7.3.3.2 Type curve application

The advantage of the Cinco-Ley *et al.* type curve approach lies in the fact that only data of the transient phase from linear flow to radial-acting flow are needed. In other words, the method can be used even if the test did not fully reach the radial-acting flow phase.

The application of Cinco-Ley *et al.* type curve method for a pumped well is similar to that of the Theis type curve method. After matching the data curve with the type curve, the transmissivity T and storage coefficient S of the formation are calculated by substituting the values for the match point co-ordinates in Eqs. (2.45) and (2.46) (Section 2.7.2.3.2). The evaluation of the storage coefficient requires the knowledge of the fracture half-length x_f, which can only be determined if data from at least one observation well are available and the relative location to the fracture is known.

The transmissivity of the fracture T_f can be determined using the transmissivity of the formation and estimated values for the fracture's aperture and half-length in the equation for the relative conductivity Cr (Eq. 2.46). The storage coefficient of the fracture S_f can only be uniquely determined using Eq. (2.37) if the relative storage capacity $CD_f > 10^{-4}$.

2.7.3.3.3 Determination of skin effects

Skin effects can appear at the well (well bore skin) between fracture and matrix (fracture skin) or on both (Cinco-Ley and Samaniego, 1981b). If drawdown data from wells situated in a vertical fracture with finite conductivity are obscured by skin effects, the curve shows an almost horizontal drawdown at the early time data followed by the radial-acting flow period after a transition zone. The total skin factor ξ_t can be graphically determined using the pumped well data represented in a lin $t^{1/4}$ plot (Fig. 2.57) and Eq. 2.42 (Section 2.7.2.3.3).

2.7.3.3.4 Forward modelling application

If the transmissivity value is known either from the straight line approach or the type curve approach, this value should be used as the known parameter in the forward modelling to shorten the time necessary to fit the unknown parameters. The model implemented in TPA (Fig. 2.59) simplifies the model presented by Cinco-Ley *et al.* (1978) by neglecting the influence of the fracture's storage coefficient. It is considered that, for the practical times this parameter does not influence the drawdown behaviour. Furthermore, at this stage, TPA is not able to model the drawdown of observation wells and therefore the unique evaluation of the unknown parameters is not viable.

In the worst case, the known parameters are

- Hydrogeological concept of a single vertical fracture with finite transmissivity.
- Transmissivity T of the matrix.
- Skin factor ξ_{t}.

The following unknown parameters must be estimated

- Storage coefficient S of the matrix.
- Transmissivity T_f of the fracture.
- Fracture width or aperture w.
- Fracture half-length x_f.

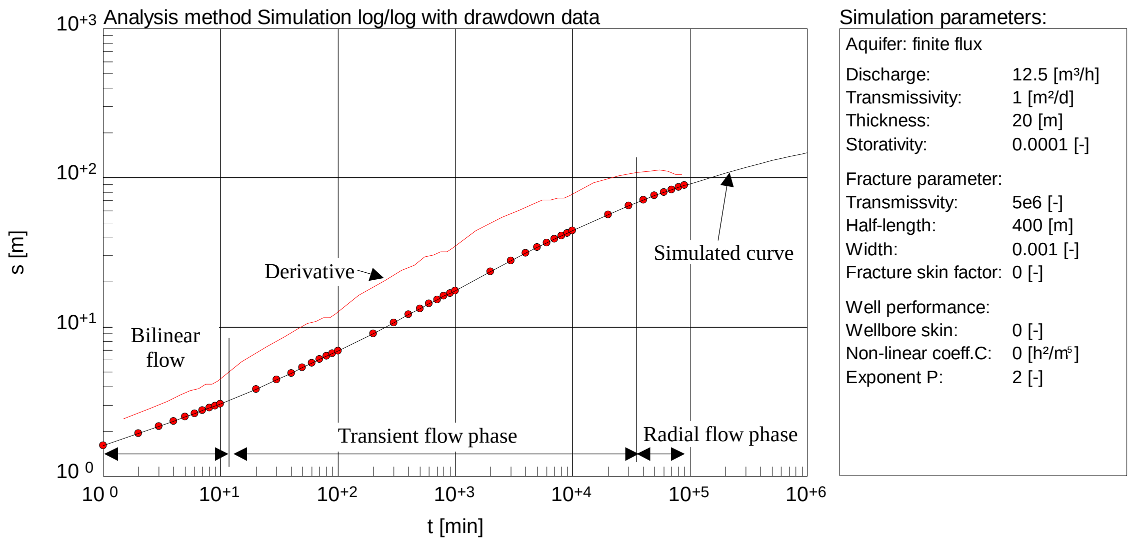


Fig. 2.59 Example of the Cinco-Ley *et al.* forward modelling for a data set (dots) that does not reach a fully radial-acting flow phase

2.7.3.4 Field example (Cinco-Ley et al., 1978)

Cinco-Ley *et al.* published a pumping test performed in an oil well with the following characteristics:

Discharge rate [m^3/h]	Q	= 2.5
Oil specific weight [kg/m^3]	ρ	= 0.9
Oil viscosity [cp]	μ	= 0.85
Formation transmissivity [m^2/d]	T	= 0.042
Formation storage coefficient [-]	S	= 0.0002
Fracture transmissivity [m^2/d]	T_f	= 260
Well bore skin factor [-]	ξ	= 0
Fracture skin factor [-]	ξ_f	= 0
Fracture width [m]	w	= 0.02
Fracture half-length [m]	x_f	= 48

The test was modelled using forward modelling and the above listed parameters. The test evaluation is presented in Fig. 2.60.

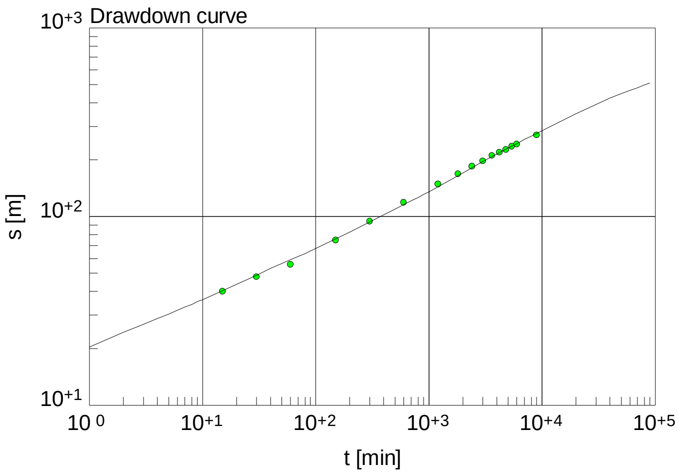


Fig. 2.60 **Example of the Cinco-Ley *et al.* pumping test evaluation for Cr = 0.82. The dots represent the data and the solid line the modelled curve**

2.7.4 Single vertical dyke with finite conductivity and infinite extent (Boonstra and Boehmer, 1986)

2.7.4.1 Theory

Boonstra and Boehmer (1986) introduced a semi-analytical model that describes the drawdown in a single vertical feature with finite conductivity, infinite length and a considerable uniform width. The feature is embedded in an infinite, isotropic, homogeneous, horizontal matrix limited by upper and lower impermeable boundaries. The model considers linear flow at early times and bilinear flow at intermediate times. Although originally presented to analysed drawdown in dykes, the model can be utiliszed to model drawdown in vertical fractures with significant width.

The equation presented by Boonstra and Boehmer (1986) reads:

$$p(\chi,t_d)=\frac{2}{\sqrt{\pi}}e^{-2\sqrt{t_d}}\int_0^{\sqrt{t_d}} e^{\left[2\sqrt{t_d-r^2}-\frac{\chi^2}{4r^2}\right]}dr \qquad (2.47)$$

where

p = dimensionless drawdown in the feature [L]
χ = $(2n/m^{1/2})\cdot x$ [-]
n = $\alpha\cdot(S\cdot T)^{1/2}/w\cdot S_f$ [$T^{-1/2}$]
m = T_f/S_f [L^2T^{-1}]
x = distance between the pumped well and observation point along the feature[L]
α = dmpirical parameter (generally α = 0.94) [-]
T = matrix transmissivity [L^2T^{-1}]
T_f = feature transmissivity [L^2T^{-1}]
S = matrix storage coefficient [-]
S_f = feature storage coefficient [-]
w = feature width [L]
t_d = $4\cdot n^2\cdot t$ [-]
r = Integration variable

Eq. (2.47) is valid up to a pumping time:

$$t \approx 0.28\frac{S\cdot\left(w\cdot T_f\right)^2}{4T^3} \qquad (2.48)$$

For larger pumping times, the flow in the formation deviates from the parallel flow and develops gradually into the radial-acting flow.

Boonstra and Boehmer (1986) demonstrated that for small values of dimensionless time (t_d < 0.003). Eq. (2.47) at the pumped well reduces to the form:

$$p(0,t_d)=\frac{2}{\sqrt{\pi}}\sqrt{t_d} \qquad (2.49)$$

that coincides with the Cinco-Ley and Samaniego (1981) solution for linear fracture flow.

For large values of the dimensionless time (t_d > 100), Eq. (2.47) at the well reduces to the form:

$$p(0,t_d) = \sqrt[4]{t_d} \qquad (2.50)$$

which for α = 0.82 is identical to the equation derived by Cinco-Ley and Samaniego (1978) for the bilinear flow case.

2.7.4.2 Diagnosis

The drawdown curve in a pumped well situated in a vertical feature with finite conductivity, infinite length, and considerable width presents the following flow periods:

- Linear flow at early times, which shows a typical slope of 0.5 in a log-log plot (Fig. 2.61a) or a straight line in a lin $t^{½}$ plot.
- Bilinear flow at intermediate times, which shows a typical slope of 0.25 in a log-log plot (Fig. 2.61a) or a straight line in a lin $t^{¼}$.
- Radial flow at late times, which plots as a straight line in a lin-log plot (Fig. 2.61b).

The drawdown curves show transition zones between all the different flow phases.

In the Boonstra and Boehmer model, the feature has a finite conductivity and therefore the reaction in an observation well located in the same feature as the pumped well is not instantaneous like in the infinite conductive fracture case (Gringarten *et al.*, 1974). At early time the shape of the drawdown in the observation well is not similar to that of the pumped well, like in the Cinco-Ley *et al.* model (Fig. 2.61).

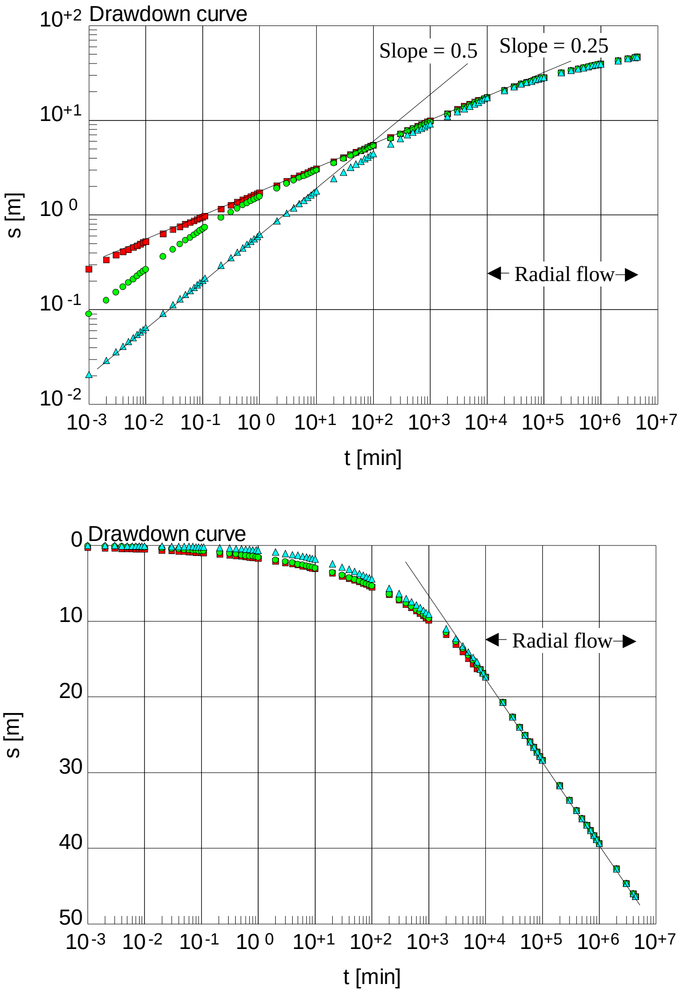


Fig. 2.61 Different flow periods in drawdown curves from wells located in a vertical finite conductive feature with infinite length and considerable width

Whenever affected by skin, the drawdown at the pumped well develops similarly to the infinite conductive fracture case. In a log-log plot, it graphs initially as a horizontal straight line and after a transition period, it shows the normal radial flow shape (Figs. 2.62a and 2.62b). If the early time data shows linear formation flow (slope of 0.5 in a log-log plot), the early time data graphs as a straight line with a positive shift from the origin in a lin $t^{½}$ plot (Fig. 2.62a). The bilinear flow at early time data (slope of 0.25 in a log-log plot) graphs as a straight line with a positive shift from the origin in a lin $t^{¼}$ (Fig 2.62b). Additionally, the derivative of the drawdown curve can be used to determine the different flow phases, as this curve is not affected by skin.

Due to the finite conductivity of the fracture it is not possible to graphically determine the location of the skin as described in Section 2.7.2.2 for the infinite conductive fracture case.

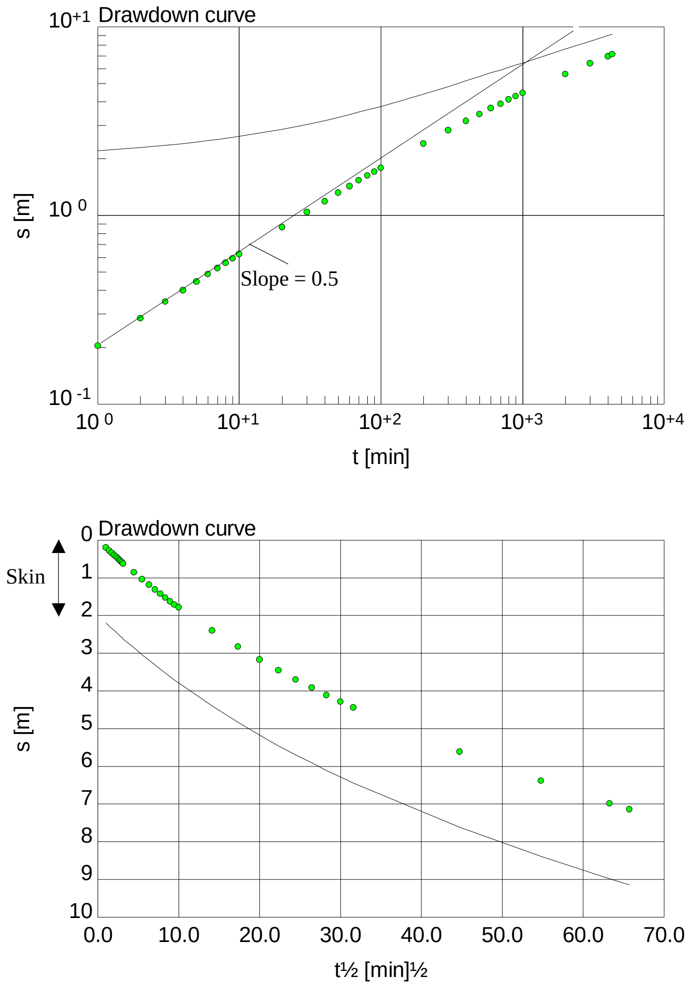


Fig. 2.62a Skin effects on drawdown curves from the pumped well (squares) and observation well (line) with linear flow at early time data. Both wells are located in the same finite conductive feature with finite length and considerable width

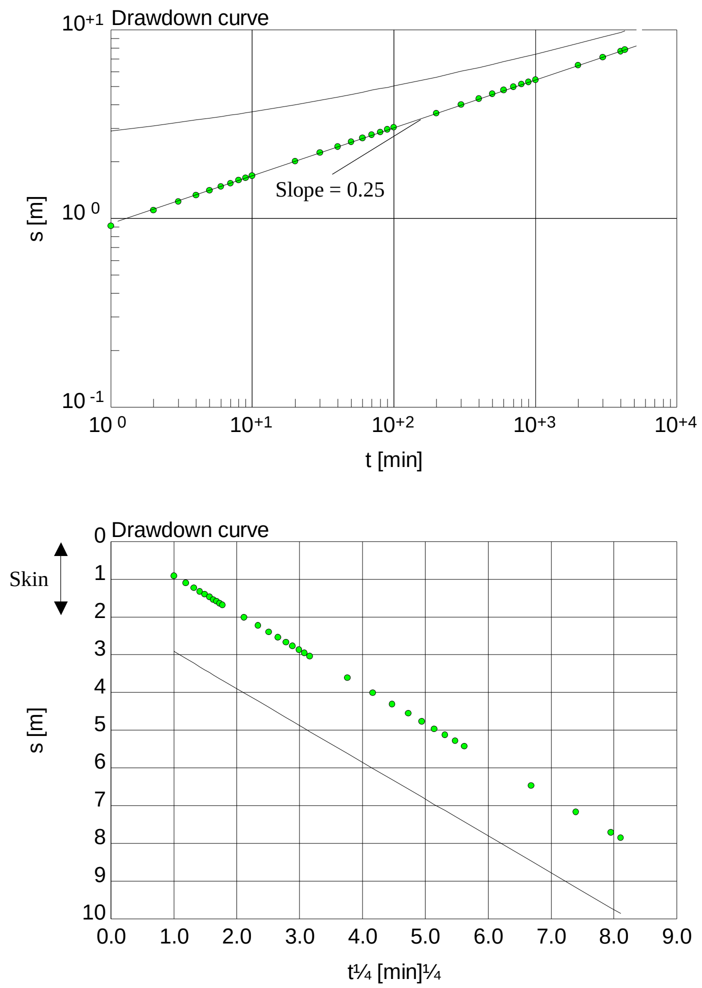


Fig. 2.62b **Skin effects on drawdown curves from the pumped well (squares) and observation well (line) with bilinear flow at early time data. Both wells are located in the same finite conductive feature with finite length and considerable width**

2.7.4.3 Method of analysis

Using the Boonstra and Boehmer model (1986), the following parameters can be determined if the feature width is known:

- Reservoir transmissivity.
- Reservoir storage coefficient.
- Feature transmissivity.
- Feature storage coefficient.

Basically, two methods of analysis for the drawdown data are used to determine these parameters: a straight line method using a lin-log plot and a type curve method using a log-log plot. If recovery (build up) data are available, the recovery method of Theis (1935) or Agarwal (1980) can be utilised. Alternatively, a forward modelling using TPA can be applied to determine the aquifer parameter for both the drawdown and recovery phase in the pumped well.

The methods can be applied if the following conditions are true:

- Matrix is infinite.
- Aquifer (feature and matrix) is confined.
- Darcian flow prevails in feature and matrix.
- Well and feature penetrate the aquifer fully.
- Negligible well bore storage.
- Negligible well bore skin and feature skin.
- Straight line method can be applied only in the radial-acting flow period. Crosscheck, where the first derivative becomes horizontal.

2.7.4.3.1 Straight line application

The straight line application allows the determination of the reservoir transmissivity T using the radial-acting flow of drawdown or recovery data in a lin-log plot for pumped well and/or observation well data. The approach is similar to the methods of Cooper-Jacob (1947) and Theis (1935). The transmissivity T of the formation can be determined using Eq. (2.37) (Section 2.7.2.3.1).

The straight line method cannot be applied for the estimation of the reservoir storage coefficient S.

The Theis (1935) and Agarwal (1980) recovery methods are applicable for the determination of the formation transmissivity T if a significant portion of the recovery curve shows radial-acting flow behaviour. The handling of the straight line recovery method is similar to the drawdown approach. The transmissivity of the formation can be determined using Eq. (2.39) (Section 2.7.2.3.1).

2.7.4.3.2 Type curve application

The handling of the Boonstra and Boehmer type curve method is similar to that of the Theis type curve method. After matching the data curve of the observation well with the type curve, the following parameter products can be calculated by substituting the values for the match point co-ordinates in the following equations:

$$w \cdot T_f = \frac{Q \cdot p(\chi, t_d) \cdot x}{2 \cdot s(x,t) \cdot \chi} \qquad (2.51)$$

$$w \cdot S_f = \frac{Q \cdot p(\chi, t_d) \cdot \chi \cdot t}{2 \cdot s(x,t) \cdot t_d} \qquad (2.52)$$

$$S \cdot T = \frac{Q^2 \cdot p^2(\chi, t_d) \cdot \chi^2 \cdot t}{14 \cdot s^2(x,t) \cdot x^2 \cdot t_d} \qquad (2.53)$$

where
Q = constant discharge rate [L^3T^{-1}]
s(x,t) = drawdown [L]
t = time [T]

If the radial-acting flow phase is present, the formation transmissivity T can be calculated using the straight line method and by replacing T in Eq. (2.53), the formation storage coefficient S can be estimated. If the feature's width w is known, it is possible to determine the feature's parameters T_f and S_f by replacing w in Eqs. (2.51) and (2.52), respectively.

The type curve fitting using the pumped well data yields following product of parameters:

$$\left(w \cdot S_f\right)\left(w \cdot T_f\right) = \frac{\left[\dfrac{Q \cdot p(0, t_d)}{s(0,t)}\right]^2}{\dfrac{4 \cdot t_d}{t}} \qquad (2.54)$$

$$\left(w \cdot T_f\right)\sqrt{(S \cdot T)} = \frac{\left[\dfrac{Q \cdot p(0, t_d)}{s(0,t)}\right]^2 \cdot \sqrt{\dfrac{t}{t_d}}}{7.5} \qquad (2.55)$$

Eq. (2.54) is valid for those cases where the data curve fits small dimensionless time values (t_d approximately 0.003), which indicates linear feature flow and is characterised by a slope of 0.5 in a log-log plot. Eq. (2.55) must be used in those cases where the curve fits for t_d in the range of 100, which describes bilinear flow and is represented by a slope of 0.25 in a log-log plot.

2.7.4.3.3 Determination of skin effects

Skin effects can appear at the well (well bore skin), between fracture and matrix (fracture skin) or on both (Cinco-Ley and Samaniego, 1981b). If drawdown data from wells situated in a vertical feature with finite conductivity are obscured by skin effects, the curve shows an almost horizontal drawdown at the early time data followed by the radial-acting flow period after a transition zone. In those cases where the early time data show linear flow (slope of 0.5 in a log-log plot), the total skin factor ξ_t can be graphically determined using the pumped well data represented in a lin $t^{½}$ plot (Fig. 2.62a) and Eq. (2.42) (Section 2.7.2.3.3). The same procedure can be followed for the determination of the skin factor when the early time data show a bilinear flow case (slope of 0.25 in a log-log plot), but the data must be graphed in a lin $t^{¼}$ plot (Fig. 2.62b). If the early time data do not show linear or bilinear flow, it is not possible to apply this method for the determination of the skin factor.

2.7.4.3.4 Forward modelling application

If the formation transmissivity value is known from the straight line approach, this value should be used as the known parameter in the forward modelling to shorten the time necessary to fit the unknown parameters. The Boonstra and Boehmer model is implemented in TPA for both pumped well and observation well located in the same feature.

In the worst case the known parameters are:

- Hydrogeological concept of a single vertical feature with finite transmissivity.
- Transmissivity T of the matrix.
- Skin factor $\xi_{t.}$

The following unknown parameters must be estimated:

- Storage coefficient S of the matrix.
- Transmissivity T_f of the feature.
- Storage coefficient S_f of the feature.
- Fracture width w

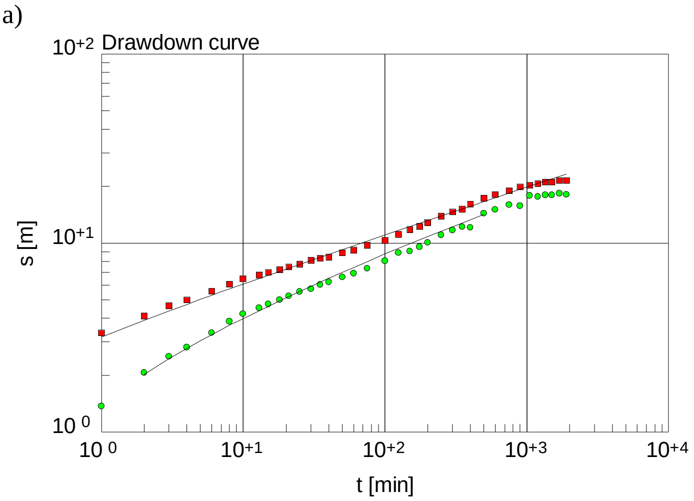
2.7.4.4 Field example

Dolerite dyke at Brandwag Tweeling, Republic of South Africa (Boonstra and Boehmer, 1986 and Boehmer and Boonstra, 1987):

A pumping test was performed in a dyke intruded in the Beaufort Series of the Karoo Formation. The test set-up consisted of a group of three wells. The pumped well, located within a 10 m wide dyke an observation well located 100 m apart from the pumped well within the same dyke, and another observation well situated 20 m from the pumped well perpendicular to the dyke (Fig. 2.63).

Discharge rate [m^3/h]	Q	= 50
Formation transmissivity [m^2/d]	T	= 9.3
Formation storage coefficient [-]	S	= 0.000034
Dyke transmissivity [m^2/d]	T_f	= 2390
Dyke storage coefficient [-]	S_f	= 0.000043
Well bore skin factor [-]	ξ	= 0
Dyke skin factor [-]	ξ_f	= 0
Distance of the observation well 1 [m]	r_1	= 200
Distance of the observation well 1 [m]	r_2	= 20

a)



b)

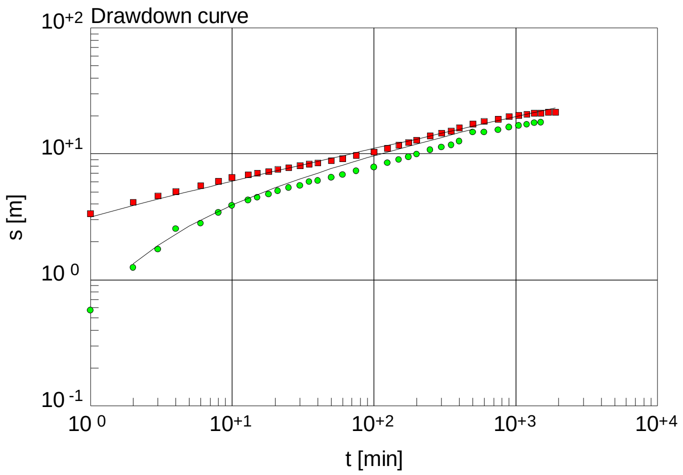


Fig. 2.63 **Pumping test in a dyke. The upper plot shows the pumped well (squares) and the observation borehole (dots) both located in the same dyke. The second plot shows the pumped well (squares) and the observation well (dots) located perpendicular to the dyke**

2.7.5 Bedding plain fracture with infinite conductivity and finite extent (Gringarten and Ramey, 1974)

2.7.5.1 Theory

Gringarten and Ramey (1974) introduced an analytical model that describes the drawdown in a penny-shape bedding plane fracture with infinite conductivity and finite extension with uniform flux. The fracture is embedded in an infinite, homogeneous, horizontal matrix that has anisotropic radial and vertical conductivities and is limited by upper and lower impermeable boundaries. The model considers linear flow followed by radial-acting flow after a transition period.

The equation presented by Gringarten and Ramey (1974) assumes that the flux is constant and uniform along the fracture. It is obtained by means of the Green functions and reads:

$$p(r_d, z_d, t_d) = \int_0^{t_d} \frac{e^{\left(-\frac{r_d^2}{4t_d'}\right)}}{t_d'} \left[\int_0^1 I_0\left(\frac{r_d r_d'}{2t_d'}\right) \cdot e^{\left(-\frac{4r_d'^2}{4t_d'}\right)} \cdot r_d' \cdot dr_d' \right] \cdot \rho \cdot dt_d' \qquad (2.56)$$

with

$$\rho = \left[1 + \frac{4h}{\pi h_f} \sum_{n=1}^{\infty} \frac{e^{\left(-\frac{n^2\pi^2 t_d'}{h_d^2}\right)}}{n} \cdot \sin n\pi \frac{h_f}{2h} \cdot \cos n\pi \frac{z_f}{h} \cdot \cos n\pi \frac{z}{h} \right]$$

where

p = dimensionless drawdown in the fracture [L]

$r_d = \frac{r}{r_f}$ [–]

$z_d = \frac{z}{r_f}\sqrt{\frac{k_r}{k_z}}$ [–]

$t_d = \frac{k_r \cdot t}{S \cdot r_f^2}$ [–]

$h_d = \frac{h}{r_f}\sqrt{\frac{k_r}{k_z}}$ [–]

r = observation distance [L]
r_f = radius of the penny-shaped fracture [L]
z = vertical distance to the reservoir lower boundary [L]
z_f = vertical distance from the fracture to the lower boundary of the reservoir [L]
k_r = radial hydraulic conductivity of the formation [LT^{-1}]
k_z = vertical hydraulic conductivity of the formation [LT^{-1}]
t = time of flowing [T]
S = Storage coefficient of the formation [-]
I_0 = modified Bessel function of the first kind of order 0
h = formation thickness [L]
h_f = fracture thickness [L]
r_d’, t_d’ = dimensionless variables of integration

The model is able to describe four different flow phases as follows (Gringarten and Ramey, 1974):

- Fracture storage flow, which duration depends on the fracture thickness.
- Linear formation flow, which duration depends on the fracture radius and distance to the upper and lower formation boundaries.
- Transient flow, which occurrence depends on the dimensionless parameter h_d, and
- Radial-acting flow, which starts at $t_d = 5$.

2.7.5.2 Diagnosis

The drawdown curve in a pumped well situated in a horizontal penny-shape fracture with infinite conductivity, finite extension and uniform flux shows the following flow periods:

- Fracture storage flow at very early time, which shows a typical slope of 1 in a log-log plot (Fig. 2.64).
- Linear formation flow at early time characterized by a slope of 0.5 in a log-log plot (Fig. 2.64) or a straight line in a lin $t^{½}$.
- A transition period that shows a typical slope of 1 in a log-log plot (Fig. 2.64).
- Radial-acting flow at late time (Fig. 2.64), which plots as a straight line in a lin-log plot.

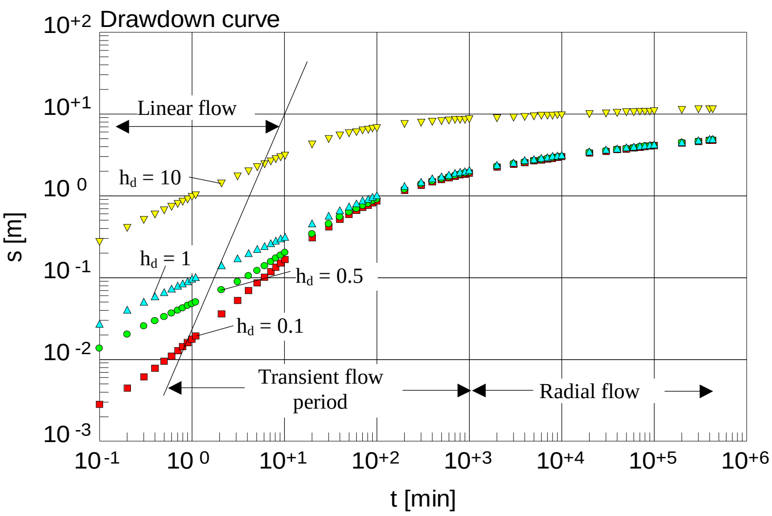


Fig. 2.64 Flow periods in wells located in a penny-shape horizontal fracture

The fracture has an infinite conductivity and therefore the reaction in an observation well located in the same fracture as the pumped well is instantaneous, as described in the infinite conductive vertical fracture case (Gringarten *et al.*, 1974). The shape of the drawdown in the observation well at early times is similar to that of the pumped well (Fig. 2.65).

The skin effect can be determined using the method described for the infinite conductive vertical fracture (Gringarten *et al.*, 1974) as shown in Fig. 2.65. The location can be graphically determined following the method presented by Bardenhagen (1999), as described in Section 2.7.2.2 for the infinite conductive fracture case. However, this method cannot be applied if the drawdown shows fracture storage flow.

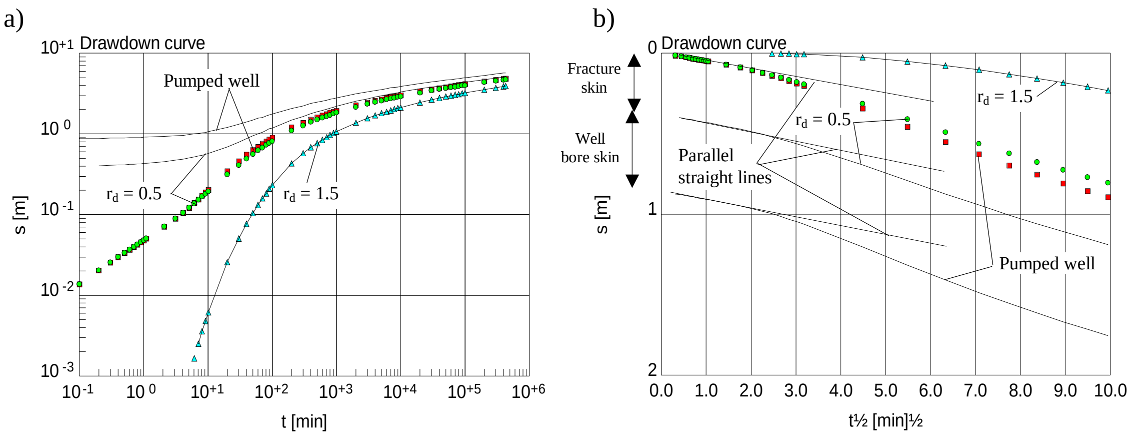


Fig. 2.65 Drawdown for h_d = 0.5 in a pumped well (squares) and two observation wells, one at r_d = 0.5 (dots) and the other at r_d = 1.5 (triangles) from the pumped well, respectively. The solid lines represent the drawdown in the same wells when affected by well bore and fracture skins. Fig. a) presents the whole test and Fig. b) the early time data

2.7.5.3 Method of analysis

Basically, two methods of analysis can be used to determine the formation or matrix transmissivity T from the drawdown data: a straight line method using a lin-log plot and a type curve method using a log-log plot. If recovery (build up) data are available, the recovery method of Theis (1935) or Agarwal (1980) can be used. Alternatively, a forward modelling using TPA can be applied to determine the aquifer parameters T and S for both the drawdown and recovery phases. Note that the determination of the actual fracture transmissivity T_f is not possible because it is a priori considered infinite.

The methods can be applied if the following conditions are true:

- Matrix is infinite.
- aquifer (fracture and matrix) is confined.
- Darcian flow prevails in fracture and matrix.
- Well penetrates the aquifer fully.
- Negligible well bore storage.
- Negligible well bore skin and fracture skin.
- Straight line can be applied if td > 5. Cross-check, where the first derivative becomes horizontal.

2.7.5.3.1 Straight line application

The straight line method allows the determination of the reservoir transmissivity T using the radial-acting flow phase of drawdown data in a lin-log plot for pumped well and/or observation well data. The approach is similar to the methods of Cooper-Jacob (1947) and Theis (1935). The transmissivity T of the formation can be determined using Eq. (2.37) (Section 2.7.2.3.1).

The straight line method can be applied for the estimation of the reservoir storage coefficient S, if the observation well is located at a distance greater than five times the fracture radius.

The Theis (1935) and Agarwal (1980) recovery methods are applicable for the determination of the formation transmissivity T, if a significant portion of the recovery curve shows radial-acting flow behaviour. The handling of the straight line recovery method is similar to the drawdown approach. The transmissivity of the formation can be determined using Eq. (2.39) (Section 2.7.2.3.1).

2.7.5.3.2 Type curve application

The handling of the Gringarten and Ramey type curve method is similar to that of the Theis type curve method. After matching the data curve of the pumping well with one of the type curves, the reservoir transmissivity T can be calculated by substituting the values for the match point co-ordinates in Eq. (2.40) (Section 2.7.2.3.2). The matching also provides the value of h_d, which allows the estimation of the vertical hydraulic conductivity k_z, using the following equation:

$$k_z = \frac{T \cdot h}{\left(h_d \cdot r_f\right)^2} \qquad (2.57)$$

For the given h_d, a different set of curves dependent on r_d can be calculated and graphed on a lin-log plot. A new match of the drawdown data with these curves will provide a value of r_d that permits the calculation of r_f ($r_f = r/r_d$), which is required for the evaluation of the storage coefficient, using Eq. (2.41) as presented in Section 2.7.2.3.2.

2.7.5.3.3 Determination of skin effects

Skin effects can appear at the well (well bore skin), between fracture and matrix (fracture skin) or on both (Cinco-Ley and Samaniego, 1981b). If drawdown data from wells situated in a horizontal fracture with infinite conductivity are obscured by skin effects, the curve shows an almost horizontal drawdown at the early time data followed by the radial-acting flow period after a transition zone.

In those cases where the early time data show formation linear flow (slope of 0.5 in a log-log plot), the skin locations and the skin factors can be determined as described in Section 2.7.2.3.3 for the infinite conductive vertical fracture case.

2.7.5.3.4 Forward modelling application

If the formation transmissivity value is known from the straight line approach, this value should be used as the known parameter in the forward modelling to shorten the time necessary to fit the unknown parameters. The Gringarten and Ramey model is implemented in TPA for both the pumped well and observation well.

In the worst case, the known parameters are:

- Hydrogeological concept of a single horizontal fracture with infinite conductivity.
- Transmissivity T of the matrix.
- Skin factor ξ_t.

The following unknown parameters must be estimated

- Storage coefficient S of the matrix.
- Radius of the penny-shape horizontal fracture.
- Thickness of the fracture.
- Location of the fracture relative to the upper and lower reservoir boundaries.
- Vertical reservoir conductivity.

2.7.5.4 Field example

A constant discharge test was performed during 390 minutes in the test field of the University of the Free State in Bloemfontein, South Africa, where a bedding plane is embedded in the Karoo Formation. Due to the shortness of the test, the radial-acting flow was not reached; therefore the estimated aquifer parameters are relatively uncertain. The test set-up consisted in a pumped well and three observation wells. The test evaluation is presented in Fig. 2.66.

The obtained aquifer characteristics are:

Discharge rate [m^3/h]	Q	= 4.5
Formation transmissivity [m^2/d]	T	= 12
Formation storage coefficient [-]	S	= 0.002
Formation thickness [m]	h	= 20
Vertical conductivity [m/d]	k_v	= 0.00085
Fracture radius [m]	r	= 280
Fracture width [m]	w	= 0.2
Fracture elevation [m]	h_f	= 10
Distance of the observation well 1 [m]	r_1	= 5
Distance of the observation well 1 [m]	r_2	= 22
Distance of the observation well 1 [m]	r_3	= 32

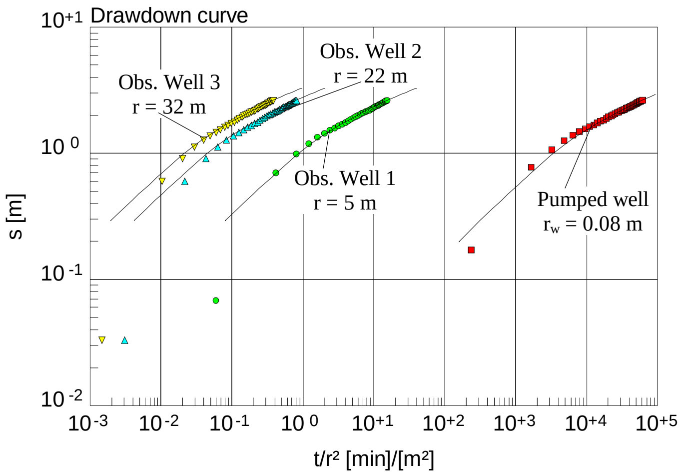


Fig. 2.66 **Evaluation of the pumping test performed in the test field of the University of the Free State in Bloemfontein, South Africa. The data are represented by the symbols and the modelled curve using the Gringarten and Ramey model is graphed with solid lines**

2.7.6 Generalised radial flow model for fractured reservoirs (Barker, 1988)

2.7.6.1 Theory

Barker (1988) introduced an analytical model that describes the drawdown in a fractured aquifer for various flow dimensions including linear, radial and spherical flows. These flow dimensions are seen as dependent on the fracture connectivity rather than as aquifer dimensions and are described by a factor n. Flow dimensions equal aquifer dimensions for integer values of n: for n = 1 the flow is strictly linear, for n = 2 the flow is radial (Theis model) and for n = 3 the flow is spherical. The non-integer values of n describe the excess or lack of fracture connections compared to fracture networks with perfect connections in 1, 2 and 3 dimensions (Leveinen *et al.* 1998).

The solution is valid for a homogeneous and isotropic fractured medium and considers 1, 2, and 3 dimensional sources with a finite storage capacity. The source dimensions are defined by b^{n-3}, where n = 1 implies a very thin cube source, n = 2 a cylinder source and n = 3 a sphere source. The model also incorporates the possibility of infinitesimal skin located at the source.

Due to the fact that the model considers the fractured aquifer as an isotropic homogeneous medium, the flow dimension is strictly defined by the dimension of the source. In other words, a one-dimensional flow can be obtained when the aquifer is an infinite strip of a certain width b and thickness b and the source is a surface that intersects the entire strip. The two-dimensional flow is obtained when the aquifer is infinite with a thickness b and the source is a cylinder of height b that fully penetrates the aquifer. The three-dimensional flow will be obtained whenever the aquifer is infinite in all three directions and the source is a sphere. This effect restricts the use of the method to very few well-defined aquifers and tests, as practically no one- or three-dimensional flows can extend for an infinite period of time. Naturally, the flow tends to a radial-acting flow (two-dimensional flow) after a certain period of time due to the fact that most aquifers have a finite thickness compared to the horizontal extent.

Barker introduced two general equations to describe the head in the source and head in the formation both related to the abstraction rate (Eqs. 2.58 and 2.59, respectively). The equations obtained by means of the Laplace transformation read:

$$\frac{\bar{H}(p)}{\bar{Q}(p)} = \frac{\left[1+\xi\Phi_\nu(\mu)\right]}{pS_w + K_f b^{3-n}\alpha_n r_w^{n-2}\Phi_\nu(\mu)} \qquad (2.58)$$

$$\frac{\bar{h}(r,p)}{\bar{Q}(p)} = \frac{\rho^\nu K_\nu(\mu\rho)}{K_\nu(\mu)\left[\rho S_w\left(1+\xi\Phi_\nu(\mu)\right)+K_f b^{3-n}\alpha_n r_w^{n-2}\Phi_\nu(\mu)\right]} \qquad (2.59)$$

where

$\bar{H}(p)$ = drawdown at the source in the Laplace space [L]
$\bar{h}(p)$ = drawdown in the reservoir in the Laplace space [L]
$\bar{Q}(p)$ = abstraction rate in the Laplace space [L^3T^{-1}]
p = laplace transform variable
n = dimension of the fracture flow system [-]
b = extent of the flow region [L]

r_w = radius of the source [L]
K_f = hydraulic conductivity of the fracture system [LT^{-1}]
S_w = Storage capacity of the source [-]
S_{sf} = Specific storage of the fracture system [L^{-1}]
ξ = skin factor [-]

$$\alpha_n = \frac{2\pi^{n/2}}{\Gamma(n/2)}$$

ρ = r/r_w [-]
μ = λr_w
λ = $(p \cdot S_{sf} / K_f)^{½}$
ν = 1-n/2

$$\Phi_\nu(\mu) = \frac{\mu K_{\nu-1}(\mu)}{K_\nu(\mu)}$$

$K_\nu(\mu)$ = modified Bessel function of fractal order
$\Gamma(x)$ = Gamma function

2.7.6.2 Diagnosis

Based on the Barker model, the following flow characteristics can be observed (Fig. 2.67):

- For n > 2 steady state situation at late time, which shows a typical slope of 0 (horizontal line) in a log-log plot.
- The Theis curve corresponds to n = 2, which is characterized by a straight line in a lin-log plot.
- For n < 2 straight lines at late time with slopes of $\nu = 1 - n/2$ in a log-log plot.

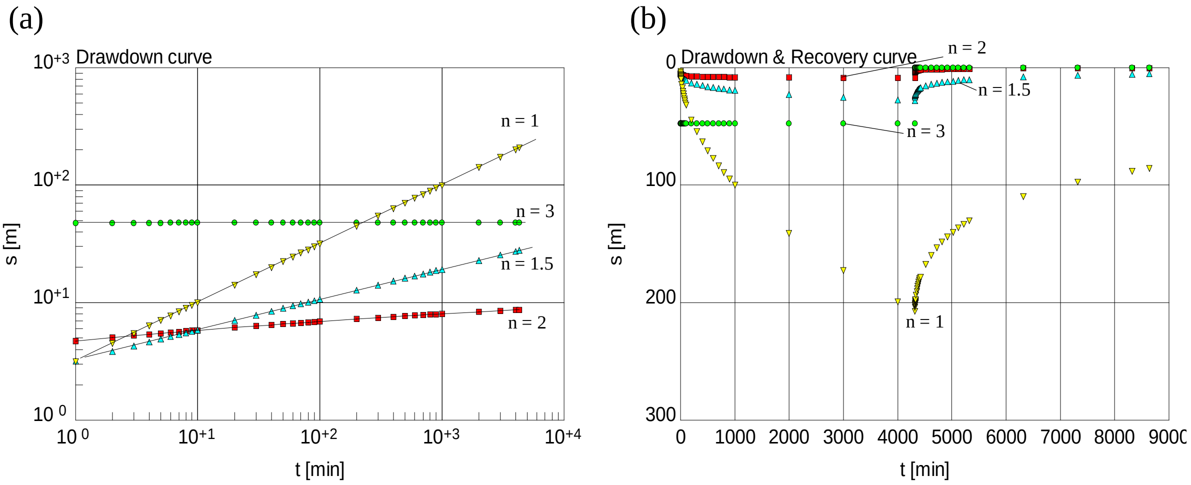


Fig. 2.67 Drawdown obtained using the same aquifer parameters for different flow dimensions n in a log-log plot (a). The graph (b) shows the drawdown and recovery for the same parameters in a lin-log plot

The disadvantages of the method can be summarised as (Fig. 2.68):

- The main difference between the Barker model and the above described models lies in the fact that the Barker model does not provide for radial-acting flow at late times for any n ≠ 2. This makes the use of the type curve fitting for the evaluation of pumping tests difficult.
- Additionally, the method in the case of n = 1 provides a drawdown curve that shows a straight line with slope 0.5 (linear flow) even, which makes it impossible to obtain a unique curve fit.

Fig. 2.68 shows the type drawdown curves for different flow dimensions n. The function F(n, u) is evaluated as:

$$F(n,u)=\frac{1}{\nu}\left[\left(\frac{1}{u}\right)^{\nu}-\frac{\Gamma(1-\nu)}{\nu}\right] \qquad (2.60)$$

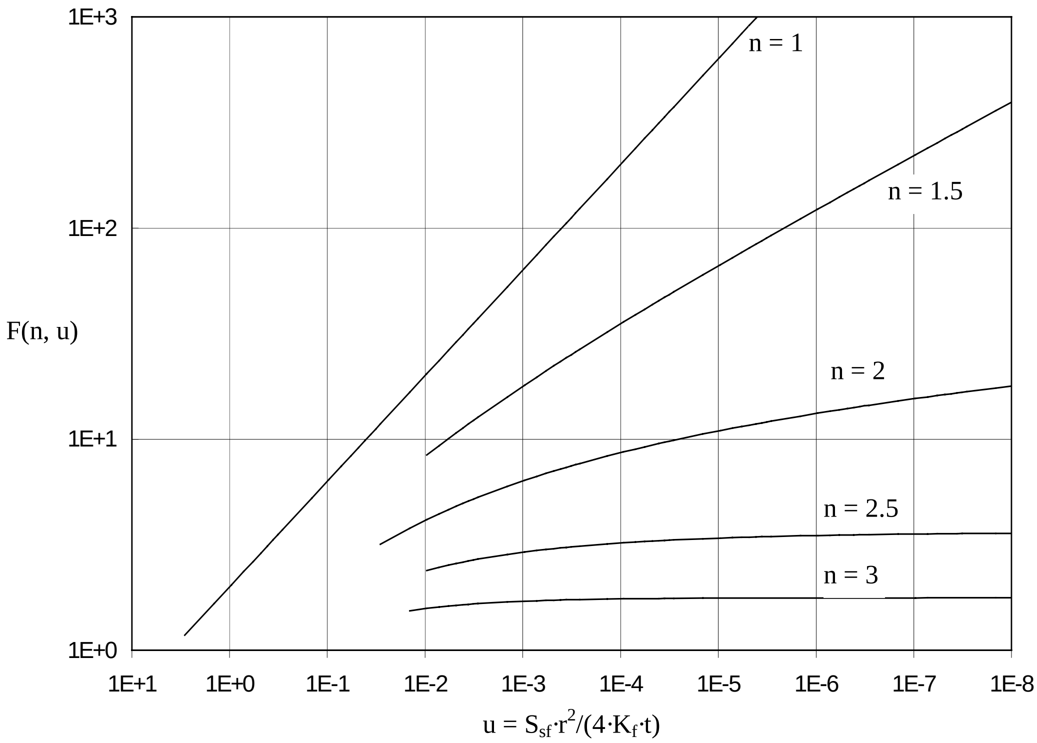


Fig. 2.68 Type curves for different flow dimensions (n) based on Eq. (2.60)

2.7.6.3 Method of analysis

Basically, only the type curve method using a log-log plot and the forward modelling using TPA can be used to determine the formation or matrix transmissivity T from the drawdown data. Common straight line methods in the lin-log plot can only be applied for the cases where n = 2.

The methods can be applied if the following conditions are true:

- Matrix is infinite
- Aquifer (fracture and matrix) is confined
- Darcian flow prevails in fracture and matrix
- Negligible fracture skin
- Straight line can be applied for n = 2 if u < 0.01. Crosscheck, where the first derivative becomes horizontal

2.7.6.3.1 Straight line application

The straight line method allows the determination of the reservoir transmissivity T using the radial-acting flow phase of drawdown data in a lin-log plot for the pumped well and/or observation well data. In the Barker model, this method is applicable only if n = 2. The approach is similar to the methods of Cooper-Jacob (1946) and Theis (1935). The transmissivity T of the formation can be determined using Eq. (2.38) (Section 2.7.2.3.1).

The Theis (1935) and Agarwal (1980) recovery methods are applicable for the determination of the formation transmissivity T if a significant portion of the recovery curve shows radial-acting flow behaviour. The handling of the straight line recovery method is similar to the drawdown approach. The transmissivity of the formation can be determined using Eq. (2.39) (Section 2.7.2.3.1).

2.7.6.3.2 Type curve application

The handling of the Barker type curve method is similar to that of the Theis type curve method. Data from the pumped well can be used only if the well bore storage and the well bore skin are negligible, but data from observation boreholes can always be analysed using this method.

After matching the data curve of the pumping well with one of the type curves, the product $K_f \cdot b^{3-n}$ (that equals the transmissivity of the fracture system for n = 2) and the inverse of diffusivity of the fracture system S_{sf} / K_f can be calculated by substituting the values for the match point co-ordinates in Eqs. (2.61) and (2.62), respectively.

$$K_f \cdot b^{3-n} = \frac{Q}{4\pi^{1-\nu} h(r,t)\nu} \left[\left(\frac{4K_f t}{S_{sf}} \right)^{\nu} - \Gamma(1-\nu) r^{2\nu} \right] \qquad (2.61)$$

$$\frac{S_{sf}}{K_f} = \frac{4ut}{r^2} \qquad (2.62)$$

Barker (1988) suggests that any two of these three parameters (K_f, S_{sf}, or b) can be determined if the third is estimated or known.

2.7.6.3.3 Determination of skin effects

Skin effects can appear at the well (well bore skin), between fracture and matrix (fracture skin) or on both (Cinco-Ley and Samaniego 1981b). If drawdown data from wells situated in a horizontal fracture with infinite conductivity are obscured by skin effects, the curve shows an almost horizontal drawdown at the early time data followed by the radial-acting flow period after a transition zone.

In those cases where the early time data show formation linear flow (slope of 0.5 in a log-log plot), the skin locations and the skin factors can be determined as described in Section 2.7.2.3.3 for the infinite conductive vertical fracture case.

2.7.6.3.4 Forward modelling application

If the formation transmissivity value is known from the straight line approach, this value should be used as the known parameter in the forward modelling to shorten the time necessary to fit the unknown parameters. The Barker model is implemented in TPA for both the pumped well and observation well.

In the worst case the known parameters are:

- Hydrogeological concept of a single horizontal fracture with infinite conductivity.
- Transmissivity T of the matrix.
- Skin factor ξ_{t}.

The following unknown parameters must be estimated:

- Storage coefficient S of the matrix.
- Radius of the penny-shape horizontal fracture.
- Thickness of the fracture.
- Location of the fracture relative to the upper and lower reservoir boundaries.
- Vertical reservoir conductivity.

2.8 BASIC INSTRUCTIONS AND LIMITATIONS FOR THE ANALYTICAL METHODS

As discussed previously in this document, all the analytical models for fractured aquifers have special assumptions which yield to limitations of their use. On the other hand, there are a lot of uncertainties and effects during the pumping test, which can make it difficult to estimate the time drawdown data correctly. In this section, some of these uncertainties and limitations will be discussed and possible solutions explained.

2.8.1 Comparison of drawdown and recovery data

The drawdown curve is often disturbed by variations in the pumping rate, which can be easily recognised by comparison of the drawdown and recovery curve shapes. In the absence of no-flow boundary effects, both curves must show the same behaviour due to the superposition theory (Fig. 2.69). If a closed reservoir is pumped, the shapes of the drawdown and recovery curves have a characteristic difference as illustrated in Fig. 2.70.

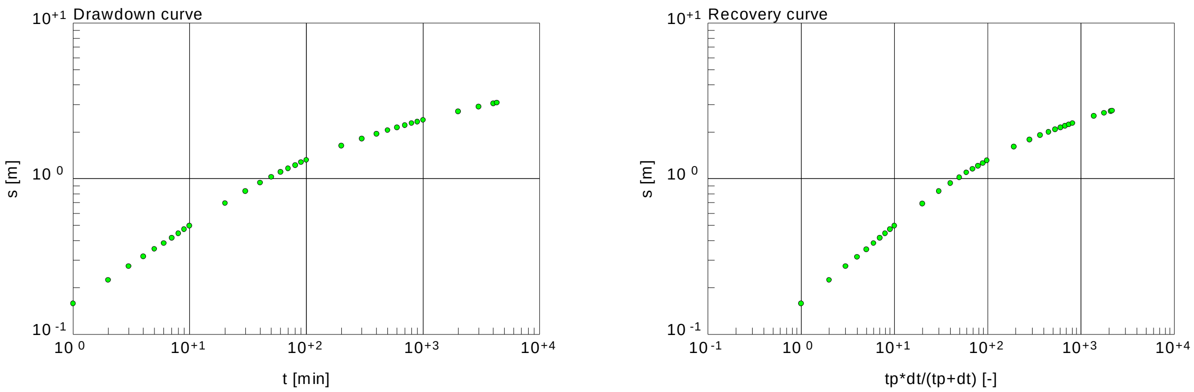


Fig. 2.69 The superposition theory implies that the shapes of the drawdown and recovery curves are similar. This effect can be used to determine the quality of the drawdown data

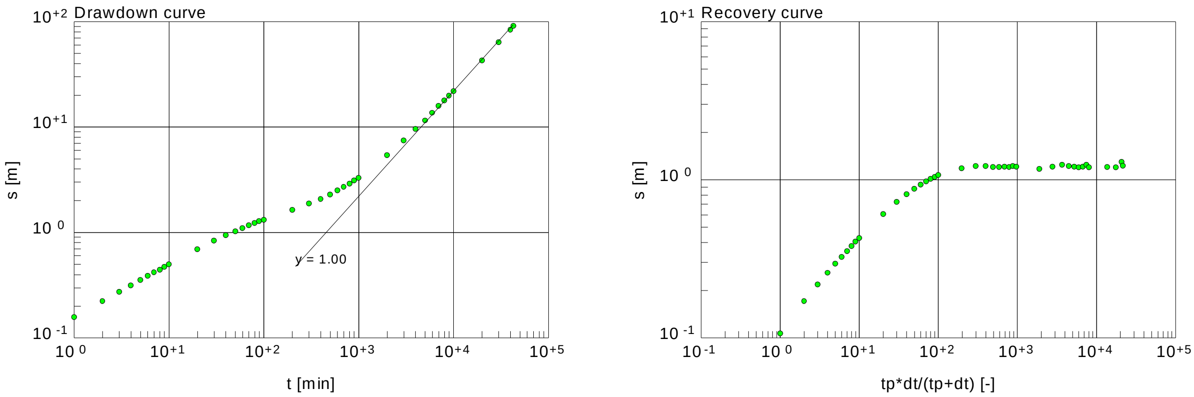


Fig. 2.70 If a limited reservoir is pumped, the drawdown and recovery curves present characteristic graphs

2.8.2 Correction for discharge variations

In cases of step drawdown tests or extreme variations in the abstraction rates during a constant discharge test (pump failure, etc.), only the recovery or build up curve is suitable for common analysis methods like Theis (1935) or Agarwal (1980). However, these methods lead to correct results if the discharge variations are eliminated using the Birsoy and Summer (1980) correction, which writes:

$$t_{corr} = \prod_{i=1}^{n-1} \left(\frac{t - t_i}{t - t_i^{'}} \right)^{\frac{Q_i}{Q_n}} \qquad (2.63)$$

where
t_i = start time of the i^{th} discharge period
t_i' = end time of i^{th} discharge period
Q_i = constant discharge rate of i^{th} period
Q_n = last constant discharge rate

If the time correction is ignored, significant differences in the recovery behaviour are observed (Fig. 2.71), which would result in a wrong determination of the transmissivity.

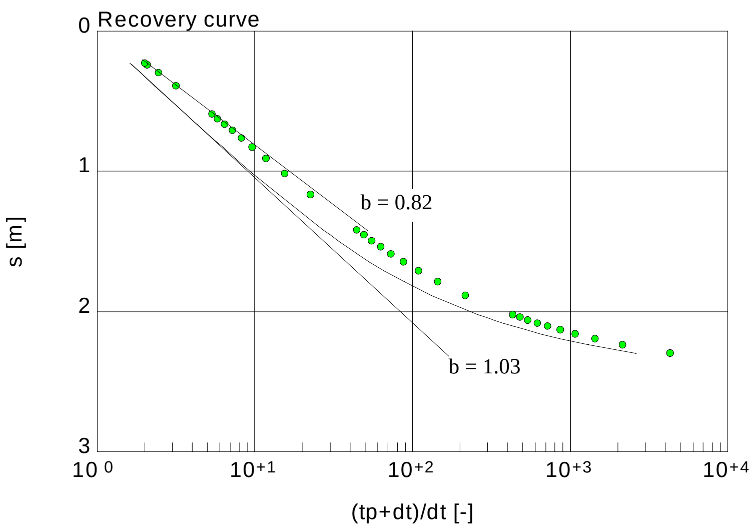


Fig. 2.71 The curves represent the recovery from a step test run at Q = 4 m^3/h, Q = 8 m^3/h and Q = 12 m^3/h. The dots graph the recovery considering a constant average discharge of Q = 8 m^3/h and no time correction. The solid curve shows the same recovery data with time correction. The Theis recovery method estimates transmissivity values of T = 50 m^2/d for corrected data and T = 43 m^2/d for uncorrected data, due to the differences in the applied discharge rate (Q = 12 m^3/h for corrected data and an average of Q = 8 m^3/h for uncorrected data)

The corrected time t_{corr} replaces the pumping time t_p in the Theis (1935) superposing correction for the recovery curve, which then reads:

$$t' = \frac{t_{corr} + dt}{dt} \qquad (2.64)$$

where
t' = time corrected for the superposition effects in the drawdown phase
dt = time increment after the recovery (build up) phase starts

The time correction for the recovery curve in the Agarwal (1980) method reads

$$t' = \frac{t_{corr} \cdot dt}{t_{corr} + dt} \qquad (2.65)$$

The advantage of Agarwal's method lies in the possibility of using common type curve methods for the determination of the transmissivity and storage coefficient (if skin effects are negligible).

2.8.3 Influence of the pseudo-skin effect

Common analysis methods for primary aquifers (Cooper-Jacob or Theis) can be applied to pumping test results from a fracture network with limited extent for the determination of the transmissivity T, but only after the radial-acting flow phase is reached. However, these methods cannot be used for the estimation of the storage coefficient S, due to the pseudo-skin effects.

The storage coefficient in the common methods is calculated using the following equation:

$$S = \frac{2.25 \cdot T \cdot t_0}{r^2} \qquad (2.66)$$

where
T = transmissivity [L^2T^{-1}]
r = distance to the pumped well [L]
t_0 = time at which the first straight line intercept the time axis [T]

and the pseudo-skin effect gives wrong values for t_0 as illustrated in Fig. 2.72.

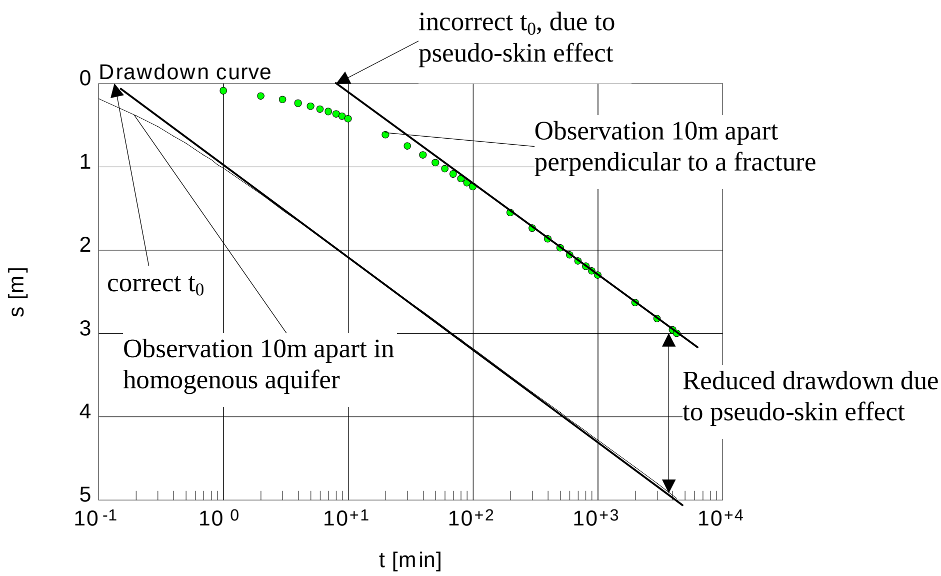


Fig. 2.72 The application of the Cooper-Jacob approach for the determination of the storage coefficient gives wrong results due to the pseudo-skin effect

Fig. 2.73 shows the extremely large error made in the calculation of S, when the Cooper-Jacob straight line method is applied to the drawdown data measured in observation wells located in various directions near an infinite conductive vertical fracture with uniform flux. Indeed, common methods are only applicable in the calculation of S when the observation well is located at a distance of at least 5 times the fracture half-length from the fracture. This distance represents the REV of such system and is identical to the location where the cone of depression reaches the radial-acting flow phase.

It must be borne in mind that the drawdown of an observation well in a discontinuous fractured aquifer is a function of its location and not of the extraction time, as long as the radial-acting flow phase in the observation well has been reached. Fig. 2.73 can also be used to either determine the correct storage coefficient if the relative position of the observation well to the fracture is known, or to determine the fracture half-length if the storage coefficient is known.

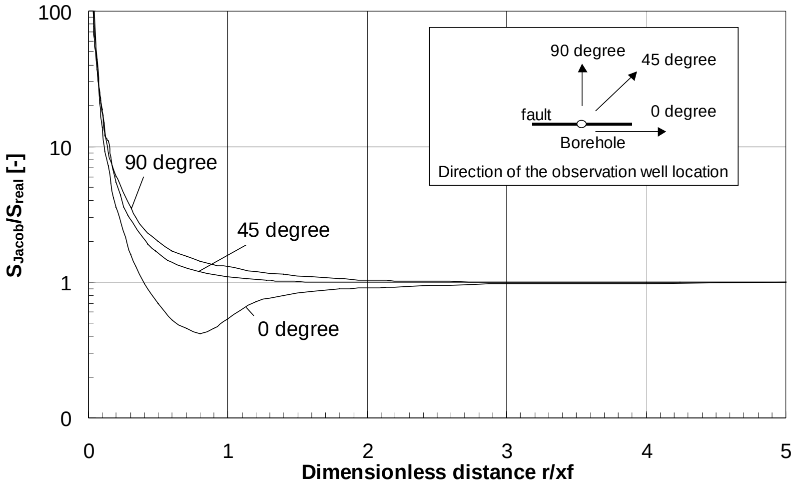


Fig. 2.73 Deviation from the real storage coefficient calculated using the Cooper-Jacob straight line method for data of the radial-acting flow phase in observation wells in the vicinity of a single vertical infinite conductive fracture with uniform flux

2.8.4 Applying Porous Media Methods

Consider the geological set-up and geometry of the fractured-rock aquifers in South Africa, in general a three-dimensional groundwater flow must be considered when estimating aquifer parameters with pumping test data. However, recently most pumping test data are still evaluated using analytical solutions such as Theis or Cooper-Jacob, which were derived for porous media and therefore cannot be applied correctly to fractured rock environments, as it will be shown in this section.

2.8.4.1 Field Example - Hydraulic Test UO26 on the Campus Test Site

The constant discharge test on the Campus Test Site was conducted at borehole UO26 for abstraction with a discharge rate of 0.71 l/s for a period of 13 hours. The boreholes UO27, UO28 and UO29 were used to observe the water level in the aquifer permanently with a pressure transducer and automatically data logger. All the boreholes intersected the same fracture zones and Fig. 2.74 shows the pumping test data in this case. The drawdown values in UO27, UO28 and UO29 were very similar and did not converge to the same drawdown value as the abstraction borehole UO26.

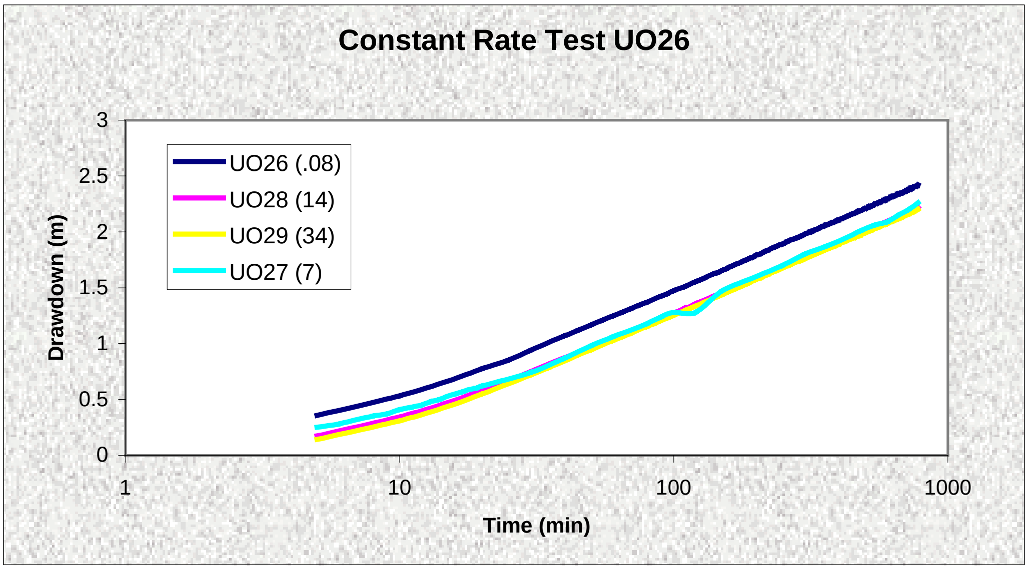


Fig. 2.74 Drawdown in the abstraction and observation boreholes during the constant rate test at UO26. The values in parenthesis give the distance between individual boreholes and UO26

The test data were analysed with different methods to evaluate the aquifer parameter transmissivity T (m^2/d) and storage coefficient S (-). The results of the analyses are shown in Table 2.1. The estimated T-values, using the methods of Cooper-Jacob or Theis, are of the same order of 10 m^2/d, but the estimated S-values differ between 3 E-5 and 2E-3 with a distance dependency (the larger the observation distance, the smaller the estimated S-value). **The reason for this phenomenon is the application of a wrong method to the real field situation**. In a fractured aquifer, we are dealing with two systems, i.e. fracture and matrix. During a pumping test, groundwater is released from matrix, flows vertically to the fracture and then along the fractured zone towards the abstraction borehole. However, neither the Theis nor Cooper-Jacob methods consider this situation. When applying these methods, only the portion of groundwater, which is released outside of the radius of the observation borehole, is considered by the analytical solution. This portion is decreasing with the increasing distance of the observation borehole. This explains the decreasing of the S-values with the distance. As a result of applying Theis or Cooper-Jacob methods, the estimated T and S-values represent a mixture of the matrix and fracture properties (Chiang and Riemann, 2001).

By using the distance-drawdown method (known as Cooper-Jacob II, see Fig. 2.75), the estimated T-value is much higher and the S-value much smaller than with Cooper-Jacob or Theis. With the Cooper-Jacob II method, the estimated values mainly represent the fracture zone, while the estimated values with the other methods represent the whole aquifer (fracture + matrix).

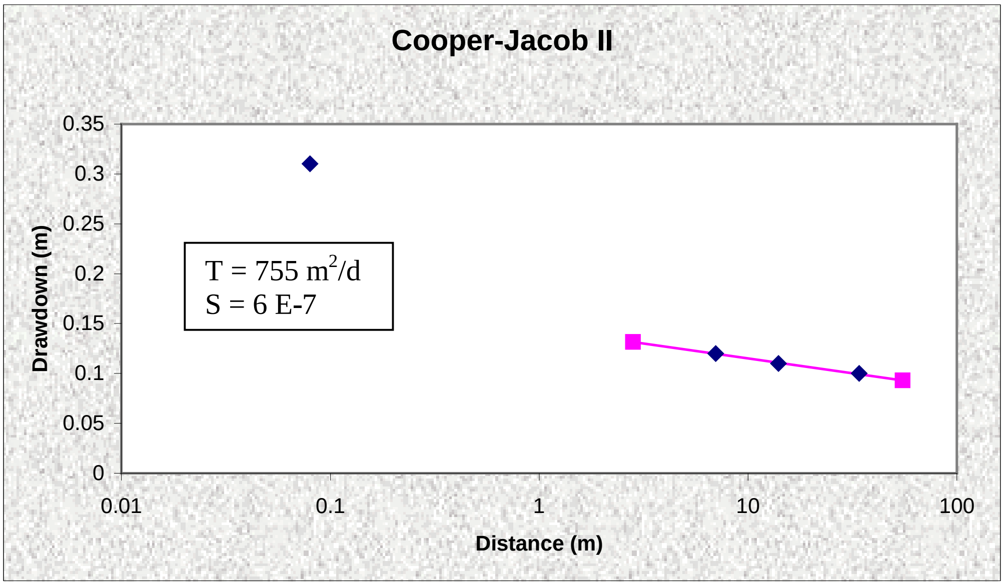


Fig. 2.75: Measured drawdown after 2 minutes vs. distance of observation during constant rate test at UO26

From the graphs and the results, it can be seen, that the standard methods for estimating aquifer parameter from hydraulic tests (i.e. Theis and Cooper-Jacob) cannot be applied correctly to fractured aquifers, because the vertical flux from the matrix to the fracture and the storage of the matrix are nearly neglected in these models.

Table 2.1 Results of the evaluated aquifer parameter for the constant rate test UO26

Borehole	Cooper-Jacob Method		Theis Method		Comment
	T (m^2/d)	S (-)	T (m^2/d)	S (-)	
UO26	10.6	2.75 E-3	10	3.30 E-3	S-value calculated with effective BH Radius of 25 m
UO27	10.6	2.13 E-3	10	2.45 E-3	T- and S-values valid for whole aquifer (fracture + matrix)
UO28	10.5	5.41 E-4	10	5.80 E-4	
UO29	10.7	8.92 E-5	11	8.60 E-5	
All BH's	755	5.87 E-7			Cooper-Jacob II for fracture zone

2.8.4.2 Field Example - Hydraulic Test M1 on the Meadhurst Test Site

The constant discharge test on the Meadhurst Test Site was conducted at borehole M1 for abstraction with a discharge rate of 1.4 l/s for a period of 16 hours. The boreholes M2, M3, M5 and M6 were used to observe the water level in the aquifer over the whole period of the test. Fig. 2.76 shows the data and graphs of the 1st test for the abstraction borehole M1 and the nearest observation borehole M2. The behaviour of the drawdown in both boreholes are very similar, even the jump in the drawdown due to the change of the discharge rate after around 8 minutes, is clearly to observe in both boreholes.

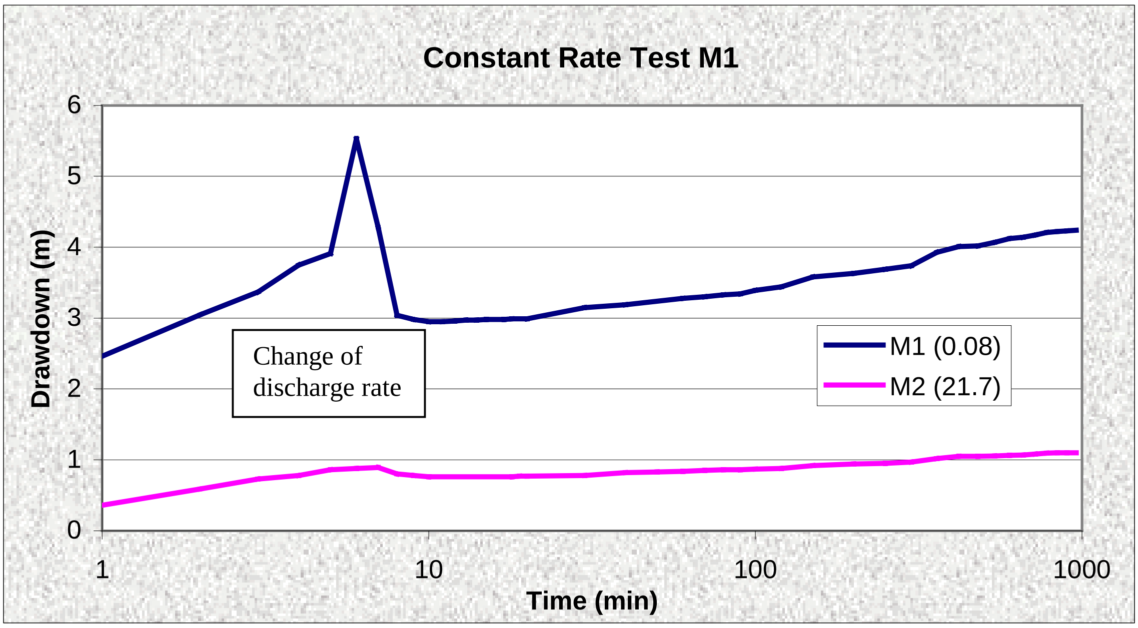


Fig. 2.76 Drawdown in the abstraction borehole M1 and the observation borehole M2 during the constant rate test with 1.4 l/s at Meadhurst Test Site

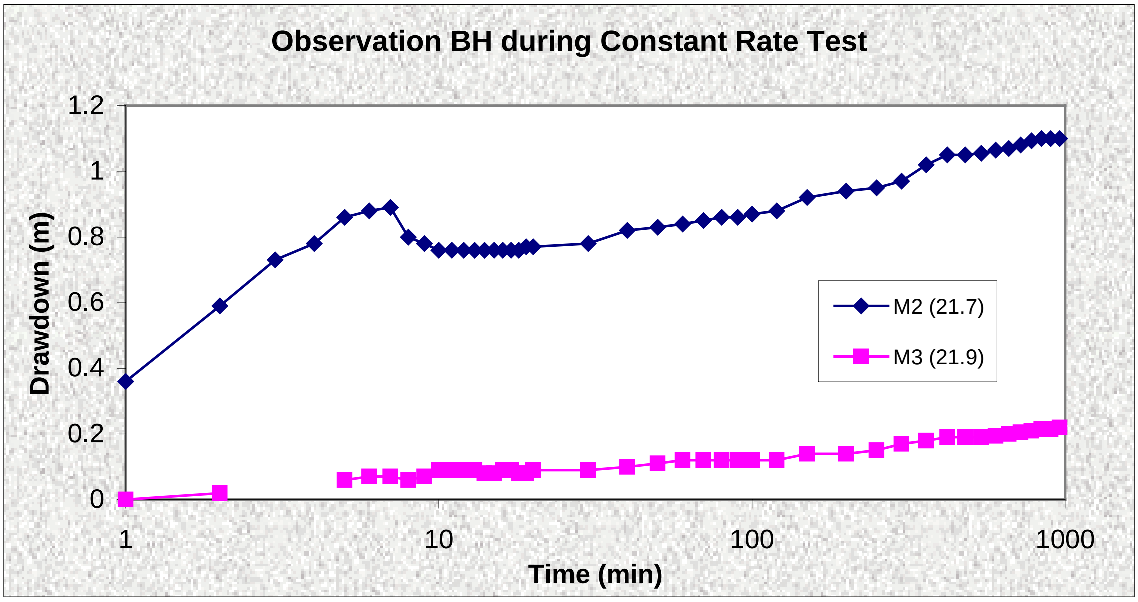


Fig. 2.77 Drawdown behaviour of the observation boreholes M2 and M3

The drawdowns in the boreholes M2 and M3, both intersecting the dyke with a distance of 2 m to each other and having almost the same distance to the abstraction borehole M1, show a different behaviour. While drawdown values in M2 and M3 react in the same way as those of the abstraction borehole M1 (Figs. 2.76 and 2.77), the drawdown in M3 is much less (Fig. 2.77). This shows that the hydraulic contact between M1 and M2 is better than between M1 and M3. A similar behaviour can be observed in M6.

The estimation of the transmissivity for the different boreholes, used in the methods from Theis and Cooper-Jacob, yields values between 23.8 m^2/d and 294 m^2/d (see Table 2.2). The high values were estimated for boreholes with a small drawdown like M3, M5 and M6. In the same boreholes, large values for the storage coefficient, between 3E-3 and 3E-2, were estimated, while the storage coefficient in boreholes M1 and M2 was estimated at 2E-5.

Using the drawdown-distance method (Cooper-Jacob II), an average T-value for the fracture is estimated as 35 m^2/d and an average storage coefficient S as 3E-5. From the graph (see fig. 2.78), it can be shown that these values are only valid for boreholes M1, M2 and M5, while the plots of boreholes M3 and M6 lie outside of the straight line. This even shows that these boreholes do not have a good hydraulic contact with the fracture system M1 – M2 – M5.

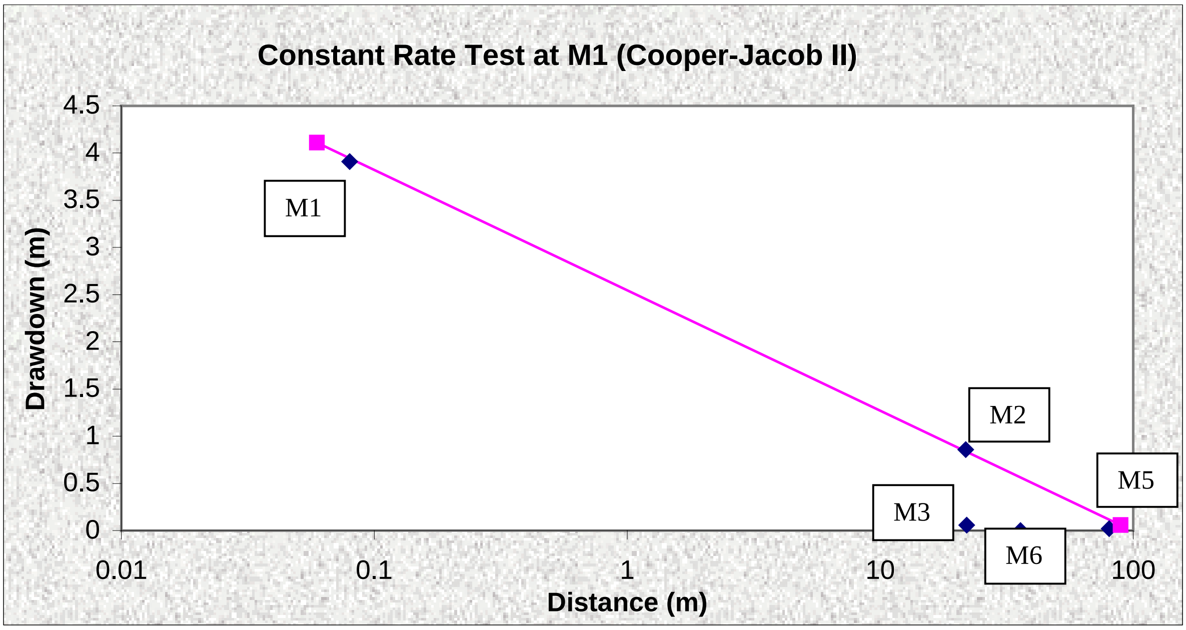


Fig. 2.78 Drawdown-distance plot of the constant rate test at Meadhurst Test Site

From the results listed in Table 2.2, it can be clearly seen that the application of Cooper-Jacob or Theis methods cannot yield correct values in this case due to the different behaviour of the fractured aquifer, because these methods are only valid for porous media and under restrictive assumptions.

Table 2.2 Results from the Constant Rate Test at Meadhurst Test Site, using the standard methods of Cooper-Jacob and Theis

Borehole	Cooper-Jacob Method		Theis Method		Comment
	T (m^2/d)	S (-)	T (m^2/d)	S (-)	
M1, 1st	23.8	3.00 E-3	24	4.70 E-3	S-value calculated with effective BH Radius
M1, 2nd	21.3	5.00 E-3	22	4.80 E-3	
M2	82.1	1.90 E-5	82	1.90 E-5	
M3	167.5	1.09 E-2	182	8.50 E-3	T- and S-values are too high for the system
M5	294.4	3.87 E-3	292	4.40 E-3	
M6	273.7	3.19 E-2	273	3.49 E-2	
All BH's	34.75	3.07 E-5			Cooper-Jacob II for fracture zone

2.8.4.3 Conclusions from Field Experiments

To apply the porous media methods like Cooper-Jacob or Theis for analysing hydraulic tests in fractured aquifers will yield wrong values, as shown above. The main errors are listed below:

- Estimated S-values depend on the distance from the observation to the abstraction borehole.
- Boreholes in badly connected fractures will yield too high T- and S-values.
- Estimated T-values resulted from conductivity in the fracture, horizontal and vertical conductivity in the matrix and aquifer thickness.
- Applying the straight line method (Cooper-Jacob) or the Theis method to the phase of radial flow (derivative is horizontal) will yield parameter values, which are not representing the fracture or the formation alone (harmonic mean of the values).

In general, the spread of the drawdown wave can significantly be affected by several hydrogeological parameters, including:

- the horizontal hydraulic conductivity of the matrix (k_{hm}),
- the vertical hydraulic conductivity of the matrix (k_{vm}),
- the specific storage coefficient of the matrix (S_{sm}),
- the horizontal hydraulic conductivity of the fracture (k_{hf}),
- the vertical hydraulic conductivity of the fracture (k_{vf}) and
- the specific storage coefficient of the fracture (S_{sf}).

Theis or Cooper-Jabcob methods and therefore all other methods, which are based on these methods, can only provide an estimate of the transmissivity (a mixture of k_{hm}, k_{vm}, k_{hf}, k_{vf} and aquifer thickness) and storage coefficient (a mixture of S_{sm}, S_{sf} and aquifer thickness).

2.8.5 General Limitations of Analytical Methods

In Section 2.7, the different flow models, derived for fractured aquifers, were described and the underlying assumptions were listed. For example, the Warren and Root or Kazemi method for a double porosity aquifer can be applied only if the conditions on the left side of Table 2.3 are true. On the right side, the real situation in most of the aquifers in South Africa are described.

Table 2.3: Comparison of typical assumptions for applying analytical methods to pumping test data in fractured aquifers with the normally real situation of the aquifer

<table>
<tr><th>Assumptions of Analytical Method</th><th>Real Situation of Aquifers in SA</th></tr>
<tr><td>• Aquifer is infinite</td><td>• All aquifers have a limited extent due to no-flow or recharge boundaries, so the aquifer is big but not infinite.</td></tr>
<tr><td>• Aquifer is confined</td><td>• Often the fractures in the aquifer are connected through smaller fractures to the surface or a phreatic aquifer on top, which implies semi-confined or unconfined conditions.</td></tr>
<tr><td>• Darcian flow prevails in fracture network and matrix</td><td>• In fractures and under high abstraction rates, non-linear or turbulent flow will occur as shown in Chapter 4.</td></tr>
<tr><td>• Fracture network is considered as continuum during the whole abstraction period</td><td rowspan="2">• In cases of single fracture aquifers, the REV is often much larger than the influence of the pumping test, so you can’t consider the continuum approach.
• During a pumping test, the volume of the fracture network, which takes part in the test, is changing with time.</td></tr>
<tr><td>• Matrix is considered as continuum during the whole abstraction period</td></tr>
<tr><td>• Well penetrates the aquifer fully</td><td>• Usually, the boreholes are fully penetrating the main aquifer, consisting of single fracture and matrix. In some cases, even overlying aquifers are penetrated.</td></tr>
<tr><td>• Negligible well bore storage</td><td>• Well bore storage is mostly higher at the beginning of a pumping test and can overlie other effects, which are important for the parameter estimation. With late times, the effect disappears.</td></tr>
<tr><td>• Negligible well bore skin</td><td>• Boreholes in fractured aquifers will mostly show a negative well bore skin, which is not negligible, because the effective borehole radius will thus increase to some metre.</td></tr>
</table>

CHAPTER 3

ESTIMATION OF THE SUSTAINABLE YIELD OF A BOREHOLE IN FRACTURED AQUIFERS

3.1 INTRODUCTION

An increasing number of boreholes in Southern Africa have dried up during the past years, in spite of favourable hydrologic conditions. An investigation of reliable estimates for the sustainable yield of the boreholes was therefore required. Overestimation of the borehole yield was due to the application of improper extrapolation of drawdown curves, which ignored barrier boundaries and neglected parameter uncertainties arising from the imperfect knowledge of the effective aquifer properties. Sami and Murray (1998) give a summary of methods that are commonly used in SA to estimate the sustainable yield of a borehole and the methods include:

(i) the Recovery Method,
(ii) the late T-method,
(iii) the Drawdown-to-boundary Method and
(iv) the Distance-to-boundary Method.

Naafs (1999) compared the methods above using the FC-method (Van Tonder *et al*., 1998) and found that the Recovery method and the late T-method are not to be used because they gave a too high sustainable yield in most of the cases tested. Naafs adapted the late T-method by introducing a variable available drawdown. In the case of this adapted late T-method, it yielded very similar results if compared to the Drawdown-to-boundary and Distance-to-boundary methods. Both the adapted late T-method and the Drawdown to-boundary methods are special cases of the FC-method.

The following sections show how to estimate the sustainable yield of a borehole by quantifying the effects of no-flow boundaries as well as the uncertainties in the values of transmissivity, storativity and distances to the boundaries.

3.2 ESTIMATION OF THE SUSTAINABLE YIELD OF A BOREHOLE

The ratio of drawdown s to pumping rate Q is a constant for a well (if corrected for well losses). This constant only depends on the aquifer properties transmissivity T and storativity S: If t_{long} describes the maximum operation time in which the drawdown s shall not exceed a maximum drawdown $s_{available}$, the extrapolation of the measured pumping test drawdown can be used to determine the sustainable yield $Q_{sustainable}$:

$$Q_{Sustainable} = Q_{PumpingTest} \frac{s_{Available}(t = t_{long})}{s_{PumpTest}(t = t_{long})} \tag{3.1}$$

The available drawdown is, for instance, the position of the main water strike in the borehole. If the drawdown exceeds this position, a drastic decrease in the yield of the borehole occurs and it may dry up. The problem of extrapolating the drawdown measured during the pumping test from the time of the end of the pumping test to a time t_{long} of around two to five years, remains. This extrapolation is traditionally done by applying the Theis solution. A more sophisticated extrapolation of the pumping test drawdown beyond the time of the end of the measurement is obtained by using a Taylor series expansion based on the extrapolation of the measured drawdown curve including drawdown derivatives, and by accounting for boundaries.

3.2.1 Extrapolation of pumping test drawdown

The drawdown measured during a pumping test is the sum of the drawdowns due to the production well, s_{Well}, and the boundaries, $s_{Boundary}$:

$$s(t = t_{long}) = s_{Well} + s_{Boundary} \tag{3.2}$$

The drawdown due to the production well (s_{Well}) is extrapolated by a Taylor series expansion around the late measurement points of the drawdown at $t \approx t_{EOP}$ (subscript EOP denotes end of pumping test). The Taylor series expansion is performed with respect to the logarithm of time, $\log_{10}$. A second order approximation is assumed to be sufficient:

$$\begin{aligned} s_{Well}(t = t_{long}) \approx s(t = t_{EOP}) &+ \frac{\partial s}{\partial \log t}\Big|_{t=t_{EOP}} (\log t_{long} - \log t_{EOP}) \\ &+ \frac{1}{2}\frac{\partial^2 s}{\partial(\log t)^2}\Big|_{t=t_{EOP}} (\log t_{long} - \log t_{EOP})^2 \end{aligned} \tag{3.3}$$

The time t_{EOP} has to be large enough to ensure that the drawdown has already passed the early time flow behaviour that is due to well bore storage, fracture flow and double porosity effects. This can clearly be monitored by looking at the derivative plot $\partial s / \partial \log t$ (van Tonder, 1998; Bourdet *et al*., 1984). Usually, the effect of the boundaries can only be seen at very late times of the pumping test. The extrapolation of Equation (3.3) therefore does not in general include boundary information.

For simple geometries of the boundaries, image well theory is applied to analyse the effects of the boundaries on the drawdown ($s_{Boundary}$).

The analytical expressions and the simplified boundary configurations already yield far better estimates of the sustainable yield than the traditional Theis extrapolation, which assumes an aquifer of infinite extent. The estimate can be improved further by taking into account uncertainties in the required parameters like the late time transmissivity T, storativity S and the distances to the boundaries a and b.

3.2.2 Risk analysis by uncertainty propagation

Kunstmann and Kinzelbach (1998) showed computational efficient methods of quantifying uncertainties in groundwater modelling. The Gaussian Error Propagation method can easily and most advantageously be applied to analytical formulas. It is applied to the drawdown equations presented and described below.

The drawdown in the pumping well is a function of the parameters *t*, *Q*, *T*, *S*, *a* and *b*, where *a* and *b* are the distances to boundaries. It is assumed that the latter four parameters are not known perfectly, but are within a range around their mean values:

$$T = \hat{T} \pm \sigma_T\,, \quad S = \hat{S} \pm \sigma_S\,, \quad a = \hat{a} \pm \sigma_a\,, \quad b = \hat{b} \pm \sigma_b \tag{3.4}$$

The mean drawdown $\hat{s}$ can be approximated by evaluating the drawdown equations at the mean values of the input parameters:

$$\hat{s} \approx s(\hat{T}, \hat{S}, \hat{a}, \hat{b}) \tag{3.5}$$

The standard deviation (describing the uncertainty of the drawdown) can be approximated by the following formula:

$$\sigma_s \approx \sqrt{\left(\frac{\partial s}{\partial T}\Big|_{T=\hat{T}}\right)^2 \sigma_T^{\ 2} + \left(\frac{\partial s}{\partial S}\Big|_{S=\hat{S}}\right)^2 \sigma_S^{\ 2} + \left(\frac{\partial s}{\partial a}\Big|_{a=\hat{a}}\right)^2 \sigma_a^{\ 2} + \left(\frac{\partial s}{\partial b}\Big|_{b=\hat{b}}\right)^2 \sigma_b^{\ 2}} \tag{3.6}$$

σ_S is required at the extrapolation time t_{long}, since the uncertainty of the extrapolated drawdown is of interest. Equation (3.6) shows that the uncertainty σ_S is determined by the input parameter uncertainties $\sigma_T, \sigma_S, \sigma_a, \sigma_b$, and the sensitivities $\partial s / \partial T$, $\partial s / \partial S$, $\partial s / \partial a$, $\partial s / \partial b$.

The sensitivity of the drawdown with respect to the parameters is the sum of the sensitivity of the well drawdown and the sensitivity of the image wells, i.e. the boundary drawdown. In case of the transmissivity, for instance, this can be written as:

$$\frac{\partial s}{\partial T}\Big|_{t=t_{long}} = \frac{\partial s_{Well}}{\partial T}\Big|_{t=t_{long}} + \frac{\partial s_{Boundary}}{\partial T}\Big|_{t=t_{long}} \tag{3.7}$$

The well drawdown is extrapolated by a second order Taylor series expansion (Equation (3)) from the end of the pumping test to the time t_{long} (that describes the maximum operation period of the borehole in the case of no recharge). Since the extrapolated well drawdown is based on a measured drawdown curve, its sensitivity with respect to the parameters cannot be calculated. The sensitivity of s_{Well} is therefore approximated by assuming a Theis sensitivity, e.g.

$$\frac{\partial s_{Well}}{\partial T}\Big|_{t_{long}} \approx \frac{\partial s_{Theiss}}{\partial T}\Big|_{t_{long}} \tag{3.8}$$

The analytical expression of the Theis sensitivity can easily be evaluated by a finite difference approximation. The uncertainty of the extrapolated drawdown σ_s can now be included in the estimation of the sustainable yield. The available drawdown has to be corrected by the uncertainty of the drawdown that arises from the imperfect knowledge of the aquifer parameters and the distances to the boundaries:

$$s'_{available} = s_{available} - 2\sigma_s \qquad (3.9)$$

This leads to a risk-oriented estimate of the sustainable yield.

A correction of the available drawdown by two standard deviations yields a probability of 95.5% for not exceeding the available drawdown (assuming a normal distribution for the uncertain s). A correction by one standard deviation still yields a safety of 68.3%. The owner of the borehole has to decide on the safety requirement (i.e. the probability of failure). In this manner a conservative and therefore sustainable yield should be estimated.

Application of this methodology required the determination of *T* and *S*. These parameters can be estimated by the interpretation of the drawdown curve. Moreover, to get an estimate of the available drawdown and the water strikes, the flow regime behaviour has to be investigated to identify the main fractures and the water strikes. In the following section we present a new, heuristic approach for the identification of *T* and *S* and a way to obtain a better knowledge on the flow regime.

3.3 IDENTIFICATION OF CHARACTERISTIC FLOW REGIMES

3.3.1 Use of drawdown derivatives

A specific flow regime has a characteristic pumping test curve. The use of derivatives of pressure head has been used for many years in the oil field to evaluate flow regime characteristics (Bourdet *et al*., 1984; Horne 1997). Use of the derivative of pressure head versus time is mathematically satisfying because the derivative is directly represented in one of the diffusivity equations, which is the governing equation for all the models of transient pressure behaviour currently in use in well test analysis. Consequently, the derivative response is much more sensitive to small phenomena of interest which are all integrated and, hence, diminished, by the pressure head versus time solutions usually present in well test interpretations. Accurate field measurements of drawdown versus time are, however, required.

Analytical equations describing the drawdown in a borehole are of the form:

$$s = 2.3\frac{Q}{4\pi T}\log C \qquad (3.10)$$

where s is the drawdown in the borehole, *Q* is the abstraction rate of abstraction borehole, *T* is the transmissivity of the aquifer system and *C* is a time dependent expression that varies according to aquifer type and contains the ratio *T/S* where *S* is the storativity (see Table 3.1).

From Equation (3.10), the derivative of the drawdown with respect to log (t) is found to be:

$$\frac{\partial s}{\partial \log t} = 2.3\frac{Q}{4\pi T} \tag{3.11}$$

From this equation the *T*-value can be calculated for each time. The derivative of the logarithmic drawdown with respect to log (t) is given by:

$$\frac{\partial \log s}{\partial \log t} = \frac{1}{\ln C} = \frac{1}{\ln(act_0)} = \frac{1}{b\ln 10} \tag{3.12}$$

so that the S-value could be estimated from:

$$a = \frac{T}{S} = \frac{10^b}{ct_0} \tag{3.13}$$

We refer to Equation (3.11) as "derivative" and to Equation (3.12) as "log-derivative". From Equation (3.12) the ratio *T/S* can be calculated and the combined use of Equations (3.11) and (3.12) therefore gives an *S*-value for each time. The derivatives are calculated numerically by a linear regression line yielding the slope.

Table 3.1 Values of the parameter C in Equation (3.10) for a few typical types of aquifers, with X_f the half-width of the vertical fracture, W_d the width of the dyke or fault and b a constant= 3 for an orthogonal fracture system and 1 for a linear system (Kruseman and De Ridder, 1991)

C†)	c	Aquifer
$(2.25Tt_0)/(r^2S)$	$2,25/r^2$	Homogeneous porous (Theis-model)
$(2.25T_ft_0)/(r^2S_f)$	$2,25/r^2$	Dual porosity (early time)
$(2.25T_ft_0)/[r^2(S_f + bS_m)]$	$2,25/r^2$	Dual porosity (late time)
$(16.59Tt_0)/[S(X_f)^2]$	$16,59/[(X_f)^2]$	Single vertical fracture
$(40T^3t_0)/[S(W_dT_d)^2]$	$(40T^2)/[(W_dT_d)^2]$	Conductive dyke or fault zone

†) The subscripts f, m and d refer to the fracture, rock matrix and dyke respectively

The characteristics that can be obtained from the derivative graph are as follows:

- Well bore storage shows a line with slope = 1 at early times.
- Infinite radial shows a horizontal straight-line 1 to 1.5 log cycles after well bore storage.
- A double porosity aquifer shows a characteristic dip after well bore storage.
- A single no-flow boundary shows, at most, a doubling of the derivative
- Two no-flow boundaries show, at most, a tripling of the derivative.
- A closed no-flow boundary shows a straight line with slope = 1 at late times.
- Two parallel and a U-shape no-flow boundary show a slope = 0.5 graph.
- A recharge boundary (river or dam) shows a drastic decrease in the value of the derivative

- Positions of fractures are usually seen by a typical sinus wave form (i.e. at the fracture position, the derivative decreases and after de-watering of the fracture, the derivative increases again).

If the second derivative (say s’’) of drawdown is taken, the following important characteristics could be obtained:

- The value of s’’ reaches a value of exactly 1 for a closed no-flow boundary.
- The value of s’’ is equal to zero for a homogeneous infinite aquifer (Theis model).

3.3.2 A Heuristic Approach for the Estimation of Effective T-and S-values

The effective *T*-and *S*-values are obtained by the evaluation of the derivatives as described above. We suggest taking the highest value of the drawdown derivative observed during the pumping test to estimate the effective *T*-and *S*-values. However, this holds only after having passed well bore storage effects and before having reached the boundaries. The reason for this heuristic approach is the following. When the water level reaches the position of a fracture, a flattening of the water level is observed. At this stage, one would obtain an erroneous effective T-value by using the derivative of this flattened part. The flattening of the drawdown curve is due to the fact that flow conditions have changed from confined to unconfined. At the position of the fracture, the drawdown will be according to the specific yield and not to the storage coefficient of the fracture. Because the specific yield is much higher than the storage coefficient, a flattening of the drawdown curve is observed.

The maximum drawdown derivative coincides with the maximum of the log-drawdown derivative due to the monotonic behaviour of the decimal logarithm. The effective S-value curve will thus always show an upward trend in field situations. At early times, the drawdown is according to the storage coefficient of the fracture, and then changes to the specific yield at the position of the fracture. At late times, the S-value changes to the storativity of the matrix.

3.4 JUSTIFICATION BY SYNTHETIC EXAMPLE

The MODFLOW program was used to generate a typical pumping test solution:
Case: Two-layer generated pumping test (2020 x 2020 m square closed boundary) with typical parameter values for fractured aquifers in the Karoo rocks of Southern Africa with a fracture zone in the bottom layer. Thickness of the first layer = 19 m and bottom layer = 1 m. Fracture zone is situated in bottom layer (220 x 220 m). Abstraction borehole (2 l s^{-1}) is situated in the center of this fracture zone in the bottom layer. First layer: T_m=1 m^2 d^{-1}; S_m=0.001 (typical matrix values for the Karoo aquifers). Fracture zone T_f=20 m^2 d^{-1} and S_f=1x10^{-4}. Rest of bottom layer has the same T_m and S_m as top layer. Vertical K_z = 0.1 m d^{-1}. Fig. 3.1 shows the Modflow generated values for a period of 2 years.

The use of the different derivative graphs could be used with great effect to identify certain specific flow characteristics (see Fig. 3.1) of fractured-rock aquifers.

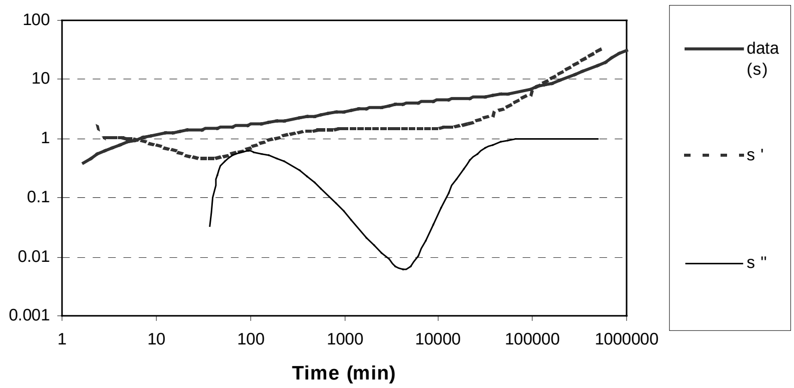


Fig. 3. 1 Modflow generated data

The Modflow program was run for a period of 2 years with an abstraction rate of 2 l s^{-1}. The generated data values for times up to 3 days (i.e. the typical length of a pumping test) were used in the FC-method to estimate the drawdown and sustainable yield by extrapolating drawdown to 2 years. The correct answer is, of course, 2 l s^{-1}. Table 3.2 shows the results:

Table 3.2. Comparison between Modflow and FC-results for synthetic generated data

Parameter	Result	
	Modflow	FC
s ($t=t_{EOP}$) (m)	3.82	-
s($t=t_{long}$) (m)	30.56	29.2
Q_sust (l s^{-1})	2	2.09

The recommended yield estimated with the FC-method is within 4.5 % of the Modflow solution. Further examples of the successful application of the method to various synthetic and real case field studies can be found in (Van Tonder *et al*., 1998 and Van Tonder *et al*., 2001).

CHAPTER 4

WELL PERFORMANCE TESTS AND NON-LINEAR RELATIONSHIPS BETWEEN DRAWDOWN AND ABSTRACTION RATE

4.1 INTRODUCTION

Step drawdown and multirate tests present convenient tools for the estimation of the long-term yield of boreholes. However, the analytical methods commonly employed for the analysis of such tests are all based on the assumption that the drawdown in a borehole is a linear function of the discharge rate. Numerous constant rate tests, of which a few are discussed in this chapter, have shown that this is not necessarily the case with boreholes drilled in the fractured formations of South Africa. The drawdowns in these boreholes are not only influenced by the peculiar geometry of the aquifers, but also the non-linear deformation of the aquifers during the pumping of a borehole. The two new non-linear models for the analysis of step drawdown and multirate tests introduced here, try to account for these factors, in particular the deformation of the aquifer, flow dimension and dewatering of discrete fractures. Although the model proposed for multirate tests is still based on constant time steps, the one for step drawdown tests allows the user to use arbitrary time steps, when performing the test in the field.

Non-linearities in drawdown curves should always be treated with caution, especially when used to assign sustainable yields for boreholes. However, the example of a step drawdown test performed at the Campus Test Site of the University of the Free State, shows that non-linearities can be addressed with an appropriate model.

4.2 STEP DRAWDOWN TESTS

Step drawdown tests were introduced by Jacob (1947) to study the influence that the discharge rate, Q, has on the drawdown, s(r, t), of the water level in a borehole. His conclusion, based on a number of drawdown tests, was that the observed drawdown consists of two components—one linear in Q and the other one non-linear. He also showed that the linear component can be divided into what he called 'the linear aquifer loss coefficient', which he denoted by the symbol $B_1(r_w, t)$, and a 'linear well loss coefficient, B_2, caused by the loss of energy in the borehole itself. The former of these components can be viewed as the drawdown one would observe if water could be withdrawn from an aquifer without the loss of energy represented by the term B_2. In other words, $B_1(r_w, t)$ can be interpreted as the theoretical solution of the groundwater flow equation for the actual, physical aquifer. It is consequently impossible (at least at this moment) to distinguish between the two linear losses in practice. Jacob therefore combined the two terms into the linear loss coefficient, defined by the equation

$$B(r_e, t) = B_1(r_w, t) + B_2$$

where r_e is known as the effective radius of a borehole, with physical radius r_w.

Jacob defined r_e as the radial distance from the vertical axis of the borehole to a point where the water level in the aquifer equals the water level in the borehole. This interpretation led him to describe the observed drawdown in a pumped borehole, s_w, with the equation

$$s_w = B(r_e, t)Q + CQ^2 \qquad (4.1)$$

where the term CQ^2 represents the non-linear losses.

The main effect of the non-linear losses is to drive the water level in the borehole down, without contributing to Q. This could not only affect the operational costs of a borehole adversely, but also cause irreparable damage to the borehole, pump and even the aquifer. It is therefore very important that one should never operate a borehole in such a way that the non-linear energy losses become dominant. However, it may sometimes be necessary to sacrifice energy for the borehole to perform optimally. Since this was the main motivation for Jacob to introduce step drawdown tests, it is not surprising to find that Eq. (4.1) can be very useful in this regard.

It is common practice to assume that the coefficient C in Eq. (4.1) is constant and attribute the existence of the term CQ^2 to turbulent flow, caused by the pump in and near the borehole (Helweg, 1994). However, there are indications that the drawdown is not only a function of Q, but also the geometry of the aquifer and that this may contribute to the non-linear term in Eq. (4.1) and cause the parameters B_2 and C to be time dependent. Helweg therefore suggests that Eq. (4.1) should be replaced by the equation

$$s = [A + B' \log(t)]Q + [C' \log(t)]Q^p \qquad (4.2)$$

which he claims is more general than Eq. (4.1). This is certainly true in the sense that Eq. (4.2) allows the coefficients to be time dependent. However, to achieve this he assumed that the theoretical drawdown, $B_1(r_w, t)$, could be represented by the Cooper-Jacob approximation of the Theis solution for an infinite uniform aquifer. Since this assumption is not necessary in Eq. (4.1), the possibility exists that Eq. (4.1) may describe the drawdowns of boreholes in heterogeneous aquifers better than Eq. (4.2), if the time is kept constant.

Another consequence of Helweg's assumption is that the flow towards the borehole must be radial, which need not be the case. This seems to be particularly the case with the shallow aquifers in the geological formations associated with the Karoo Supergroup in South Africa. These formations, which underlie approximately 50% of the country, consist mainly of sandstones, mudstones, shales and siltstones. The isostatic uplift of Karoo sediments and the intrusion of Drakensberg lavas and dolerites have fractured these formations, particularly the sandstone layers that are less elastic than the rest of the rocks in the Supergroup. Karoo aquifers therefore normally contain one (sometimes a few) bedding parallel fracture, as illustrated in Fig. 4.1, that serves as the main conduit of water for boreholes in the aquifers (Botha *et al.*, 1998). The drawdown observed during a constant rate test on such a borehole consequently displays a completely different behaviour than the drawdown in a conventional borehole. If borehole storage is neglected, the drawdown initially follows a linear trend, as shown in Fig. 4.2, which suggests that the borehole receives its water from the bedding plane fracture. This period is followed by one in which the flow is bilinear, when the borehole receives water from both the fracture and the rock matrix. Although the water level at first continue to decrease during this period, it ultimately tends to stabilise on or just above the bedding parallel fracture, provided that the discharge rate of the borehole does not exceed the rate at which the matrix can recharge the fracture. Otherwise, the water level will begin to decline again and stabilizes above another fracture (if one is present), or simply drop to the pump intake. The drawdowns observed in Karoo boreholes therefore depend not only on the discharge rates of the boreholes, as implied in the derivations of Eqs. (4.1) and (4.2), but also on the geometry of the aquifer.

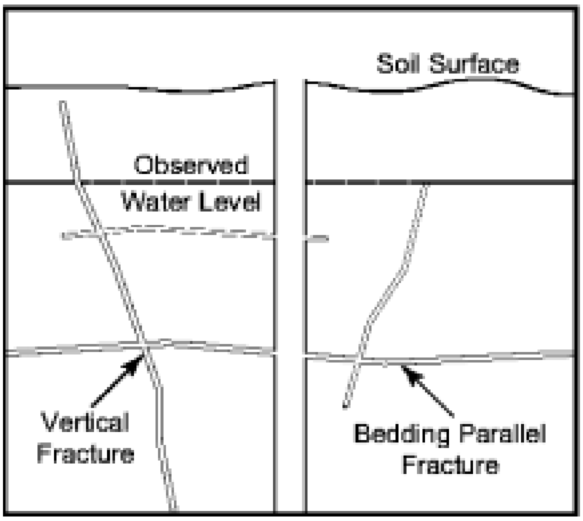


Fig. 4.1 Schematic cross-section through a typical Karoo aquifer

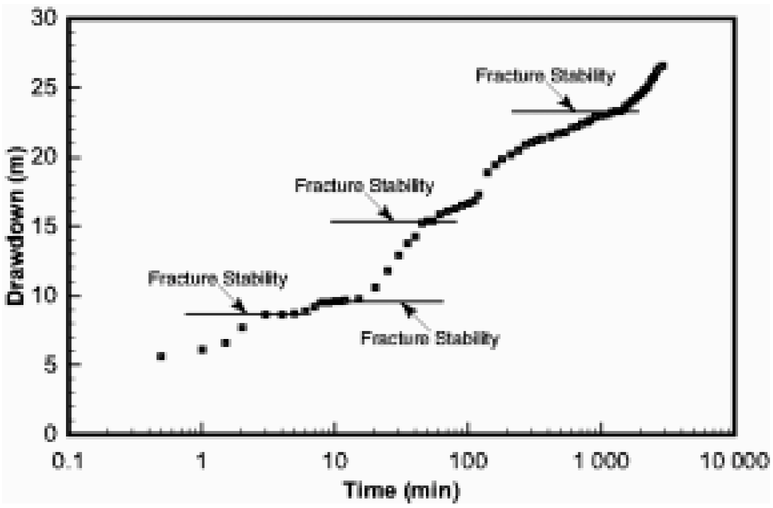


Fig. 4.2 The drawdown observed during a constant rate test on a borehole in the Karoo Supergroup pumped at a constant rate of 15 L s^{-1}

The dependence of the observed drawdowns in Karoo boreholes on the geometry of the aquifer is not restricted to the dewatering of fractures alone, as illustrated by the results of two constant rate tests performed on borehole UO5 on the Campus Site with discharge rates of 0.5 L s^{-1} and 1.25 L s^{-1}. In these tests, water levels were monitored simultaneously in UO5 and a piezometer installed in borehole UO6, situated 5 m from UO5. The piezometer was installed 2 m above the bedding parallel fracture that intersects both UO5 and UO6 at a depth of 23 m below surface within a sandstone layer that contains the fracture. The results are summarised in Table 4.1 and Fig. 4.3.

Table 4.1 Water levels observed in Borehole UO5 on the Campus Test Site and a piezometer installed in Borehole UO6, 5 m from UO5, during two pumping tests with different abstraction rates

	Q	Drawdown after 1 Day (m)	
	(L/s)	UO5	Piezometer
	0.50	1.72	0.024
	1.25	4.82	0.156
Ratio	2.50	2.80	6.500

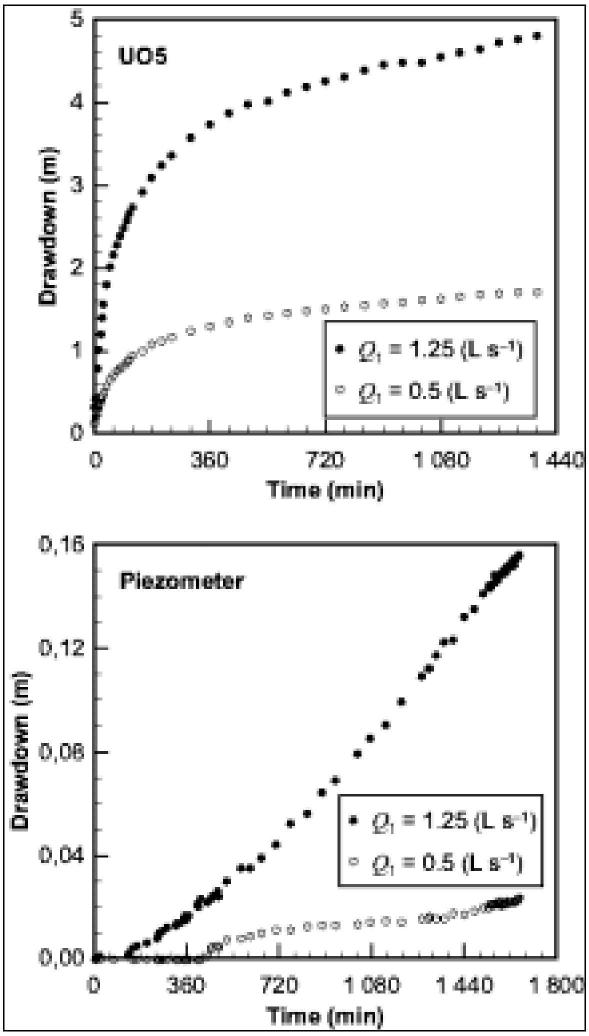


Fig. 4.3 Drawdowns observed in UO5 and the piezometer installed in UO6, 5 m from UO5, when UO5 was pumped at the given rates

As shown in Fig. 4.3, the water level in UO5 never dropped below the fracture in both tests, while the ratio of its water levels (after one day of pumping) in Table 4.1 is also very similar to the ratio of the discharge rates. However, the drawdowns observed in the piezometer differ by a ratio of 6.5. This behaviour can be briefly explained as follows.

It is known that the rocks serve as the main reservoir for water in Karoo aquifers. However, the vertical and horizontal hydraulic conductivities of the Karoo rocks are very low (~10^{-7} m s^{-1} to 10^{-8} m s^{-1}). The bedding parallel fractures, on the other hand, have very high horizontal hydraulic conductivities (~10^{-4} m s^{-1}) and can consequently transmit large quantities of water quickly. Very little water therefore flows through undisturbed Karoo aquifers under natural conditions. It is only when the piezometric pressure in a bedding parallel fracture is disturbed that the vertical piezometric gradient and gravity force the water to flow from the rock matrix to the fracture. The main direction of flow in these aquifers is therefore vertical and linear, and not horizontal and radial as in conventional aquifers (Botha *et al.*, 1998).

Because the flow is vertical, a Karoo borehole will first dewater the rock matrix in its immediate vicinity, before it begins to dewater the matrix at more distant points. One would, of course, expect the opposite situation to arise after the pump has been switched off that the water levels in the pumped borehole and surrounding rock matrix will restore more rapidly than water levels at more distant points in the aquifer. It is therefore very interesting to note that although the water levels in UO5 restored very quickly, the piezometer level in UO6 continued to decline for 5 days, after the pump was switched off in the 0.5 L s^{-1} test, and 14 days after the 1.25 L s^{-1} test. The water levels near the borehole therefore not only restore first, but also at the expense of the piezometric levels within the rock matrix. However, there are indications that this delay in the restoration of the water levels in the matrix is enhanced by the restoration of the fracture geometry, which was deformed during the pumping operations.

The previously described heterogeneous behaviour of the water levels has been observed in many constant rate tests performed in Southern Africa. The results of four of these tests are summarised in Table 4.2 and illustrated graphically in Fig. 4.4. The first test was performed on a borehole in the mudstone aquifer at Meadhurst (just west of Bloemfontein) and the second on a borehole in the calcrete aquifer of Khorixas in Namibia. The other two were performed on the boreholes Zonnebloem 1 and 2 of the Middelburg Municipality in the North Cape Province.

There is a possibility that the sharp increase in the drawdown of the Khorixas borehole at the late times in Fig. 4.4 was caused by boundary effects. The reason for this believe is that the increase in the drawdown is smooth and not stepwise as in the other boreholes.

Table 4.2 Ratios of the abstraction rates, Q, and associated drawdowns, $s(t_d)$, observed during pumping tests with durations t_d on four boreholes in Southern Africa

Meadhurst	**t_d 950 min**		**Khorixas**	**t_d 420 min**	
	Q (L s^{-1})	**$s(t_d)$ (m)**		**Q (L s^{-1})**	**$s(t_d)$ (m)**
	1.5	4.24		1.66	1.9
	2.0	6.83		4.1	8.8
Ratio	1.33	1.61	**Ratio**	2.47	4.6
Zonnebloem 1	**t_d 540 min**		**Zonnebloem 2**	**t_d 540 min**	
	Q (L s^{-1})	**$s(t_d)$ (m)**		**Q (L s^{-1})**	**$s(t_d)$ (m)**
	14	5.68		15	4.01
	19	9.98		35	12.43
Ratio	1.36	1.75	**Ratio**	2.33	3.01

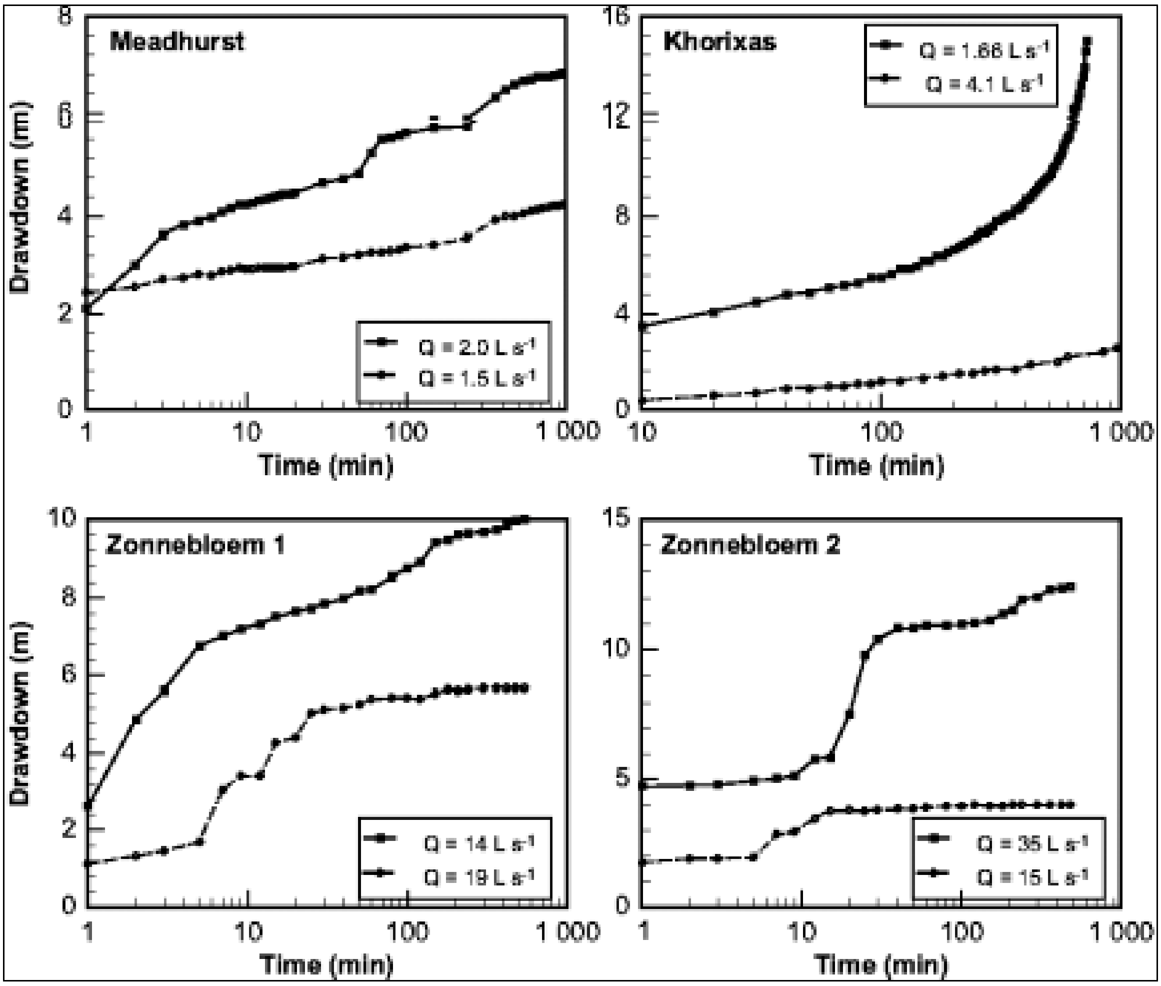


Fig. 4.4 Drawdowns observed during constant rate tests at Meadhurst, Khorixas and the boreholes Zonnebloem 1 and 2 of the Middelburg Municipality

The previous examples clearly suggest that Eqs. (4.1) and (4.2) cannot fully account for the heterogeneities in fractured-rock aquifers, caused by their particular geometry. A new method was therefore developed for the analysis of these boreholes. However, the idea behind the method may be better understood if one has a good understanding of how these tests are performed and the factors that may influence the dependence of s_w on the discharge rate. The discussion that follows therefore begins with a brief discussion of the basic principles that underlies the application of Eqs. (4.1) and (4.2) and the methods conventionally used to analyse the yields of boreholes. This is followed by a discussion of the various factors that may influence the behaviour of boreholes, the new approach developed for fractured aquifers and the application of the method to a typical borehole in a Karoo aquifer.

4.3 PRINCIPLES AND METHODS USED IN THE ANALYSIS OF BOREHOLE YIELDS

It is well-known that Eq. (4.1) can only be applied in practice once a suitable time has been chosen for a drawdown test (Helweg, 1994). The reason for this is that the coefficient $B_1(r_w, t)$ in Eq. (4.1) is time dependent. Its contribution to s_w can therefore only be neglected if the tests are performed for the same period. However, there is another implicit assumption in the equation that is often overlooked—the assumption that the theoretical drawdown can be expressed in the form

$$s(r_w, t) = QB_1(r_w, t)$$

That this is indeed an assumption follows directly from the observation that the discharge rate appears nowhere in the equation universally accepted as the one that describes the flow of groundwater (Bear, 1972)

$$S_0 D_t \varphi(\mathbf{x}, t) = \nabla \cdot [\mathbf{K} \nabla \varphi(\mathbf{x}, t)] + f(\mathbf{x}, t) \qquad (4.3)$$

This equation only contains the specific storativity, S_0, hydraulic conductivity, K, piezometric head, φ, and the strength of the sink (or source), f(**x**, t), apart from the usual spatial and time variables and their derivatives. Since it is impossible to determine the strength of the sink (borehole) with the methods available today, a borehole is commonly regarded as a line sink, and f(**x**, t) expressed as

$$f(x, t) = Q\delta(x - x_0)\delta(y - y_0) \qquad (4.4)$$

where $\delta(z - z_0)$ is the well-known Dirac delta function, and (x_0, y_0) the horizontal position of the borehole. However, this immediately removes any dependence of $s(r_w, t)$ on the heterogeneity of the aquifer and forces it to be a linear function of Q, which needs not be the case in general, as illustrated by the previous discussion of the drawdowns observed in fractured boreholes. Unfortunately, Eq. (4.4) probably represents the best assumption until it becomes possible to quantify f(**x**, t) directly. One must therefore always apply Eqs. (4.1) and (4.2) with care.

4.3.1 Field Methods Used to Estimate Borehole Losses

There are two field methods for the estimation of well losses—the well-known step drawdown test and the multirate test. The main difference between the two tests is that the water level is allowed to recover between the steps of a multirate test, while the step drawdown test is a continuous test, as illustrated by the schematic drawdowns in Fig. 4.5.

It is, in principle possible to disrupt the restoration of the water level in multirate tests. However, it must be kept in mind that the solution of Eq. (4.3) for variable discharge rates has a memory, which needs to be taken into account in the analysis of the results, as shown by the various methods, such as the Hantush-Bierschenk method, often used to analyse step drawdown tests (Kruseman and De Ridder, 1991). A similar procedure will not only complicate the analysis of a multirate test, but also affect the reliability of the results adversely.

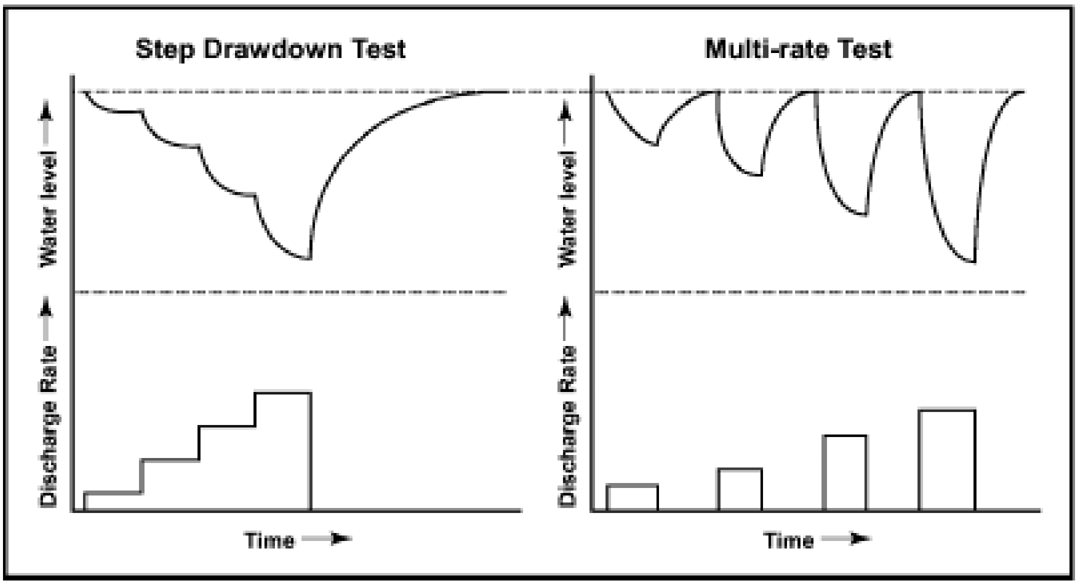


Fig. 4.5 Schematic illustration of the difference in drawdowns observed during a step drawdown and a multirate test

Multirate tests are not much favoured in groundwater hydraulics, because of the extensive periods required for the tests. However, they can be used to determine the coefficients in Eqs. (4.1) and (4.2) directly. For example, consider the case where two constant rate tests have been performed on the same borehole, one with a discharge rate Q_1 and the other with a discharge rate Q_2. Let s_1 and s_2 represent the observed drawdowns after a fixed time, t_f, and r_1 and r_2 the ratios (s_1/Q_1) and (s_2/Q_2) respectively. It is then not difficult to show that the coefficients $B(r_e, t)$ and C in Eq. (4.1) must satisfy the relations

$$B(r_e, t_f) = \frac{Q_2 r_1 - Q_1 r_2}{Q_2 - Q_1} \qquad C(t_f) = \frac{Q_1(r_1 - r_2)}{Q_2 - Q_1}$$

If applied at different periods, t_f, these expressions can also be used to determine the dependence of $B(r_e, t)$ and $C(t_f)$ on t, without any assumption on the nature of $B(r_e, t)$.

4.4 NON-LINEARITIES IN THE DRAWDOWN-DISCHARGE CURVE

As mentioned in the introduction, the non-linear term, CQ^2 in Eq. (4.1) and $C'\log(t)Q^p$ in Eq. (4.2), is conventionally ascribed to turbulence caused by the pumping of the water. However, there are three other phenomena that may also contribute to the non-linear behaviour.

1 Dewatering of discrete fractures.

2. Deformation of the fractures and/or the rock matrix.

3. A phreatic or water table-aquifer.

Turbulence

Turbulence is a characteristic property of any fluid flowing across an obstacle, caused by the interaction between the molecules of the fluid and obstacle, and depends essentially on the roughness and size of the obstacle and the flow velocity. Since turbulence is a very common phenomenon in pipe flow, geohydrologists mainly associate it with fractures and sinkholes. However, two other factors may also contribute to turbulence in a producing borehole—a high discharge rate and a restrictive entry to the borehole. The first of these factors can be controlled by using a judiciously chosen discharge rate (the main reason why Jacob introduced step drawdown tests), but the second factor presents some difficulties.

There are a number of reasons why a restrictive entry may develop in a borehole, such as the clogging of the natural pores by drilling mud, fine-grained particles deposited by percussion drills and the installation of gravel packs. This usually results in the formation of a zone, commonly referred to as a skin, in the immediate domain of the borehole whose hydraulic conductivity, K_s, differs markedly from the hydraulic conductivity, K, of the aquifer outside the skin. Such a skin can be conveniently characterised by the so-called skin factor

$$\xi = \{\frac{K}{K_s} - 1\}\log(\frac{r_s}{r_w}) \qquad (4.5)$$

where r_w is the radius of the borehole and r_s that of the skin, assuming that the skin does not store water. If it is further assumed that the water levels near the borehole could be described by the Cooper-Jacob approximation, the skin factor can be used to describe Jacob's effective radius, r_e, in Eq. (4.1) with the equation (Matthews and Russell, 1967)

$$r_e = r_w \exp(-\xi)$$

The skin factor can be positive or negative, as illustrated in Fig. 4.6. A positive skin factor indicates that the drawdown in the borehole is more than the drawdown expected theoretically for the aquifer and a negative skin factor that the drawdown is less than the theoretical drawdown. A positive skin factor often arises when the skin have been clogged during the drilling operations, while a negative skin factor indicates that the skin is well-developed. The latter situation often arises where a gravel pack has been installed around a producing borehole, but has also been observed in constant rate tests performed on boreholes that intersect horizontal fractures in fractured aquifers.

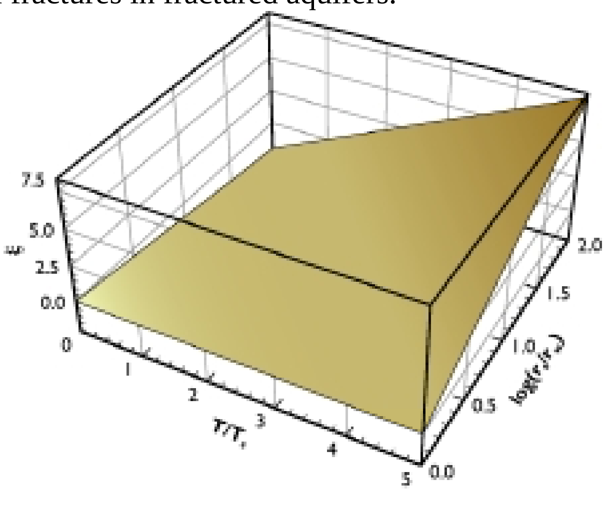


Fig. 4.6 **Graph of the skin factor, ξ, as a function of the ratios T/T_s=(K/K_s) and log(r_s/r_w) in Eq. (4.5)**

As shown in Fig. 4.2 and 4.4, the observed drawdown rate in a borehole intersected by a horizontal fracture remains practically constant if the borehole is pumped at a rate that can be sustained by the fracture, but increases sharply if this rate is exceeded. Although the increase in the drawdown rate may be viewed as a non-linearity, it must be remembered that the dewatering of a fracture is largely controlled by the hydraulic conductivity of the rock matrix surrounding the fracture (Botha *et al.*, 1998). This behaviour is similar, but not physically equivalent, to that observed in an aquifer with one or more impermeable boundaries, where the drawdown rate also increases sharply once the pumping begins to influence the water levels on its boundaries. (See for example the drawdown curve for the Khorixas borehole in Fig. 4.4.) One approach to compute discharge rates from these drawdowns would be to use the simple graphical technique, illustrated in Fig.4.7, often employed in the analysis of aquifers with impermeable boundaries. However, as a comparison of the drawdowns of the Khorixas and Zonnebloem 2 boreholes in Fig. 4.4 shows the drawdown rate for the fractured Zonnebloem 2 borehole is larger than that of the Khorixas borehole. The previous procedure may therefore not be very suitable in the case where a large increase in the drawdown rate is caused by the dewatering of a fracture.

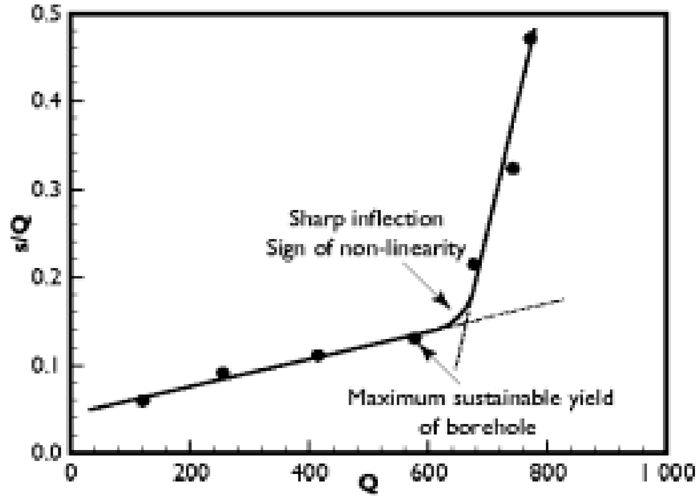


Fig. 4.7 A graph of normalised drawdown, s/Q, as a function of the discharge rate, Q, used in the analysis of drawdowns in an aquifer with one or more impermeable boundaries

Previous experience with constant rate tests on boreholes in fractured-rock aquifers has shown that the non-linear behaviour will always be observed, if the discharge rate of the producing borehole is high enough. Unfortunately, it is very difficult to predict when and at what discharge rate the non-linear behaviour will begin, since that is determined by the areal dimension of the fracture (Gringarten and Ramey, 1974). It may thus take a long time and a large number of discharge rates to determine the position of the non-linear behaviour in a borehole that intersects an extensive fracture.

It is tempting to assume that the presence of a prolonged period during which the drawdown remains constant, signifies that the discharge rate used in the test represents an acceptable measure for the long-term yield of the borehole. Unfortunately, this is not necessarily the case. For, as shown by the drawdown curves of borehole Zonnebloem 2 in Fig. 4.4, both the length of the period and the depth at which the water level stabilises depend on the discharge rate and the position of the fracture. The possibility therefore exists that one may easily over- or underestimate the yield of the borehole, by concentrating on the presence of such a period.

This was the main reason for introducing the generalised solution for step drawdown tests described below.

Deformation

It may not always be appreciated, but the only reason why groundwater can be withdrawn from the subsurface of the earth is that all the geological formations on earth (and water) are compressible. The result is that an aquifer will always deform to some extent when water is pumped from a borehole drilled into the aquifer. If the stress strain relation for the formations that govern the deformation is linear, that is obeys Hooke's law, the formations will restore to their original form once the pumping is stopped, otherwise it will continue to deform with time. Although Eq. (4.3) accounts for linear deformation in the vertical direction, through the appearance of the compressibility coefficients of the rock and water in the specific storativity (Bear, 1972), it completely neglects horizontal and non-linear deformations. This aspect is currently investigated by Cloot and Botha (2000) with a numerical model for an aquifer consisting of a central sandstone layer bounded on the top and bottom by mudstone layers. Their results can be briefly summarised as follows.

When pumping starts, perturbations in the fluid pressure that develop near the wall of the borehole propagate rapidly through the aquifer, causing deformations in both the horizontal and vertical directions, but on different scales. The horizontal displacements develop uniformly throughout the thickness of the aquifer, but the vertical displacements concentrate on the sandstone-mudstone interfaces. The maximum amplitude of the horizontal deformation, which remains more or less constant, propagates slowly from the borehole wall into the aquifer. The vertical displacement, on the other hand, remains at the interfaces where both its maximum amplitude and extent slowly increases with time. As could have been expected, the deformation does not affect the piezometric head in the different layers adversely at the beginning, but its effect becomes more noticeable with time. However, the linear stress strain relation ensures that all deformations disappear once the pumping is stopped.

The same situation also develops in the case where the stress-strain relation is non-linear, as long as the deformation is restricted to the linear leg of the relation. However, the deformation amplitudes, particularly that of the vertical displacements, quickly begin to exceed those of the linear stress strain relation, once the strains exceed the elastic limit. Moreover, the aquifer is not restored to its original dimensions after the pumping has stopped. Since the magnitudes of the pressure perturbations (thus the deformations) are essentially functions of the discharge rate, there is a possibility that a too high discharge rate may not only damage the aquifer permanently, but ultimately also causes it to collapse. This applies in particular to any fracture (vertical or horizontal) in the aquifer.

It is important to note that the relation between the computed piezometric head and the discharge rate is always non-linear, even in the case of the linear stress strain relation. Deformation of the aquifer could therefore contribute significantly to the non-linear terms in Eqs. (4.1) and (4.2), as mentioned by Helweg (1994). A common rule of thumb to regard a borehole with a coefficient C $[d^2/m^6]>10^{-7}$ as one that could be developed to make it more efficient may therefore not be valid for these boreholes.

Phreatic Aquifers

The major characteristic of a phreatic or water-table aquifer, and the one that distinguish it from all other types of aquifers, is that the water table will move in or out of the unsaturated zone that overlies the water level in such an aquifer. This type of aquifer therefore has to be described by the unsaturated-saturated flow equation

$$[S_w S_0 + C(\varphi)]D_t \varphi(\mathbf{x},t) = \nabla \cdot [\{\mathbf{K}(\varphi)\nabla \varphi(x,t)\}] + f(\mathbf{x},t)$$

where S_w is the water saturation and C(φ) the moisture capacity, while the other symbols have the same meaning as in Eq. (4.3). This is a highly non-linear equation in the mathematical sense in that it does not obey the principle of superposition (Cakmak and Botha, 1995), since both C(φ) and K(φ) are functions of the piezometric head. However, this does not imply that the drawdown in such an aquifer will be a non-linear function of Q, as is often believed. Indeed, it is not difficult to see that the latter relation will still be linear if the source strength, f(**x**, t), is approximated with Eq. (4.4). One reason for the believe that the drawdown in these aquifers is a non-linear function of the discharge rate, is probably because such aquifers are often viewed as a confined aquifer in which the transmissivity, T, varies with the saturated thickness of the aquifer. However, as shown by the Dupuit formula for phreatic aquifers (Kruseman and De Ridder, 1991), this would imply that the drawdown should behave as a function of $Q^{1/2}$ and not Q^p, with p > 1, as is conventionally assumed for the non-linear term in both Eqs. (4.1) and (4.2).

The Fractal Behaviour of Drawdown

A common implicit assumption in the analysis of drawdown tests is that all points in an aquifer are equally accessible to the borehole. In other words, the rate at which water will flow from a point A in the aquifer to the borehole will depend only on the discharge rate of the pump and not on the geometry of the aquifer in the vicinity of A. However, this may not be the case in Fig. 4.8 where UO5 will more likely withdraw water from point A than point B, even though B is situated closer to UO5. This behaviour prompted Barker (1988) to introduce what he calls the flow dimension for flow through a fracture—a concept based on his fractalization of the spherical surface element (but not the spherical space itself). This allows him to express the drawdown in a uniform infinite aquifer in his hybrid space as

$$\varphi(r,t) = \frac{Qr^{2\nu}}{4\pi^{1-\nu}K_f b^{3-n}}\Gamma(-\nu,u);\ \nu < 1; \ldots\ldots u = \frac{r^2 S_f}{4K_f t}$$

where:

r	= the radius vector in spherical space
n	= the flow dimension
S_f, K_f	= the specific storativity and hydraulic conductivity of the fracture
ν	= 1 – n/2
b	= a parameter that represents the thickness of the aquifer in radial two-dimensional space
Γ(u, z)	= the incomplete gamma function

and the other symbols have the same meaning as defined previously.

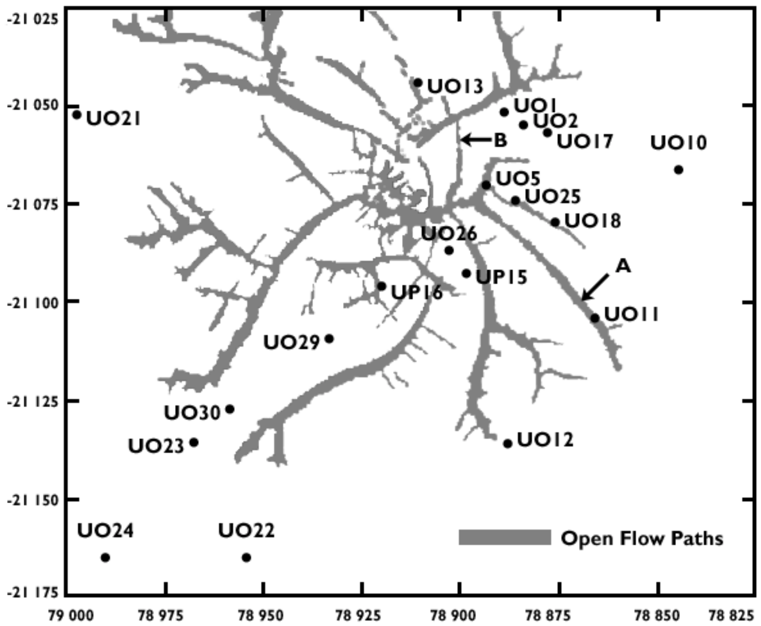


Fig. 4.8 **Conceptual model of the horizontal fracture in the sandstone layer that forms the major aquifer at the Campus Test Site (adapted from Van der Voort and Van Tonder , 2000)**

4.5 APPROXIMATION OF THE DRAWDOWNS IN FRACTURED AQUIFERS

The discussion above shows that neither the Jacob or Helweg expressions in Eqs. (4.1) and (4.2) can fit the observed drawdowns of boreholes in fractured aquifers, nor account for the possible non-linear deformation of such an aquifer, or fractal flow. Since it was not obvious how to include these factors into either the Jacob or Helweg equations, a heuristic approach was used to try and adjust these equations for flow in fractured aquifers.

The recent model of Cloot and Botha (2000) for a horizontal fracture suggested that one can account for the non-linear deformation by splitting the non-linear term in Jacob’s equation into two parts—one accounting for the usual effects of turbulence and the other one for the deformation. The model also suggested that the contribution of non-linear deformation will be non-linearly proportional to that of turbulence and time-dependent. Jacob’s equation was consequently modified to read

$$s(t) = B(r_e, t)Q + [EQ^{p=e-1} + C^{'}Q^{p}]\log(t) \tag{4.6}$$

where the non-linear term has been replaced by the one suggested by Helweg. The exponent p + e – 1 in the term EQ^{p+e-1} log(t), introduced to account for non-linear deformation of the aquifer, was chosen in such a way that Eq. (4.6) reduces to Eq. (4.1) for a fixed time if there is no deformation (or no dewatering of fractures), that is e = 1. The choice was further motivated by the fact that the model of Cloot and Botha indicated that the drawdown will increase if the deformation tends to close the fracture (e > 1). A negative exponent, e, would therefore indicate an opening of the fracture, which is not impossible (Botha *et al.*, 1998).

The analysis of a large number of constant rate and step drawdown tests performed on boreholes in the fractured formations have shown that Eq. (4.6) cannot fully account for the observed drawdowns. However, further numerical experiments indicated that this can possibly be ascribed to the fractal structure of the fractures in these aquifers (see Fig. 4.8) and that the equation

$$s(t) = B(r_e, t)Q + EQ^{p+e-1}(\log t)^{\Gamma(n/2)} + C^{'}Q^{p}\log(t) \qquad (4.7)$$

with Γ(m) the normal Gamma function and n Barker's fractal dimension, provides a better approximation for these aquifers.

The approximation in Eq. (4.7) is very similar to Eq. (4.1) and, in fact reduces to it, if e = 1, n=2 and the time is constant. It therefore shares all the advantages of Eq. (4.1), discussed above, but also the same disadvantages, the most serious of which is that it requires constant time steps in step–drawdown or multirate tests. However, the numerical experiments indicated that the same results can also be obtained with the equation

$$s(t) = AQ + B^{'}Q^{e}(\log t)^{\Gamma(n/2)} + C^{'}Q^{p}\log(t) \qquad (4.8)$$

which arises from Eq. (7) if $B(r_e, t)$ is approximated as

$$B(r_e, t) = A + Q^{e-2}(B^{'}Q - EQ^{p})(\log t)^{\Gamma(n/2)}$$

Since A is a constant, Eq. (4.8) can be applied to step–drawdown or multirate tests with variable time steps. Unfortunately, there is a price one has to pay in using Eq. (4.8) (or Eq. (4.7) for that matter), in that it is no longer possible to use the principle of superposition in the analyses of step drawdown or multirate tests. The reason for this is that both the B' and C' terms contain fractional exponents of Q, and therefore cannot be linearised by dividing the equation with Q. Moreover, the objective function that arises if one attempts to fit Eq. (4.8) with a non-linear least squares method to the observed drawdowns is non-convex (this also applies to Helweg's equation). An interactive method, called the non-linear FC-method, was therefore developed and implemented in the FC-program. The workbook also allows the user to use a non-linear least square fit, if required, but this is not recommended for someone who does not have experience with non-linear least squares approximations.

There are six unknown parameters in Eq. (4.8) that should be estimated which makes the solution non-unique. To obtain a more unique solution, the following steps are proposed:

- The coefficient A could be obtained by using the data of the first step which, if plotted in a $t^{0.5}$ versus drawdown plot (squared root of time), will yield the value of the skin (i.e. if no skin exists, the data will start at the origin of the plot; $A=s_{add}/Q$ in Eq. (2.10) in Chapter 2).
- Turbulence is due to high flow velocities according to $0.5v^2$, which implies that a value of p=2 is sufficient to account for the turbulent term.

Using of the above two rules implies that only four parameters have to be estimated.

It is also possible to estimate a T-and S-value from the generalised step test by using the Birsoy-Summers method (1980, see e.g. Kruseman and De Ridder, p. 181). Due to the changing of rates a time correction must be performed. The t in the Cooper-Jacob equation is replaced with a correction time $\beta_{t(n)(}t\text{-}t_n)$:

$$\beta_{t(n)}(t-t_n)=\prod_{i=1}^{n}(t-t_i)^{\Delta Q_i/Q_n} \qquad (4.9)$$

where

t_i = time at which the i-th pumping period started
Q_i = constant discharge during the i-th pumping period
ΔQ_i = discharge increment beginning at time t_i

The application of the method is exactly the same as the Cooper-Jacob method. The only difference is that a graph of s_i/Q_i is drawn against the corrected time. For the estimation of the S-value, the effective borehole radius must replace the real borehole radius.

4.6 A CASE STUDY

It is not possible to describe all the step drawdown tests that have been used in developing Eq. (4.8). The present discussion will therefore be concluded with just one example, the step drawdown test performed on Borehole UP16 on the Campus Test Site (see Fig. 4.8 for its position). This borehole intersects the bedding parallel fracture on the site at a depth of 21 m below the surface, while its rest water level at the time of the test was 13.2 m below the surface. Table 4.3 lists the discharge rates and drawdowns observed during the four steps used in the test. The observed drawdowns are also compared graphically with the interactive fit to Equation (4.8) in Fig. 4.9.

The coefficients determined from the fit of s(t) (expressed in m), Q (expressed in $m^3\ d^{-1}$) and t (expressed in d) to Eq. (4.8), are listed in Table 4.4. These values were used to estimate the drawdown in the borehole for a period of 2 years, such that the drawdown would not reach the position of the bedding parallel fracture, in other words s should not exceed 7.8 m (= 21 m – 13.2 m). This yielded a discharge rate of 0.5 L/s, which agrees excellently with the value of 0.48 L/s, obtained by Van Tonder *et al.* (2001) with the normal FC-method.

Table 4.3 Step sizes and discharge rates used during the step drawdown test performed on borehole UP16

Step 1		Step 2		Step 3		Step 4	
Q = 0.61 (L s^{-1})		Q = 1.18 (L s^{-1})		Q = 2.00 (L s^{-1})		Q = 3.50 (L s^{-1})	
Time (min)	s (m)	Time (min)	s (m)	Time (min)	s (m)	Time (min)	s (m)
0	0.000	22	0.730	47	1.660	65	3.150
2	0.170	23	0.780	48	1.740	66	3.300
3	0.190	24	0.830	49	1.810	67	3.440
4	0.230	25	0.870	50	1.875	68	3.560
5	0.265	26	0.910	51	1.940	69	3.670
6	0.295	27	0.960	52	1.995	70	3.770
7	0.320	28	0.990	53	2.050	71	3.870
8	0.350	29	1.010	54	2.100	72	3.970
9	0.375	30	1.045	55	2.150	73	4.085
11	0.425	31	1.070	56	2.200	74	4.175
12	0.450	32	1.100	57	2.255	75	4.260
13	0.460	33	1.135	58	2.290	76	4.350
14	0.480	34	1.160	59	2.325	77	4.430
15	0.500	35	1.180	60	2.370	78	4.510
16	0.520	36	1.200	61	2.405	79	4.585
17	0.535	37	1.230	62	2.450	80	4.660
18	0.550	38	1.250	63	2.490	81	4.730
19	0.565	39	1.275	64	2.520	82	4.800
20	0.585	40	1.290			83	4.870
21	0.600	41	1.315			84	4.940
		42	1.335			85	5.000
		43	1.355			86	5.060
		44	1.370			87	5.120
		45	1.390			88	5.180
		46	1.410			89	5.240
						90	5.290
						95	5.540
						97	5.640
						100	5.760
						105	5.960
						113	6.220
						114	6.270
						115	6.290
						116	6.310
						117	6.340
						118	6.370
						119	6.400
						120	6.430

Table 4.4 Values of the coefficients that describe the fit of Eq. (4.8) to the observed drawdowns of UP 16 in Fig. 4.9

A	B	C	p	n	e
$7.5 \cdot 10^{-4}$	$4.36 \cdot 10^{-3}$	$2.88 \cdot 10^{-6}$	2.0	1.48	1.12

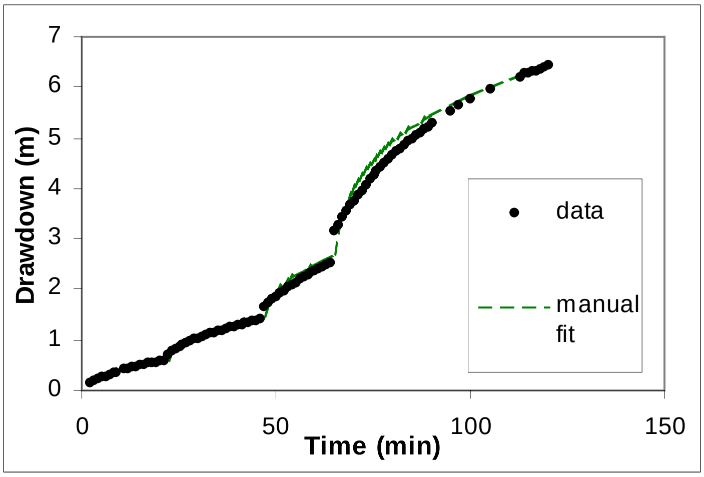


Fig. 4.9 **Graph of Eq. (4.8) when fitted to the drawdowns observed during the step drawdown test performed on Borehole UP16**

Drawdown tests are frequently the only tool available with which to assign sustainable yields for boreholes. This applies in particular to boreholes in the Karoo formations of South Africa. Unfortunately, the drawdown curves in these boreholes regularly display a non-linear behaviour that cannot be attributed to turbulent effects in the borehole alone. Two new generalised equations (Eqs. 4.7 and 4.8) were therefore derived in this chapter to analyse step drawdown test data in these aquifers.

There is no doubt that Eq. (4.7), applied to a series of at least six (preferably more) multirate tests, will yield the most reliable results. However, these tests can be very time- consuming and expensive. In such cases, the user may find the approximation in Eq. (4.8), which allows the use of the more economical step drawdown tests with variable time steps, more suitable. However, the yield estimated from this analysis should preferably be monitored for some time after the borehole is used for production purposes.

Non-linearities in drawdown curves should always be treated with caution, especially when used to assign sustainable yields for boreholes. This applies, in particular, to the extrapolation of the results into domains not covered during the test. The recommendation by Helweg (1994) that the tests should be designed to reach the maximum possible drawdown and discharge rate, cannot therefore be overemphasised. In the case of boreholes in fractured-rock aquifers, the maximum drawdown is when the water level reaches the main water-bearing fracture. This depth should never be exceeded, not even during the drawdown test, if one does not want to damage the borehole.

CHAPTER 5

DELINEATION OF BOREHOLE PROTECTION ZONES

5.1 INTRODUCTION

Results from many tracer tests indicate that the minimum distance between a pit-latrine and borehole as proposed by Xu and Braune (1995) will not be adequate in many practical cases to protect the borehole from being polluted. The idea of three protection zones, like in Germany (Kinzelbach *et al*., 1991), for boreholes at on-site sanitation areas was proposed by De Lange (1999).

The U.S. Environmental Protection Agency (EPA) was also involved in delineating well head protection areas (WHPA). In order to achieve the overall goal of delineating the zone of influence (ZOI) or the zone of contribution (ZOC) of a well in an unconfined fractured-rock aquifer, the methods used were (EPA, 1991):

1. Arbitrary fixed radius.
2. Calculated fixed radius.
3. Vulnerability mapping.
4. Flow-system mapping:

   -with time of travel (TOT) calculations.

   -with analytical equations.

5. Residence time approach.
6. Numerical flow/transport modelling.

After the WHPA is determined, the following steps are proposed by the EPA to protect the groundwater resource (Xu, 1998):

1. Identifying potential sources of contamination in the WHPA.
2. Establishing management approaches to protect the groundwater in the WHPA.
3. Developing a contingency plan for pollution events.
4. Instituting programs for public education and participation.

5.2 PROTECTION ZONES

5.2.1 Protection Zone I: Fencing

For protection zone I (i.e. the immediate fenced area around the borehole), it is proposed that the distance of the fence around the borehole must be at least 5 m. For a borehole that is supplying water to less than say 20 persons, a well-constructed sanitary seal is regarded as enough. Quality monitoring is, however, very important.

5.2.2 Protection Zone II: bacterial and nitrate pollution

A second protection zone around the borehole is proposed. The idea with this zone is to protect the drinking water from microbial (bacteria and viruses) and nitrate pollution. Many case studies have shown that bacteria usually die within 30 days after being introduced into the soil. For the delineation of this zone, Table 6 in the report by Xu and Braune (1995) will be adequate in some cases. They proposed an absolute minimum distance of 50 m between a pit-latrine and a borehole. However, in many cases in fractured aquifers, this will not be adequate, as shown in a following section.

In the case of the delineation of protection zone II, the areal extent of the fracture is very important, and a fracture could be viewed as an extended borehole with a very high T-value and a small S-value. It is thus very important to estimate the areal extent of the fracture. Once bacteria or an element reaches the fracture, its movement will be very rapid towards the abstraction borehole and one of the major mechanisms that will play a role in bacterial die-off time, is the distance from the ground surface to the water level. If a vertical fracture intersects a pitlatrine, the movement of the pollutant to the water level could be very rapid, even in the case of a very deep water level. This illustrates the difficulty to estimate travel times from the surface to the water level in a fractured aquifer. For this reason it is proposed that the whole domain above the fracture be regarded as protection zone II and must be protected. In the following section, a method to estimate the size of this capture zone is discussed.

General

A very good idea of the areal size of the fracture could be obtained by making use of early pumping test data. Considering early data of a pumping test, it was found that the ratio s/Q (i.e. drawdown in abstraction borehole/ abstraction in L/s) after 1 minute of abstracting water from a borehole, gives a good first approximation of the extent of the fracture and the following generalisation applies:

1. If $s/Q < 0.5$ very good fracture extent
2. If $s/Q < 1$ but > 0.5 good fracture extent
3. If $s/Q > 1$ limited fracture extent

For a qualified estimation of the extent of the fracture the knowledge of the following parameters are required, which can be computed from the early data of the hydraulic test:

- T-value of the fracture
- S-value of the fracture

Estimating the T-value of the fracture

The following equation (Equation 5.1 - adapted Logan equation) will give an estimate of the T-value of the fracture:

$$T = 10^{(Log\frac{104Q}{s} + Log15Q)/2} \qquad (5.1)$$

where T = transmissivity in m^2/d; Q = abstraction rate in L/s; s = drawdown in m after 1 minute.

Estimating the S-value of the fracture

For the estimation of the S-value of the fracture, two other parameters must first be determined:

First, the theoretical specific storage (S_s) must be determined. The S_s is calculated by applying Equation (5.2) (Kruseman and De Ridder, 1994).

Secondly the thickness of the fractured zone (D) must be determined (Eq. 5.3).

<u>Estimation of S_s.</u>

$$S_s = \rho g\,(\alpha + n\beta) \qquad (5.2)$$

The following values for n and α (obtained from inverse modeling and tracer tests) are proposed:
ρg= 9804 N/m^3 (specific weight of water)
n = 0.13 (porosity)
$\alpha = 5.56 \times 10^{-9}$ m^2/N (compressibility of the rock)
$\beta = 4.74 \times 10^{-10}$ m^2/N (compressibility of water)

After the application of Equation (5.2), a proposed value for S_s is 5.6 x 10^{-5}.

<u>Estimating the thickness of the fractured zone (D) (De Lange, 1999)</u>

To estimate S, the thickness of the fracture zone is required and the following equation (obtained from experience from the tracer tests) can be used:

Thickness D(m) of fracture zone = (0.2*Q/s)*0.14 (5.3)

Where s = drawdown (m) in borehole after 1 minute of abstraction at a rate of Q (L/s) from the borehole.

After the specific storage and thickness of the fracture zone have been estimated, the S-value for the fracture can be estimated by the following equation:

$$S = S_s \times D \qquad (5.4)$$

with S_s = specific storage; D = thickness of fracture zone.

Estimating the fracture half-length

By applying the Equations (5.5) –(5.9), the half-length of the fracture (x_f in the case of vertical fractures) or the radius of the fracture extent (r_f for horizontal fractures) can be estimated.

a. Vertical fracture (Gringarten and Ramey, 1974)

$$x_f = \frac{Q\sqrt{t}}{2s_w\sqrt{\pi T_f S_f}} \tag{5.5}$$

where x_f = half-length (m); Q=abstraction rate in m^3/d; s_w = drawdown in m after time t (minutes); T_f=T-value of fracture; S_f=S-value of fracture].

b. Horizontal fracture: early storage (Gringarten and Ramey, 1974)

$$r_f = \sqrt{\frac{Qt}{\pi s_w S_f}} \tag{5.6}$$

c. Vertical or horizontal fracture (proposed equation - assumed bilinear flow at early times in fracture)

$$r_f = \frac{Q(t)^{0.25}}{4s_w(T_f S_f)^{0.25}} \tag{5.7}$$

d. Vertical or horizontal fracture (Proposed equation - assumed a combination of linear/bi-linear flow in fracture at early time).

$$r_f = \frac{Q(t)^{0.333}}{4s_w(T_f S_f)^{0.333}} \tag{5.8}$$

e. Adapted Bohmer equation (bilinear flow in vertical dyke/fault)

$$x_f = \frac{Q(t)^{0.25}}{2.74s_w(T_f S_f)^{0.25}} \tag{5.9}$$

5.2.3 Protection Zone III

If persistent hazardous non-degradable elements are present, the whole catchment area of the borehole must be protected. Because of the use of the word “hazardous”, some consideration must be given to its implications. Therefore the importance of risk assessment must be considered.

The estimation of the three protection zones was coded in the FC-program and is called BPZONE.

5.3 PROGRAM *BPZONE*

Protection Zone I

If more than 20 persons are using pit-latrines, it is proposed that a fence of at least 5 m must be constructed around the borehole. Otherwise, just a sanitary seal is required.

Protection Zone II

The maximum expected NO_3 (as N) is estimated by using the following information:

1. Concentration of urine = 13000 mg/l (can be changed)

2. Percentage N leaching = 50% (can be changed).

3. Volume urine/person/day = 1 L (can be changed).

4. Average annual recharge in mm.

5. Abstraction rate of the borehole (i.e. sustainable yield) in L/s.

6. Number of persons using on-site sanitation in the catchment area of the borehole (the catchment area is estimated from the abstraction rate and recharge).

Program BPZONE then estimates the maximum expected N in boreholes and the risk to infants according to Table 5.1

Table 5.1 Nitrate value intervals and the risk (to infants) associated with each interval.

Parameter	no risk	low risk	high risk	very high risk
N (mg/l)	<5	5-10	10-20	>20

The next step is to estimate the extent of the vertical or horizontal fracture by using early pumping test data (i.e. the drawdown after 1 minute). To estimate the extent, the program uses the five equations presented earlier (Eq. (5.5) – (5.9)). The mean of the estimated fracture extent together with the standard deviation is given.

Step 3 is the estimation of the travel time from the surface to the water level and the position of the water strike. For the estimation of travel times, a number of parameters is required, i.e.:

i) The saturated vertical K-value of the unconsolidated material. If the K-value is not known, it was decided to try and estimate it by means of the clay content of the unconsolidated material. Data were gathered of soils with known values for clay content and K-values. A graph was drawn of clay content vs. K-value and the equation of the exponential trend line fitted with the data was incorporated in the program BPZONE.

$$K_u = 30\, e^{-0.3*cl} \tag{5.10}$$

where cl = clay content (%).

ii) Vertical K-value of the consolidated material. If unknown, it is suggested that a value of 0.01 be used (usually between 0.001 and 0.1 for South African rocks).

iii) Distance from surface to water level.

iv) Thickness of the unconsolidated and consolidated material and their kinematic porosities.

The total travel time to the water strike is estimated from (assuming a gradient of 1):

$$t = \frac{n_{eu} D_u}{K_u} + \frac{n_{ec} D_c}{K_c} \tag{5.11}$$

where t = travel time (d); n = kinematic porosity; D = thickness (mm) and K = hydraulic conductivity (m/d) . Subscripts u and c denote unsaturated and saturated zone repectively.

Risk of microbial pollution is assigned by using the following criteria:

Table 5.2 Travel times and the risk of microbial pollution assigned to each

Parameter	no risk	low risk	high risk	very high risk
Travel time (days)	>100	30-100	3-30	<3

The program then estimates protection zone II by using the following criteria:

Table 5.3 Criteria involving risk and proposed distances for protection zone II.

Risk	Protection zone II
High or very high risk of microbial pollution.	2 times fracture half-length.
Low risk for microbial pollution.	1 times fracture half-length.
No risk for microbial pollution.	0.5 times fracture half-length.
High risk for N for infants.	2 times fracture half-length.
Low risk for N for infants.	1 times fracture half-length.
No risk for N for infants.	0.5 times fracture length.

Protection Zone III

If hazardous chemical elements are present, the program estimates the catchment area of the borehole by using the recharge and the abstraction rate (Eq. (5.12)).

AREA = Q / Recharge (5.12)

5.4 JUSTIFICATION WITH *MODFLOW* GENERATED EXAMPLES

The well-known *MODFLOW 3D* finite difference program (*PMWIN*, Chiang and Kinzelbach, 2000) was used to generate three typical case studies in fractured-rock aquifers. The model constructed has 10 layers with a thickness of 1 m each, except for the bottom layer (i.e. the fracture zone) which was assigned a thickness of 0,2 m (i.e. the typical situation on the Campus Aquifer). The following parameters were assigned to the different layers (Table 5.4):

Table 5.4 Parameter values assigned for the generated Modflow model(S_s = specific storativity [1/m]; K_h = horizontal K-value; K_v = vertical K-value)

Layer	S_s	K_h(m/d)	K_v(m/d)
1-9	5e-5	0.1	0.005
10	5e-5	3600	.005

This gives a T-value of the matrix = 1 m^2/d and T-value of the fracture = 720 m^2/d as on the Campus of the University of the Free State Test Site.

A fracture zone was assigned to the bottom layer (i.e. layer 10) and three fracture radii were used, i.e. 60 m, 100 m, and 200 m (i.e. the case of a horizontal bedding plane fracture). The model was run for 300 minutes for each of the set-ups and the drawdown after 1 minute (with an abstraction rate of 1.25 l/s) was supplied to the *BPZONE* program. The results are shown in Table 5.5.

Table 5.5 Comparison between *MODFLOW* and *BPZONE* results (mean and standard deviation of the 5 equations) for the estimation of the radius of the fracture.

PROGRAM	r_f (m)	r_f (m)	r_f (m)
MODFLOW (value assigned)	60	100	200
BPZONE (mean of 5 equations)	76 ± 15	113 ± 19	175 ± 40

5.5 DISCUSSION

1. Due to the heterogeneity of South African conditions concerning the groundwater environment, this document can be seen as a continuation of the work that has already been done on delineating protection zones. The information in this chapter must be regarded as such and not as the ultimate solution to assigning protection zones. There is room for many more approaches as well as studies on this topic.
2. In this chapter the focus is on nitrates and microbes due to their association with pitlatrines and septic tanks. The same approach can be applied to more parameters depending on what type of study is being done.
3. The application of risk assessment and management is of great importance. However, great care must be taken and the person(s) responsible should be adept in the process and be able to evaluate a given situation with care before any major decisions are reached.
4. The program BPZONE must be used with great care and the answers obtained must be viewed within the existing conditions of a given situation or problem.

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

6.1 CONCLUSIONS

From the research, it can be concluded that:

1) The real reason why a particular pumping test is to be conducted must be established before the start of the test. The purpose of the pumping test will play an integral part in the actual method of conducting and analysing the test (length and abstraction rate). In South Africa, pumping tests are performed for mainly two reasons; to determine the long-term sustainable yield of a borehole or to estimate the aquifer parameters.

2) Because of the composition of the South African aquifer systems (water table aquifer behaviour and elasticity), there is a non-linear relationship between the abstraction rate and the drawdown. At different abstraction rates, different fractures will be dewatered and, in some cases, the end of the fracture extent will be reached. This results in a smaller effective transmissivity (T) value, giving rise to a unique drawdown value per abstraction rate. This non-linearity between drawdown and abstraction rate has the consequence that it is very difficult to extrapolate the behaviour of the water level in an aquifer, if a rate different to that used during the constant rate test is applied. This feature should be treated with caution when it comes to the assigning of sustainable yields to boreholes.

3) Incorrect analytical methods are often used to estimate parameter values. Because of assumptions such as homogeneity and infinite areal extent used in developing most of these analytical methods as well as the unique composition and characteristics of South African Karoo aquifers, unrealistic parameter values are produced. These parameter values (such as storativity that is distance dependent) are then used in sustainable yield calculations. These incorrect sustainable yield values result in the borehole drying up.

4) Pumping tests should be performed as accurately as possible. An aquifer (groundwater resource) that is to be utilised is not visible to the geohydrologist (can not be seen with the eyes like surface water) and because of this, many unknowns and uncertainties exist. This can result in incorrect assumptions and interpretation of data, causing serious problems. In an effort to minimise the uncertainties, it is important that the visible or known part of the investigation should be performed properly. It is therefore important that the pumping test and the collection of data during the test should be performed properly.

6.2 RECOMMENDATIONS

1) The purpose or objective of a pumping test should be established before a test is conducted. If the objective of a pumping test is to estimate the parameters of the aquifer, the following is recommended:
   - A modified minimum one-hour step drawdown test should be performed to locate the depth of the main water strike as well as to determine an abstraction rate for the constant rate pumping test.
   - A constant rate pumping test, long enough to ensure interpretable drawdown curves at the observation points, should be performed. The abstraction rate during this constant rate test is very important because no water-yielding structures should be dewatered during the test.
   - The recommended minimum duration of the constant rate pumping test is 8 hours, but depending on the parameters that is to be estimated, it can last for several days.

2) Great care should be taken when analysing pumping test data for parameter estimation. A suitable analytical model should be used, taking into account the unique aquifer conditions in South Africa. In some cases, a two-dimensional numerical model such as RPTSOLV, developed for the conditions in this country, should be used to estimate aquifer parameter values. By applying incorrect analytical methods, wrong storativity values (S-values) are obtained. When using numerical models such as RPTSOLV, acceptable storativity values are obtained. For more accurate data fits, a three-dimensional numerical model must be used.

3) If the objective of the pumping test is to estimate the long-term sustainable yield of the borehole, the following is recommended:
   - Once again a modified minimum one-hour step drawdown test should be performed to locate the depth of the main water strike as well as to determine an abstraction rate for the constant rate pumping test.
   - The constant rate pumping test should be performed, straining the aquifer in order to drop the water level down to the position of the main water strike within 8 hours.
   - It is recommended that the FC-program be used for the long-term sustainable yield calculation.

4) It is recommended that the non-linearity between drawdown and abstraction rate, causing problems in extrapolating the future behaviour of the water level in an aquifer if a rate different to that used during the constant rate test is applied, be taken into account when assigning sustainable yields to boreholes. If a non-linear well loss coefficient has to be established, it is recommended that the revised minimum one-hour step drawdown test be performed.

5) The proper planning and execution of a pumping test will yield reliable data and this data will enable the user to estimate realistic parameter values or long-term sustainable yields. It is therefore recommended that great care should be taken when doing a pumping test and when gathering data during a pumping test.

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MANUAL ON PUMPING TEST ANALYSIS IN FRACTURED-ROCK AQUIFERS

APPENDICES

APPENDIX A

FIELD GUIDE FOR CONDUCTING PUMPING TESTS IN FRACTURED-ROCK AQUIFERS

APPENDIX B

CHECK-LIST FOR PUMP TEST EQUIPMENT

APPENDIX C

CHECK-LIST FOR OTHER EQUIPMENT

APPENDIX D

PUMP TEST DATA SHEETS

APPENDIX A

FIELD GUIDE FOR CONDUCTING PUMPING TESTS IN FRACTURED-ROCK AQUIFERS

Note: Many of the ideas in this Appendix are adapted from Hobbs and Marais (1997).

1. INTRODUCTION

Drilling and developing a borehole is an expensive exercise and in all cases the performance of such a borehole will be of utmost importance to somebody. This can be a farmer using the water for irrigation or even a whole community depending on the borehole for their water supply.

The policy of the present government is to address the basic needs of all the people in South Africa and in doing this, they have in the past few years supplied basic water needs to between 12 and 18 million people in approximately 15 000 villages in the rural and remote areas of the country. In doing this, groundwater with its widespread, mostly low-yielding occurrence was widely used and because of this, it became a national asset of strategic importance. Because of this, legislation was passed to change the status of ownership of water in South Africa, including that of groundwater.

It is estimated that some 90 percent of local groundwater occur in secondary aquifers consisting primary of shallow zones of weathering and fracturing. A lack of understanding of the occurrence, movement and recharge of groundwater led to this resource not being utilised in a sustainable manner. The consequent failure of boreholes, in some instances, has unfortunately promoted the belief that groundwater is an unreliable source of water supply and that it should be replaced by the more reliable surface water supply. Because of this, it is going to be a very difficult and uphill task to re-establish groundwater as a reliable water source and to give it its rightful place as a source of reliable water supply.

Contrary to surface water, groundwater is not visible and this makes understanding the art of groundwater resource development and determination that more difficult. The depletion of the country’s surface water resources is a matter of great concern and in the not too distant future alternative water resources will have to be found. The obvious alternative is to develop the groundwater resources, which will put tremendous strain on this resource. Proper control over the development, as well as the management of the groundwater resources, will thus be very important in future. The new water law in the country states that all water belongs to the government and abstraction can only take place after a permit had been issued. This lays the foundation for the control over the development and management of the groundwater resources.

Due to the above, pump testing of boreholes will, in future, become even more important. In performing a pumping test, we can determine the possible long-term sustainable yield of a specific borehole as well as the strain that the abstraction from the borehole will place on the aquifer. Performing accurate pumping tests can therefore not be over-emphasised.

2. INITIAL INFORMATION

Before the actual pumping test is done there are several initial actions that should take place. If these steps are not performed properly there might be uncertainties that eventually might result in claims being put in against the different parties involved. The pumping test contractor may even run the risk of not receiving his or her money for work done.

2.1 CONTRACTUAL MATTERS

Whether a pumping test is to be done for a private person, the government or any local authority, it is of utmost importance that the person doing the test must enter into a proper contract with the other party. Normally, a tender is being put out by the representative of the party or the party itself that wants the pumping test done and the pumping test contractor will tender to do the work for a certain price. In a tender, the scope of the work to be done must be described in detail. In some cases, the pumping test contractor will be asked to give a quotation to perform a pumping test and in this quotation, the work that will be done must be stated in detail.

In both the above instances, the pumping test can only be performed after a letter of appointment had been issued. A verbal agreement is very dangerous and it might lead to serious misunderstandings that might even result in differences being settled in court.

2.2 LOCATION OF BOREHOLE

If the borehole to be tested and other observation boreholes are numbered, these numbers should be obtained from the person or authority that requires the pumping test. These numbers are normally listed in the tender and they should also be listed in the quotation handed in by the pumping test contractor.

When putting out a tender for a pumping test, it is good practice to include a map of the location and position of the borehole to be tested as well as possible observation boreholes. This will eliminate any possible misunderstandings that might occur.

The co-ordinates of the borehole that are to be tested, as well as possible observation wells should be obtained in writing. A Global Positioning System (GPS) can then be used to locate the borehole where the pumping test will be performed as well as the positions of observation boreholes.

A good practice is also to request the party that requires the pumping test or his representative to physically go and point out the exact location of the borehole to be tested and possible observation boreholes. This takes the form of a site meeting and during such a site meeting all uncertainties can be cleared up. The positions can then be verified and mutually marked on 1 : 50 000 maps. With this strategy, any uncertainties as far as the positions of the boreholes can be cleared.

2.3 BOREHOLE FITTED WITH PUMP

It is very important to establish whether the borehole that is to be pump tested is fitted with a pump. If this is the case, it must be determined beforehand who would be responsible for the removal as well as the re-installation of the equipment.

The detail about the responsibility for the removal of equipment from the borehole to be tested should be included in the tender or in the quotation handed in. Removal of existing equipment can be a timeous exercise and proper provision should be made for it in the quotation that is handed in to do the pumping test. In some instances, the borehole to be tested is situated inside a building or enclosure and removal of the equipment can only be carried out by removing the roof of the building, making it a very difficult exercise. In most cases, a tripod type of frame fitted with a block and tackle or a winch will be the most suitable to remove the equipment.

Care should be taken when removing the equipment not to damage it and after removal, information such as serial numbers on the pump, make and model of the pump and any defects present should be written down. The depth at which the pump was situated should also be written down.

After the completion of the pumping test, the equipment should be re-installed and the party responsible for doing it should be identified beforehand.

2.4 BOREHOLE LOGS

When drilling a new borehole, the drilling contractor will supply samples of the rock formation being drilled out of the new borehole. This is done by inspecting the rock chips or drill cuttings brought to the surface during drilling. These samples are taken at one-meter intervals and are placed on the ground in the order that they are taken from the newly drilled borehole. The picture below **(*Fig. A1*)** shows these samples placed on the ground at a newly drilled borehole.

Fig.A1 Samples taken from a newly drilled borehole

The samples are then described by a qualified person according to the prescribed guidelines. The drilling contractor will give a description of the colour of the formation, the relative size of the drill cuttings as well as the possible rock types. This is called a written log of the borehole and below is an example of such a log ***(Table A1)***.

Table A1 Written borehole log

DEPTH	DESCRIPTION
0.00 – 7.00 m	Coarse reddish soil (loose, soft)
7.00 – 12.00 m	Coarse and fine reddish-brown soil (loose, soft)
12.00 – 24.00 m	Fine yellow brown sand (loose)
24.00 – 28.00 m	White calcrete (medium)
28.00 – 35.00 m	Bluish green shale (fine, solid)

This information is combined in a schematic layout and it is known as the log of the borehole. A typical log of a borehole is shown below (***Fig. A2***).

For large areas of South Africa, detailed geological maps had been produced and these maps can assist in understanding the local geology of a specific area. These maps can also be used to compare the borehole logs to the geological map of the area.

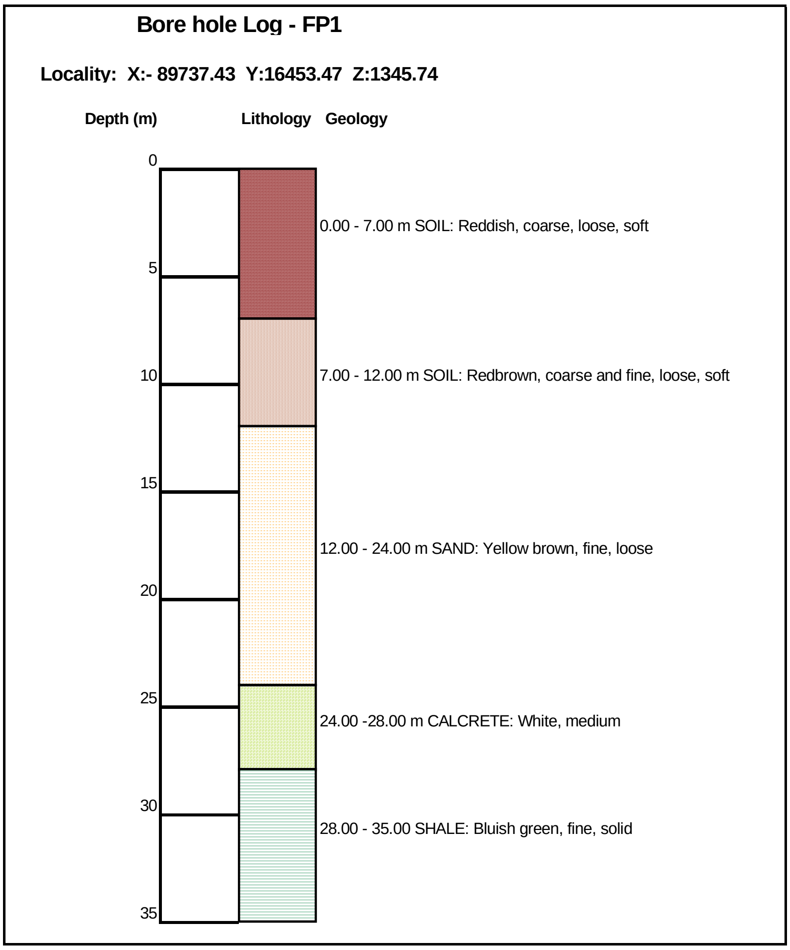

Fig. A2 A typical borehole log

The borehole logs supply important information such as possible fracture positions and water-bearing formations. It is therefore very important to gather as much as possible information about the geology of the area. If no geological map or borehole log exists, the geology of the area around the borehole will have to be interpreted. Interpreting the geology of the area, however, requires extensive knowledge and experience.

2.5 EXISTING PUMPING TESTS

It is important to gather as much as possible information about the borehole being tested. In many instances pumping tests had already been done at these boreholes and these tests can yield very important information.

The science of Geohydrology is continuously being developed and therefore the existing pumping tests can be re-evaluated with the aid of the latest technology and methods available. A very good example of such new technology that became available only recently is the Flow Characteristic Method (FC-method) developed by Van Tonder et al., 1999.

Information such as the rest water level compared with the present water level can give an indication of whether the groundwater level has fallen or risen. An example of a major drop in the rest water level is in the Molopo area where the water, up till a few years ago, formed a natural lake above the ground. Agricultural development in the area led to an increase in the abstraction of the groundwater. This caused the water level in the aquifer to drop to a level of 23 m below the surface.

The yield of the borehole can be compared against the existing pumping tests and this can give an indication of whether the groundwater resource had been over exposed.

2.6 AVAILABILITY OF POSSIBLE OBSERVATION BOREHOLES

It is also good practice to obtain information about possible observation boreholes in the vicinity of the borehole to be pump tested. Normally, the representative of the party that requires the pumping test will supply this information. Details about observation boreholes should be included in the contract.

If no observation boreholes are specified in the contract, it is very important that the person doing the pumping test should try and identify possible observation boreholes close to the borehole that is to be tested. People staying in the area will know about boreholes that exist in the vicinity.

The importance of an observation borehole can not be over-emphasized. With the aid of observation boreholes the parameters obtained for a borehole can be verified. Parameters such as storativity, which is distance dependent if analysed incorrectly, can also be tested and verified.

2.7 MAPS AND AERIAL PHOTOGRAPHS OF AREA

It is very important to get hold of maps of the area in which the pumping test is to be performed. Information such as access roads, height above mean sea level, contour information, possible observation boreholes, as well as property boundaries, could be identified on these maps. Possible aquifer boundaries could also be sighted from these maps.

Aerial photographs could also yield valuable information such as development of the land and growth of trees in the area. Rocky outcrops on aerial photographs could indicate possible dykes. Excessive tree growth in a definite line, away from a river, might indicate a water strike, and this will only be visible on aerial photographs. The increase in such tree growths can be determined from these photographs.

2.8 EQUIPMENT REQUIRED FOR A PUMPING TEST

Normally, boreholes that are to be pump tested are situated in remote areas away from towns or businesses. This means that is very difficult to go out and buy any spares or equipment that was left at home. To ensure that all the equipment is taken along, it is good practice to draw up a list of equipment that is used during a pumping test. An example of such a list is included in ***Appendix B***. The equipment required during a pumping test will now be described in detail below.

2.9 POWER SUPPLY TO THE PUMP

In some cases, especially new boreholes, the borehole to be tested is not fitted with electrical power and this means that the test contractor will have to supply his own power. A power source, normally a generator, powerful enough to supply power to the pump for a long enough period of time, must be used. If a generator is to be used as a power source, enough fuel should be taken along to keep the generator going for the duration of the pumping test.

At some of the boreholes, electrical power will be available, but this can either be single phase (220 volts) or three phases (380 volts). Not knowing what kind of electricity it is that is available can cause serious damage to equipment. If electricity is to be used, a long enough lead cable should be taken along to supply the electricity from the power source to the pump.

The type of power that is to be used is not that important. However, the power source should be reliable and it should be able to supply sufficient power to the pump for the duration of the pumping test. A pumping test normally runs through the night and therefore it will be a bonus if the power source could also supply power to flood lights that are to be used.

2.10 PUMP SELECTION TO DO THE TEST

There are mainly two types of pumps used to perform pumping tests. They are mono and submersible pumps, available in different shapes and sizes on the market. Because of this, pump selection is a study topic on its own. It is, however, important to note that the pump that is to be used for the pumping test should be capable of operating continuously at a **constant discharge** for a period of at least 72 hours.

According to the South African Bureau of Standards Code of Practice on the Development, Maintenance and Management of Groundwater Resources (SABS 0299-4:1998 Part 4: Test-pumping of water boreholes) the quality of the pump should be such that the variation in discharge must be less than 5 % for a constant rate pumping test. If the variation exceeds this limit, the pumping test should be stopped and after recovery, the test should be restarted, using suitable equipment.

Normally, a positive displacement type pump (mono pump) is used when performing pumping tests. This type of pump can be divided into two components: the actual pump situated inside the borehole and the power supply situated outside the borehole. With a mono pump, the discharge rate can be changed by varying the speed or revolutions of the power supply. This can be done with the aid of a gearbox or by regulating the fuel throttle. No valve can be used to increase or decrease the pumping rate of a mono pump. The main advantages of a mono pump are the constant rate at which it can pump or discharge water for a long period of time, as well as the large volumes of water that it can pump.

It may be acceptable, under certain circumstances, to use a submersible pump (negative displacement pump) for testing purposes. In the case of a submersible pump, both the pump and power supply are situated under the water inside the borehole. When a submersible pump is to be used, it is very important that the unit be fitted with a non-return valve at the bottom of the pump column. This prevents any return flows after the pump was shut down and the recovery period has started. A submersible pump cannot deliver very large volumes of water, compared to mono pumps. The discharge from a submersible pump is increased and decreased by using a valve in the delivery piping. It is difficult to maintain a constant discharge rate over a long period of time with a submersible pump.

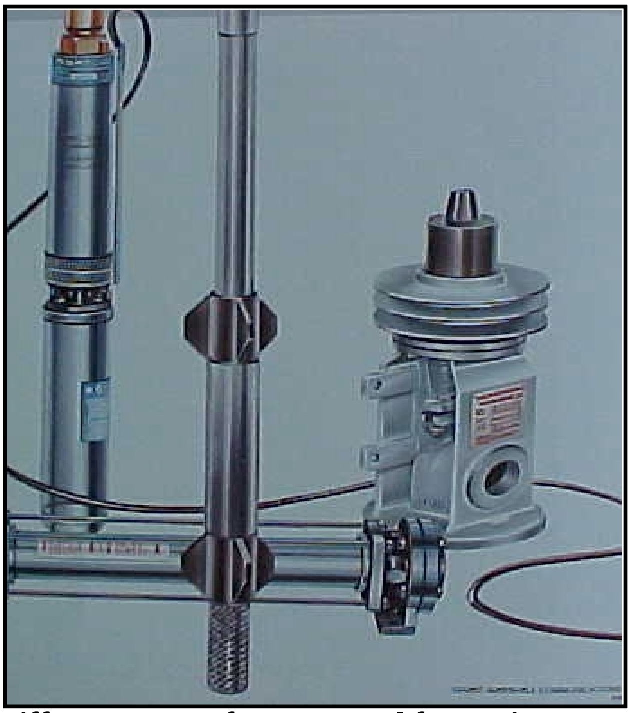

Fig. A3 Different types of pumps used for testing

In the figure above ***(Fig. A3)*** on the left, a submersible pump is showed. In the center is a mono pump with its discharge head on the right.

The pump used for a pumping test must be capable of delivering water at a rate in excess of the expected maximum yield of the borehole to be tested. The capacity of the pump and the rate of discharge should be high enough to produce good measurable drawdowns in observation boreholes as far away as 200 m from the pumping borehole, depending on the aquifer conditions. In many cases, some of the pump testing contractors carry more than one pump, each capable of pumping a different rate, depending on the requirements.

2.11 EQUIPMENT TO REMOVE EXISTING PUMPING EQUIPMENT

The responsibility for the removal of existing pumping equipment should be sorted out in the scope of the work to be done, in the contract or in the tender. This is very important because in many cases, this can be a very difficult and time-consuming task. Provision for the removal and the re-installation of existing equipment should be made in the tender, taking into account the risks attached to this exercise. Some of the boreholes that are to be tested are inside little enclosures, making the removal of equipment a very difficult task.

In some cases, it is almost impossible to get equipment out of existing boreholes because of tree roots growing into the boreholes or borehole sides falling in. Sometimes bees build their hives inside boreholes and this also clog up the upper part of the borehole. Disconnecting rusted delivery pipes as the equipment is removed from the borehole is no easy task, sometimes even completely impossible.

A possible method of removing existing equipment is to make use of a tripod type of frame ***(Fig. A4)***, and a block and tackle or winch. The same frame and hoist gear can be used to install and remove pump testing equipment in the borehole. In some extreme cases, existing equipment can be pulled from the borehole with the aid of a vehicle, but this is not recommended.

Fig. A4 Tripod to remove equipment

2.12 EQUIPMENT TO DETERMINE DEPTH OF BOREHOLE

It is important to determine the depth of the borehole in order to determine the depth at which to install the pump used in testing the borehole. When performing a pumping test, the pump should be placed as deep as possible inside the borehole, but without any interference of silt or debris lying at the bottom of the borehole.

A normal 50 or 100 m measuring tape can be used to determine the depth of the borehole. A weight should be attached to the tape and then it can be lowered into the borehole. When the tape starts picking up slack, the weight has reached the bottom of the borehole and a reading on the tape will indicate the depth of the borehole.

Another method of determining the depth of the borehole is to drop a bailer down the borehole and marking the cable when it starts picking up slack. The distance from the bottom of the bailer to the mark on the cable can be measured and this will indicate the depth of the borehole. This method is preferred because it will clearly indicate any obstruction in the borehole and while the bailer is inside the borehole, any silt or loose material can be removed. This will limit any interference from the loose material during the pumping test.

2.13 SLUG TEST EQUIPMENT

A slug test is a quick and easy method that can be used to predict the yield of the borehole by measuring the rate of recovery of the water level after a sudden change. This test is performed by suddenly raising or lowering the static water level in the borehole with the aid of a closed cylinder ***(Fig. A5)***. The cylinder replaces its own volume of water in the borehole, thus increasing the pressure in the borehole. The equilibrium in the water level is changed and it will recover or stabilise to its initial water level. By measuring the rate of recovery or recession (time taken to recover) of the water level, the borehole’s transmissivity or hydraulic conductivity can be measured.

Fig. A5 Slug with known volume to do slug test

To perform this test we need a closed cylinder with a known volume, tied to a length of rope. Instrumentation to measure the rate of recession of the water level inside the borehole is also required. For this purpose a data logger can be used. The borehole diameter must be 165 mm in order to use this method. The recession time to recover to at least 90 % of its initial value is used in a formula to determine the yield of the borehole. The formula (Vivier *et al.*, 1995)

$$\mathbf{y = 117155\ x^{-0.824}} \qquad \textbf{(A1)}$$

where x = recession time in seconds

will give the possible yield of the borehole in L/h.

The graph below ***(Fig. A6)*** was drawn up from results obtained by testing 32 boreholes. The graph shows that a straight line is obtained, using log-log scale. If the recession time for the borehole is entered on the x-axis, the possible yield can be read off on the y-axis.

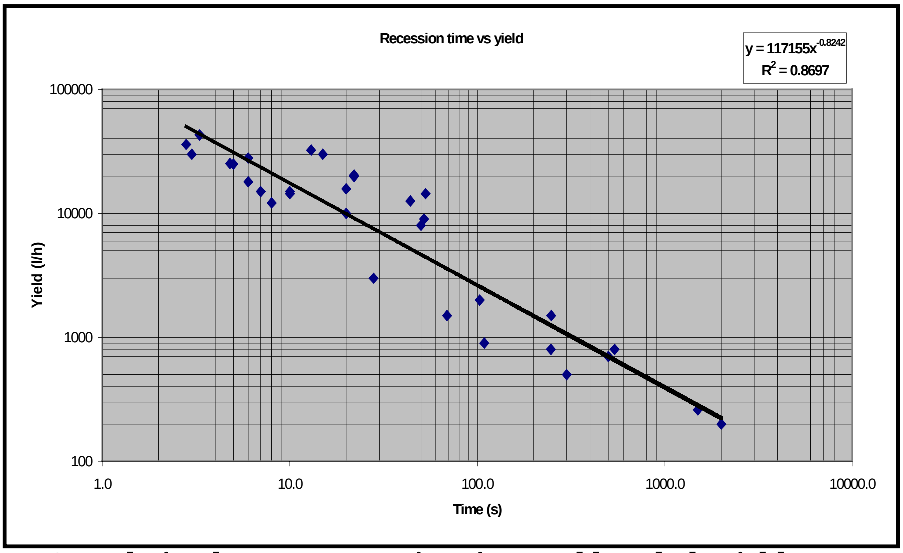

Fig. A6 Correlation between recession time and borehole yields

If a slug test indicates that the potential yield of a borehole will be less than 0.3 L/s, then performing any additional tests should be reconsidered. If the potential yield is more than 0.3 L/s, then the contractor should proceed with other tests such as step drawdown, multirate and constant rate pumping tests.

2.14 WATER-LEVEL MEASURING EQUIPMENT

When performing a pumping test, the static water level inside the borehole is lowered and this change in water level is measured against time. This information is the only insight into the behaviour of the borehole as well as the aquifer and it is therefore very important to measure the water levels and time intervals as accurately as possible. Gathering correct and accurate information during the pumping test is of utmost importance. These measurements can be done by hand or with an electronic data logger.

2.15 HAND READINGS

Hand readings are taken with a dip-meter ***(Fig. A7 - right)***, a tape measure or, in some case, two electric wires with a break in the circuit at the zero point of the device ***(Fig. A7 - left)***. As the zero point reaches the water, the water closes the circuit and a light flashes or a buzzer sounds. The distance from the collar of the casing of the borehole to the water level is measured and recorded. The predetermined time intervals are measured with the aid of a stopwatch and also recorded with the depth of the water level. This information is recorded on data sheets drawn up specifically for this purpose. The data sheets will be discussed later in this report.

Sometimes the turbulence caused by the pump, as well as return flows into the pumping borehole can make water-level measurement with a dip-meter very difficult or even impossible. To overcome this problem, a plastic conduit tube, normally 16 mm diameter, is introduced down the pumping borehole. This conduit is attached to the riser main of the pump at 2 to 3 m intervals. Water-level measurements are then taken inside this conduit tube.

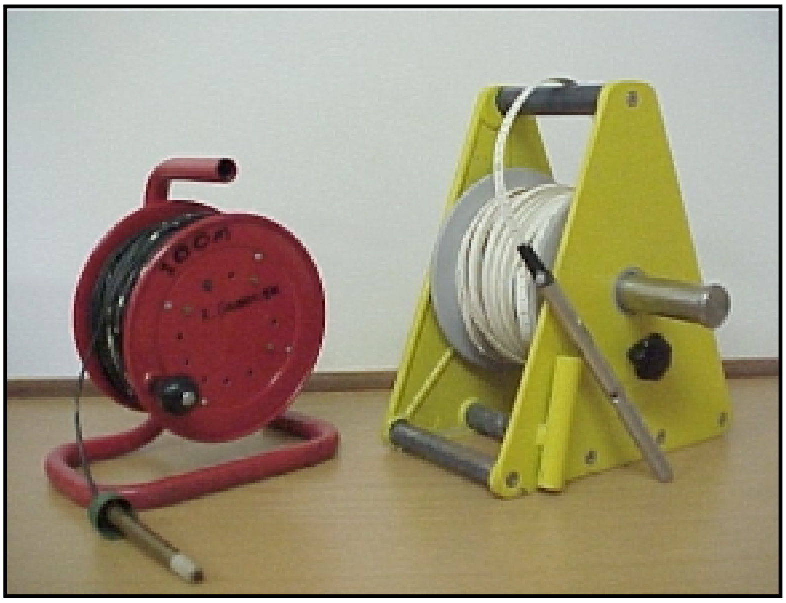

Fig. A7 Two types of dip-meters used for water-level measurement

With the method of hand water-level measurement, the ever-present human error can always creep in and cause valuable information to be recorded inaccurately. To overcome this problem, electronic data loggers are used. These data loggers are reliable and they can be set up to take readings at specified intervals.

2.16 ELECTRONIC DATA LOGGERS

There are many types and makes of data loggers available on the market and therefore the specifications of all the different components of the logger should be looked at very carefully. A data logger that satisfies the needs of the pump test contractor as well as the standards set by the industry should be used. The data logger can be divided into three main components namely, power supply, the pressure probe and the logger itself.

2.16.1 POWER SUPPLY TO THE DATA LOGGER

The power consumption of data loggers is normally very low and usually a 12-volt battery will be able to supply sufficient power to the equipment for the duration of the pumping test. In some cases, especially during longer pumping tests and where the frequency of data logging are set at small intervals, solar panels ***(Fig. A9)*** are used to charge the battery that supplies power to the equipment. When electrical power is available, it can be used in conjunction with a transformer, bringing the power down to 12 volt. Normally, a power regulator is used to ensure that a good quality of power is supplied to the equipment.

2.16.2 PRESSURE PROBES

The pressure probe ***(Fig. A8)*** used for water-level measurement has got diameters that vary between 12 and 42 mm and it can therefore be installed in most of the boreholes and piezometers. The pressure probe makes use of a ceramic reference pressure-measuring cell that senses the hydrostatic pressure of the water column via a capacitive pressure diaphragm and this value is converted into an electric signal.

The power required to operate the pressure probe is only 12 volt and the output from the probe can either be 1 - 5 volt or 4 - 20 mA. The probe ranges vary from 0 - 2.5 meters to 0 - 40 m, which is sufficient for most boreholes. The accuracy of the probes is better than 1 cm for 10 m of measurement, which is the acceptable standard in the industry.

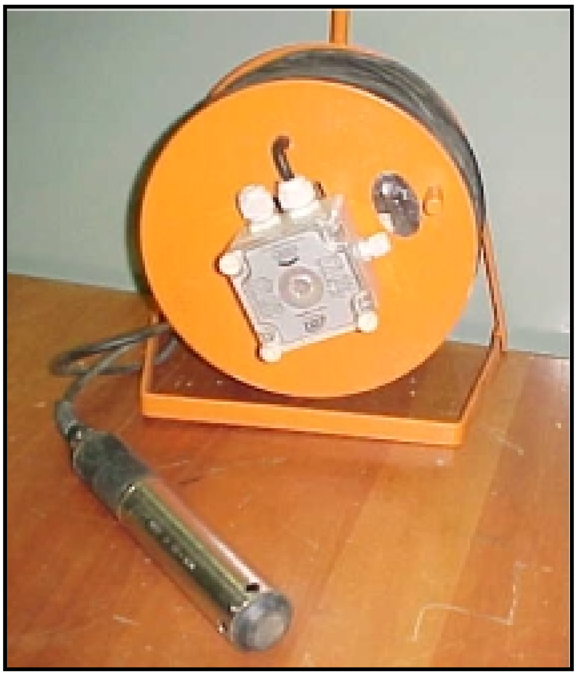

Fig. A8 Pressure probe and vented cable

2.16.3 DATA LOGGERS

A multi-channel data logger ***(Fig. A9)*** is used to convert the electronic signal from the pressure probe to a height. This is the height of the water column above the pressure probe inside the borehole or piezometer. This reading can be taken and stored at specified intervals, which can range from 5 sec. to once a day (24 h), depending on the need of the client.

The data logger being multi-channeled can accommodate up to four pressure probes. Only one data logger can therefor be used to take and store readings at the pumping borehole as well as three observation boreholes in the vicinity.

The data logger is powered by a 12-volt power system and like the pressure probe, a battery and solar panel can be used as the power source. This enables the pumping test operator to use the data logger in remote areas without any power problems.

There is also a function where the readings can be taken at certain intervals, but the average of a number of specified intervals will be stored. The logger can also be set up to take a reading only if the reading differs by a pre-set margin from the previous reading. A ring memory enables the data logger to store large amounts of data and these data are taken from the data logger with the aid of lap top computer and software.

Fig. A9 Data logger and power supply

There are various types of data loggers available on the market, and the type of data logger used does not really matter. It is, however, important that the data logger must be reliable because valuable data can be lost if the data logger fails during the pumping test.

A variation on the pressure probe and data logger combination is the data logger that makes use of an indirect measuring principle (bubble principle). A piston pump inside the instrument generates compressed air that flows through a dedicated line into a bubble chamber inside the borehole at programmable intervals. Depending on the groundwater level above the bubble chamber orifice, an air pressure equal to the hydrostatic pressure is established inside the measuring tube. Assuming a constant liquid density, there is a linear relationship between the water level to be measured and the air pressure inside the measuring tube. The bubble line pressure and the barometric pressure are measured concurrently by an absolute pressure measuring cell inside the data logger. The water level is calculated as the difference between the two signals.

2.17 INTERVALS OF WATER-LEVEL MEASUREMENTS

The water levels inside the pumping borehole as well as the observation boreholes must be measured many times during the pumping tests, as well as during the recovery stage of the test. Because the water level drops rapidly during the early times of the pumping test (first two hours), it is important that water-level readings should be taken at short intervals. As the pumping test progresses, the intervals at which readings are taken can be lengthened. The same principle applies for both hand readings taken with a dip-meter as well as readings taken with the aid of a data logger. When a data logger is used, the number of readings taken can be filtered afterwards. It is therefore good practice to take readings at short intervals for the duration of the pumping test and to filter the readings afterwards. By using this method, important events such as fracture positions and boundaries can be pinpointed and logged in detail. Other external pumping activities that might have an effect on the pumping test can also be picked up easily if this method of logging water levels is used. On the chart below ***(Fig. A10)***, it can clearly be seen that water was abstracted from another borehole (step 1) close to the pumping test borehole. The time at which the pumping activities were stopped (step 2) can also clearly be seen.

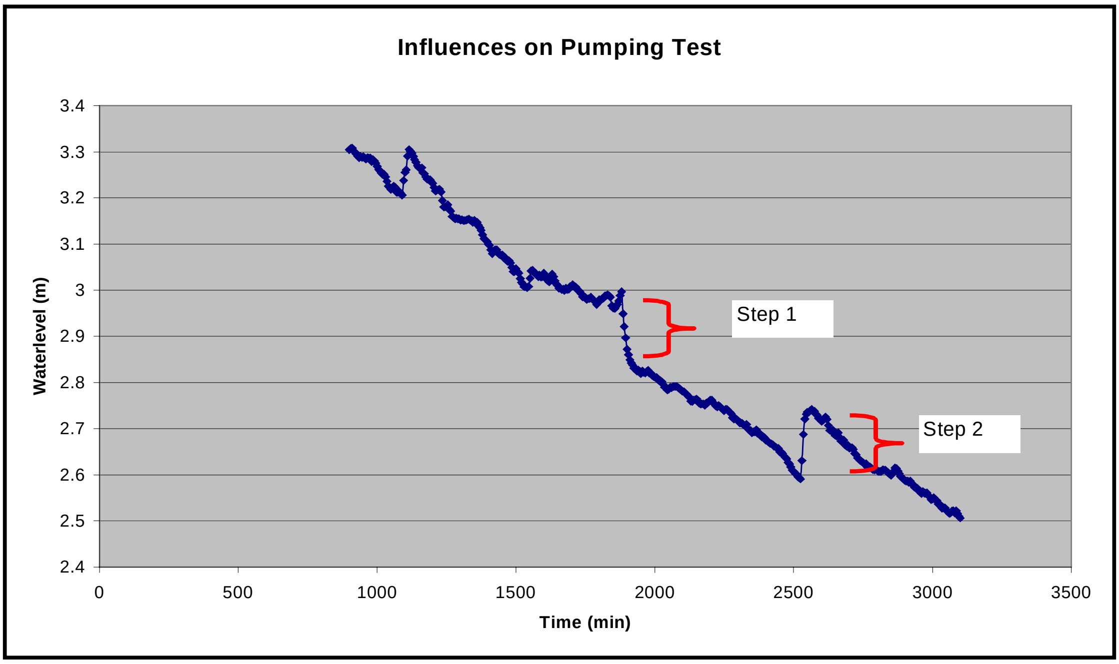

Fig. A10 The influence from external pumping activities on the drawdown

The intervals at which hand readings should be taken can be seen on the data sheet as described in the next section. It must also be noted that hand readings must be taken at all times, even if a pressure probe and data logger had been installed for the pumping test. The hand readings will verify the readings taken with the aid of the pressure probe and data logger.

2.18 DATA LOGGING SHEETS

Everything that happens during a pumping test should be recorded or placed on record. It is very important that all the information should be kept together. Information recorded on various or on small pieces of paper can easily be lost and this might lead to the pumping test being a failure. Lots of money is invested in a pumping test and if it is a failure, all this money will be wasted.

It is therefore a good practice to draw up a form or sheet on which all the relevant information can be recorded. The format of such a data logging sheet is entirely up to the individual, but it should contain all the relevant information after the pumping test was completed. This sheet should be drawn up beforehand and it should be able to accommodate information such as:

- Name of pumping test borehole
- Number of borehole
- Date of pumping test
- X co-ordinate
- Y co-ordinate
- Z co-ordinate
- Pumping rate
- Rest or static water levels
- Intervals at which readings should be taken
- Weather conditions
- Description of possible boundaries
- External factors that might have an influence on the pumping test

In some cases, a universal form or sheet is drawn up and this sheet can therefore be used to record information on slug tests, calibration tests, constant rate pumping tests, multirate pumping tests, as well as recovery tests. An example of such a data sheet is included in ***Appendix D .***

2.19 DISCHARGE OR DELIVERY PIPES TO RELAY WATER FROM THE PUMPING BOREHOLE

As already mentioned, the purpose of a pumping test is to remove water from the pumping borehole as well as from the aquifer. Care should therefore be taken that the water removed from the borehole does not end up back in the aquifer before the pumping test is finished. To ensure that this does not happen, the water must be taken away and discharged at a point far away from the pumping activities.

Discharge piping runs from the delivery side of the pump up the borehole and on the surface it takes water away from the borehole. Sometimes it is very difficult to put a continuous piece of delivery piping down the borehole and then the pipes are broken up in sections. The equal diameter sections of pipes are then added as the pump is lowered into the borehole.

The delivery pipe that runs along the ground normally consists of a large diameter continuos plastic pipe. This pipe should be at least 50 m long, but preferably 100 m. It must be free of leaks for the entire length of the pipe. Under certain circumstances, it may be required to discharge the water up to 300 m away from the borehole being tested.

According to the South African Bureau of Standards Code of Practice on the Development, Maintenance and Management of Groundwater Resources (SABS 0299-4:1998 Part 4: Test-pumping of water boreholes), the discharge point of the delivery pipe must be so far away that the discharged water does not flow back into the aquifer during the pumping test. It is also specified that in the case of

- a confined aquifer with a thick, impervious confining layer, the water must be discharged at least 10 meter away from the borehole,
- an alluvial gravel subterrain, at least 300 m, but preferably more than 500 meters away from the borehole and
- an aquifer of which the surrounding geological structure is not known, at least 1000 meters away from the borehole.

It is recognized that some water leakage will generally occur during a pumping test. This is acceptable provided that such leakage does not interfere with any water-level monitoring and the total amount of leakage to the end of the discharge pipeline does not exceed more than one percent of the pumping rate as measured at the end of the pumping test.

2.20 EQUIPMENT TO MEASURE DISCHARGE FROM BOREHOLE

For the duration of the pumping test, the discharge from the pump should be monitored and measured. Discharge measurement should take place at specified intervals to ensure the pumping rate is constant. There are various methods with which to determine the discharge rate from the borehole (Hobbs and Marais, 1997).

2.20.1 VOLUMETRIC METHOD

This is also called the container and stopwatch method. This is a very simple, but effective method to determine the discharge rate from the pump. The time it takes to fill a container of known volume is recorded and then the discharge rate can be determined. The container should stand level when it is filled and the stopwatch should be able to measure to an accuracy of one-tenth of a second. This method is fairly accurate and it is commonly used. The table below ***(Table A2)*** gives some indication of the size of the container to be used with the different discharge rates from the pump.

Table 2 Discharge rate versus container size for volumetric measurements

Discharge rate	Container size
Less than 2 L/s	20 L
2-5 L/s	50 L
5 – 20 L/s	210 L
20 – 30 L/s	500 L
30 – 50 L/s	1000 L
More than 50 L/s	Use other suitable methods

2.20.2 FLOW METERS TO MEASURE DISCHARGE RATE

The flow meter ***(Fig. A11)*** is installed in the delivery line from the pump. The flow meter must be properly calibrated before it is used and its piping must be of similar diameter to that of the discharge pipe. There must be no turbulent flow or entrained air in the discharge pipe before the meter. The discharged water must be free of solid material carried in suspension. Some flow meters have got two valves for discharge setting, a coarse and a fine setting valve.

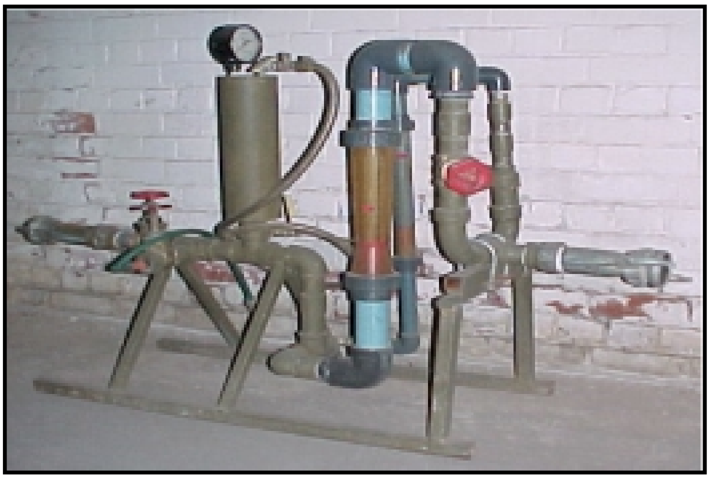

Fig. A11 Flow meter to measure discharge from pump

2.20.3 ORIFICE WEIRS TO MEASURE DISCHARGE

The orifice weir is commonly used to measure the discharge from a turbine or a centrifugal pump. It does not work when a piston pump is used because the flow from such a pump pulsates too much.

Orifice weirs must be installed in a horizontal position at the end of the discharge pipe. The orifice plate opening must be sharp, clean, bevelled to 45 degrees and have a diameter of less than 80 percent of the diameter of the approach tube to which it is fixed. The orifice plate must be vertical and centered on the end of the approach tube. There must be no leakage around the perimeter of the orifice plate mounting. The piezometer tube must not contain entrained air bubbles at the time of pressure head measurement. The latter measurement must be at least three times the diameter of the orifice.

2.21 WATER SAMPLING EQUIPMENT

Water samples should be taken during the pumping test. The sample should be taken at the end of the pumping test, 15 min. before the test is stopped. It does not matter what type of pumping test it is.

The person that takes the water sample should wash his or her hands before taking the sample (Hobbs and Marais, 1997). A 240 ml sample bottle should be used and this container should be rinsed at least three times with the water that is going to be sampled, i.e. that being pumped from the borehole. Fill the bottle so that a space of five to ten mm is left at the top. If the sample is to be sampled for macro-elements, the prescribed preservative should be added.

2.22 FLOOD LIGHTS

Long pumping tests always carry on during the night and it is always difficult to take water-level readings or to determine the delivery from the pump in the dark. It is therefore important to make sure that a strong and reliable flashlight or a floodlight forms part of the equipment on a pumping test outing. When electricity is available, electric floodlights can be placed at strategic places on the test terrain and by the flick of a switch, the necessary light is supplied to take readings easily.

2.23 OTHER EQUIPMENT

Normally, the pumping test site is in a remote area and then accommodation can become a problem. Because the pump testing contractor must be present at the pumping test site for the duration of the pumping test, it is very important that he should make arrangements to camp or stay at the site. A list of equipment needed is included in ***Appendix C***.

3. PERFORMING THE PUMPING TEST

3.1 PRE-ARRIVAL ON SITE ACTIONS

The pump test contractor must be appointed and he should be in possession of a letter of appointment before any actions to perform the pumping test are taken. A letter must confirm a telephonic appointment before any action is taken.

A date to perform the pumping test must be mutually agreed upon by the pumping test contractor and the person or authority that requires the pumping tests. If a representative is acting on behalf of the person or authority that requires the pumping test, all negotiations should be done with him.

The owner of the property should also be informed that a pumping test is going to be performed on his property and that there will be some activity on the property. This will even continue through the night with the aid of floodlights.

The removal of existing pumping equipment from the boreholes to be tested must be sorted out before the commencement of the pumping test. If it is the responsibility of the pump test contractor, he should inform the owner of the pump prior to the pumping test that the pump is going to be removed from the borehole for a period of time. If the pump is used to fill up a reservoir, the owner can do so before the equipment is removed.

All pumping activities from the pumping borehole as well as the aquifer should be stopped at least 72 hours prior to the start of the pumping test. During the pumping test, no pumping from boreholes in the vicinity of the pumping test borehole should take place. This can have a negative effect on the results of the pumping test. The responsibility to negotiate the seizure of all pumping activities in the vicinity of the borehole where the pumping test is to be done should be cleared before the start of the test. If it is the responsibility of the pump test contractor, all the affected parties should be contacted long before the start of the test.

The pump test contractor should plan the trip to go and do the pumping test properly. The equipment specified on the check-list ***(Appendix B)*** must be acquired and assembled. All the equipment must be tested at the office to ensure that it works properly before leaving for the pumping test. Camping equipment must also be packed and permission to stay on the property must also be obtained from the owner prior to the pumping test.

Information on trig beacons in the area where the pumping test is going to be performed should also be obtained. This information will be used to survey the pumping as well as the observation boreholes. This includes the elevation (z co-ordinates) as well as the positions (x and y co-ordinates).

3.2 ARRIVAL ON SITE ACTIONS

Everything had been organised and now the contractor goes out to the site to do the pumping test. The events that take place after the arrival on site normally take place in the same sequence every time and normally the one action must be finished before going on to the next task or action.

The different actions that take place during a pumping test will be discussed below. Detail on some of the topics was taken from the Minimum Standards and Guidelines for groundwater resources development for the community supply and sanitation program drawn up by PJ Hobbs and assisted by S.J. Marais (1997).

3.2.1 LOCATE CORRECT BOREHOLES (PUMPING AND OBSERVATION)

From the maps supplied with the tender documents by the representative of the person or authority that requires the pumping test, the contractor must now locate the borehole to be tested as well as all the observation boreholes. It will be best if the representative can be present on site on the first day of the pumping test so that all the boreholes involved in the pumping test can be located together. By doing this, there can be no uncertainties.

The contractor can also verify the positions of the boreholes if the representative supplied their coordinates. This can be done with the aid of a Global Positioning System (GPS). After all the relevant boreholes had been identified, they should be clearly marked and numbered.

The possibility of additional observation boreholes should also be investigated. The owner of the property or the person staying on the property can supply information about possible additional observation boreholes. This person should therefore be requested to supply information in this regard.

From the information on the observation boreholes, it must be decided which boreholes should be used for observation boreholes. The distances that these observation boreholes are away from the pumping borehole, as well as their location in relation to the pumping borehole, will help in making a decision in this regard.

3.2.2 REMOVAL OF EXISTING EQUIPMENT AT BOREHOLE

Before the pump testing equipment can be installed into the borehole to be tested, the existing equipment must be removed. This can either be the responsibility of the pump test contractor or that of the person or authority who wants the test to be done.

Great care should be taken when removing the existing equipment. This includes equipment on observation boreholes as well. Normally, the equipment had been in the boreholes for a long time and connections are rusted and very difficult to disconnect and to remove.

A tripod type of frame fitted with a block and tackle or a winch can be used to remove the equipment. The equipment should be neatly placed and stored away from the borehole to be tested so that it does not interfere with the pumping test. As much as possible information on the equipment should be recorded. This includes:

- the manufacturers name,
- type of pump,
- type of motor fitted to the pump,
- the depth to which the pump was installed,
- the power rating of the motor and
- the diameter, length and quantity of pump column sections.

All deficiencies and breaks on the equipment should be carefully written down and it should be reported to the representative as well as the owner of the equipment. The depth of the pump before removal, as well as specifications on the equipment, should be written down.

If the contractor is responsible for the removal of the equipment, he should also reinstall the equipment to the same condition that it was found in. If equipment in the identified observation boreholes might interfere with the pumping test, it should also be removed. The same procedure as described above should be followed when removing this equipment.

3.2.3 DETERMINING THE DEPTH AND DIAMETER OF THE BOREHOLE

The depth of the borehole should be determined in order to determine at what depth the pump to be used during the pumping test must be installed. To determine the depth of the borehole a bailer attached to a cable or a rope is used. The bailer is lowered into the borehole and when it reaches the bottom of the borehole, the rope or cable is marked so that the length can be determined with the aid of a tape measure. The collar of the borehole is normally used as the reference point to where the measurements are taken. The depth of the borehole should be written down on the data sheet with the other information.

The bailer that is lowered into the borehole also serves another purpose. When the bailer is lowered into the borehole, it can be determined whether or not the borehole had been closed up. Sometimes the sides fall in or tree roots block the borehole and it will be impossible to put the pump testing equipment into the borehole. The depth determined with the bailer can be compared to the depth supplied by the owner or representative.

The bailer should also be used to remove any loose debris lying at the bottom of the borehole. The silt inside the borehole may interfere with the pumping test and therefore it will be better to remove it before the pumping test commences.

The depth of the observation boreholes should also be determined. This can be done by using a weighted line and plumb bob.

The diameter of the borehole must be measured with a tape measure. This information should also be written down on the data sheet. Normally, the boreholes used in this country have got diameters of 165 mm, but it must definitely be measured.

3.2.4 DETERMINING POTENTIAL YIELD (SLUG TEST) AND DETERMINING POSSIBLE PUMPING TEMPO

Before the slug test is done, the diameter of the borehole should be measured. If it is not 165 mm a slug test cannot be performed. Before the slug test is done, the rest or static water level of the pumping borehole must be determined. This means that the distance from the collar of the borehole to the water level must be measured.

A rope must be attached to the slug with the prescribed volume and the distance from the collar of the borehole to the water level should be marked on the rope. This is done to ensure that the whole slug is submerged during the slug test. Insert a dip-meter into the borehole and get a stopwatch ready, so that the water levels can be measured at certain time intervals after the slug had been lowered into or taken out of the water. The contractor can now start with the slug test.

The slug is now quickly lowered into or taken out of the water and the increased or lowered water level is measured while the stopwatch is started. The time taken for the water level to stabilise to at least 90 percent of its original value is recorded. From the recession time versus yield chart ***(Fig. A6)***, the maximum yield of the borehole in L/h can be determined. This will give the contractor an idea of whether additional pumping tests will be warranted. If the possible yield of the borehole is found to be less than 0.3 L/s, then the consultant should be informed and a decision to continue or discontinue the testing should be taken. If the yield is more than 0.3 L/s the testing can continue.

3.2.5 INSTALLING PUMPING EQUIPMENT

If the slug test indicates that the possible yield from the borehole will be sufficient, other pumping tests can commence. The contractor must now install the pump testing equipment in the borehole that will be pump tested. There are mainly two types of pumps that are used for pump testing purposes. A positive displacement pump or a mono pump is used and in some instances, a negative displacement pump or submersible pump is used. The type of pump that is going to be used will determine the method of installation into the borehole.

If a mono pump is to be used for the pumping tests, this is the way that the contractor will go about installing the pump into the borehole. A tripod frame and winch can be used to perform this task. The section housing the rotor stator is attached to the cable of the winch and is lowered into the borehole. The shaft is connected to the rotor and the first rising mains column is attached to the housing; pushing it over the shaft does this. The section is lowered further into the borehole and when it has been lowered far enough, the next shaft and column are attached. This procedure continues until the intake of the pump is at the desired depth. Then the frame or stand, housing the discharge head, is placed over the borehole and fixed to the shafts and columns going down the borehole. From the discharge head, the horizontal delivery piping is attached. The power supply to the pump is now placed into position and it is connected to the pump by making use of vee-belts. The power supply can be a diesel or petrol engine with a gearbox or a variable speed throttle.

If a submersible pump is to be used for the pumping test, the pump can be installed into the borehole by using the same tripod frame and winch. The power supply is attached to the pump and it is properly sealed to prevent any water from getting into connections and causing the power to fail. The winch cable is securely attached to the pump in order to lower it down the borehole. In some cases, an additional safety rope is also attached to the pump to ensure that the pump can be removed from the borehole if the winch cable snaps.

The delivery pipe is also fixed to the pump and care should be taken that all connections are watertight. In some cases, shorter lengths of delivery pipes are used and then care should be taken that all connections are watertight. The non-return valve fitted at the bottom of the pump column should be checked to ensure that after the pump is stopped, no return flows take place via the delivery piping and pump. If this happens, it will have a negative effect on the accuracy of recovery test.

In some cases, the rising mains of a mono pump can have a large diameter (140-mm) and this can cause problems if the borehole diameter is not large enough. It can result in difficulty to install and remove the pump and it might also prevent the contractor from installing other equipment such as pressure probes or dip-meters in the borehole. The contractor should know the diameters of the different pumps to his disposal and he should try and obtain the diameter of the pumping borehole before the pumping test.

3.2.6 PLACEMENT OF PUMP (DEPTH)

In many instances, the depth at which the pump is to be placed will be specified in the contract, but if this is not the case, then the contractor must take that decision. The pump should be installed as low as possible in the borehole to ensure that the maximum available drawdown of the water in the aquifer is possible. This is very important, especially during longer pumping tests or in tests where the aquifer is strained. The behaviour of the water levels during shorter pumping tests differs significantly from long duration tests. After longer pumping times and higher abstraction rates, the true behaviour of the aquifer starts showing, especially when the main fractures are dewatered and water starts flowing from the matrix. This can cause the yield and therefore the water levels inside the borehole to drop significantly. This can even result in the water level reaching the intake level of the pump if it is not installed deep enough.

The pump should, however, not be installed too low in the borehole because debris and silt at the bottom of the borehole can be sucked into the pump. This will have a negative influence on the delivery of the pump and it might even result in the breakdown of the pump.

Depending on the depth of the borehole, the table below ***(Table A3)*** gives an indication of the distance at which the pump must be installed above the bottom of the borehole.

Table 3 Guidelines for test pump installation depth

WATER COLUMN IN BOREHOLE	TEST PUMP INSTALLATION DEPTH
Less than 5 m	Do not install pump
Between 5 and 30 m	Between 1.5 and 2.5 m above bottom
Between 30 and 60 m	Between 2.5 and 3.5 m above bottom
Between 60 and 90 m	Between 3.5 and 4.5 m above bottom
More than 90 m	Between 4.5 and 5.5 m above bottom

If the available drawdown (water column) in the borehole is less than 5 m, then a pumping test should not be performed. The initial drawdown during a pumping test can be ascribed to the storage effect in the borehole (well bore storage) and it lasts for only a few minutes, depending on the abstraction rate. In most instances, this initial drawdown is more than 5 m which makes a pumping test in a borehole with only 5 m of available drawdown a futile exercise.

3.2.7 INSTALLING DISCHARGE PIPING

The rising main had been connected to the pump and now the contractor must divert the water away from the pumping borehole. The water must be diverted in such a way that it does not affect the results of the pumping test in any way. The water should therefore not be able to flow back into the borehole and, more importantly, it should also not be able to circulate back into the aquifer.

To ensure there is no influence from the pumped water, it should be discharged away from the borehole. Horizontal delivery pipes are used to channel the water from the borehole to the point of discharge. This point should be at least 50 m away from the pumping borehole and, in some instances, it should even be up to 300 m away. When the contractor has to decide where the water that is pumped from the borehole is to be discharged, he should first determine the slope of the ground or the general gradient at the pumping borehole. The delivery pipes should be placed in such a way that the water will naturally drain away from the borehole and aquifer. During a pumping test, a lot of activity is taking place at the pumping borehole and the immediate vicinity. Water draining back to this area will make it impossible to move around on the site.

If a negative displacement pump or submersible pump is used for the pumping test, the volume of water pumped from the borehole during the pumping test is sometimes regulated with the aid of a valve. This valve is installed in the horizontal section of the delivery pipe and with this valve the delivery of the pump can either be increased or decreased. The discharge from a positive displacement pump is regulated by increasing or decreasing the revolutions from the motor driving the pump.

The discharge from the borehole during the pumping test is also measured on the delivery side of the pump. In some cases, a volumetric flow meter is installed in the delivery piping.

3.2.8 MEASURING DISCHARGE WATER OR PUMP DELIVERY

The rate at which water is pumped from the borehole during a pumping test must be measured and recorded at regular intervals. This information is very important because it is one of the parameters used in determining the possible yield of the borehole. There are various methods of determining the discharge rate during a pumping test, some expensive and some not that expensive. In some cases, the contract will specify the type of discharge measurement that should be used during the pumping test.

If no method is specified, the volumetric method will most probably be used. This is a cheap and easy method to use and the equipment does not take up a lot of space during travelling to the site. In this method, a container with predetermined volume is filled and the time taken to fill this container is logged. To ensure that a correct answer is obtained, the measurement must be repeated at least three times and the average of the three times should be recorded. The container must be placed level on the ground next to the end of the delivery pipe and the stopwatch should be capable of measuring in at least one-tenth of a second. The size of the container used in this method depends on the pumping rate during the test. There are certain guidelines for the container sizes and they are described in ***Table A2.*** Care should be taken that the end of the delivery pipe from the pump is not disturbed too much because this can affect the delivery from the pump.

If discharge measurement is required with the aid of a flow meter, such a meter should be installed on the horizontal section of the delivery pipe of the pump. The delivery pipe should be straight and of uniform diameter for a distance of at least four times the diameter of the pipe before the meter. A properly calibrated flow meter with a similar diameter to that of the discharge pipe should be used. There must be no turbulent flow or entrained air in the discharge pipe before the meter. The flow meter must be such that the operator should be able to adjust the flow with a course as well as a fine setting.

Another method of measuring the discharge from the pump is to install an orifice weir at the end of the discharge pipe. The orifice plate opening must be sharp, clean, bevelled to 45 degrees and have a diameter less than 80 percent of the diameter of the approach tube to which it is fixed. The piezometer tube must not contain entrained air bubbles at the time of pressure head measurement.

The discharge rate from the pump should be checked at the prescribed intervals and it should be recorded on the data sheet. The intervals are described in detail when the different pumping tests are discussed later in the report. The revolutions of the pump motor or the valves at the flow meter should be adjusted to keep flow constant. The connections between the discharge piping and the discharge measuring equipment should be watertight.

3.2.9 INSTALLING WATER-LEVEL MEASURING EQUIPMENT

Water level measurement inside the pumping borehole as well as the identified observation boreholes can be done with the aid of pressure probes and data loggers. This must be decided before the start of the pumping tests and the specifications of the equipment should be described in the contract. The contractor will be responsible to supply the equipment.

The pressure probes with an acceptable accuracy and sufficient cable length is lowered into the pumping as well as observation boreholes. The position of the pressure probe in the pumping borehole must be such that the pumping activity does not interfere with the accuracy of measurement. Turbulence caused by the pumping activity should not affect the readings produced by the pressure probe. To ensure that this does not happen the pressure probe should be installed at least 2 m above the pump. The pressure probe must also be installed deep enough so that the water level during pumping does not reach it. This also applies for the pressure probes in the observation boreholes.

The range of a pressure probe determines the accuracy of the probe. The higher the range, the less the accuracy. A probe with a 10 m range has got an accuracy of 1 cm over the full range; an acceptable accuracy in the industry, but sometimes the drawdown during pumping is more than 10 m. This means that the pressure probe will not be able to measure the total drawdown. However, there is a way around this limitation. The probe can be set up so that it will be able to measure the first 10 m of drawdown. During the pumping test, the probe can be lowered another 10 m to measure the rest of the drawdown. If necessary, the probe can be lowered even more, provided that a sufficient length of cable is available. By doing this, the probe can be utilised to measure the total drawdown to an acceptable accuracy. This technique also applies for the pressure probes in the observation boreholes, although the drawdown in the observation boreholes is normally not that dramatic. With a simple adjustment to the data, it will represent the actual drawdown picture.

The drawdown measured with the pressure probe during a pumping test is stored on a data logger. The contractor must now attach the pressure probe to a data logger. The data logger converts the electronic signal from the pressure probe to a height of pressure above the pressure probe. Normally, a multi-channel data logger is used and therefore several pressure probes can be attached to one data logger. The data logger should be placed centrally on the test site so that all the pressure probes can be connected to it. The data logger can be set up so that the water levels are stored at specified intervals. These intervals should be negotiated with the consultant before the commencement of the pumping test. A lap top computer or data reader can be used to extract the data from the data logger.

These data logging systems normally work on a 12-volt power supply. The power can either be supplied from a 220-volt system brought down with a transformer or a solar panel and regulator and, in some cases, a 12-volt battery is sufficient. The battery must be capable of keeping the system going for the duration of the test. The contractor is responsible to supply the power for the water measuring equipment. Now the system is operational and it should be set up properly.

3.2.10 DETERMINING REST OR STATIC WATER LEVEL

Before the contractor can set up the data logging equipment or start with the pumping test, rest or static water-level measurements must be taken inside the pumping borehole as well as all the observation boreholes. Installing the pump and the pressure probe in the pumping borehole has the same effect as a slug during a slug test. The static water level inside the borehole will rise, causing a pressure gradient. The water level will return to its original level with time, but it is good practice to check it against the original static water level taken for the slug test. A dip-meter is used to measure the distance from the collar of the borehole to the water level. This is known as the static or rest water level. The water levels in all the boreholes are measured and recorded on the data sheets. With this information, each channel on the data logger can be set up properly.

3.2.11 SETTING UP THE DATA LOGGING EQUIPMENT

The data logger should be powered and the reading on the display should be taken. If the range of the pressure probe is 10 m, the probe should be lowered into the water so that a reading of close to 10 m is obtained. The pressure probe should now be fixed to the collar of the borehole in such a way that it remains stationary at the same level for the duration of the test, except if the contractor lowers it to measure more drawdown. The pressure probe can be checked by powering the data logger and by pulling it up out of the water. The reading on the data logger should change when this happens. This procedure must be followed for every pressure probe that is installed. The borehole numbers must be entered into the correct channel of the data logger. For example, if the number allocated to the pumping borehole is U05 and the pressure probe from this borehole is connected to the first channel on the data logger, then the number U05 should be entered for channel one. In order to prevent confusion, the logging channel for every borehole should be noted on the data sheets. A data reset should be done on the data logger to get rid of all previously recorded data. The data logger should be allowed to register for a few minutes before the commencement of the pumping test. This will establish a definite reference line on the data logger. Now the data logging equipment is set up properly and the contractor can start with the actual testing.

3.2.12 CALIBRATION TEST

The contractor can now continue with the actual pumping test, but the pumping rate at which the test must be conducted is still unknown. In some instances, the abstraction rate for the pumping test can be prescribed as part of the contract and then this pumping rate will be used. Sometimes this is not the case. The contractor must try to determine the optimum rate at which the pumping test should be done so that the yield potential of the borehole can be determined and the productivity of the aquifer can be evaluated. The abstraction rate should allow the contractor to gather the maximum amount of information about the borehole as well as the aquifer supplying the water to the borehole. During a pumping test, the maximum amount of drawdown in the borehole should be achieved so that the parameters and characteristics of the borehole over the whole depth of the borehole can be determined.

If too low a pumping rate is used, no strain will be placed on the aquifer and shortcomings or constraints might not be picked up. If the pumping rate is too high, the water level inside the borehole will drop rapidly and it will reach the pump intake before the true characteristics of the borehole as well as the aquifer can be assessed.

To determine the optimum pumping rate, the contractor will do a calibration test on the borehole. Water is pumped from the borehole at three or more different pumping rates over short sequential periods of time. Normally, the pumping period for the calibration test is 15 min. The response of the water level to each known pumping rate is measured and recorded as per prescribed time schedule ***(Table A4)***. A complete calibration test data sheet is provided in ***Appendix D***. The calibration test provides a means of assessing the yield potential of the borehole according to the magnitude of the water-level decline associated with each pumping rate. From this information, the correct pumping rates for a stepped drawdown test as well as a constant rate pumping test can be decided.

Table A4 Time schedule for calibration test

Time (minutes)	Drawdown (meters)
1	
2	
3	
4	
5	
6	
7	
8	
9	
10	
11	
12	
13	
14	
15	

To perform the actual calibration test, the pump test contractor must:

- Insert the dip-meter into the borehole or the plastic tube inside the borehole and determine the static water level. Record the information on the data sheets.
- If no plastic tube is installed, insert the dip-meter into the borehole and determine the static water level.
- Make sure that the stopwatch is zeroed and working properly.
- Make sure that the data logger had been reset and that it is working properly.
- See to it that the equipment to measure the discharge from the pump is in place and that it is ready.
- Make sure that the capacity of the pump is known. Strain the pump so that about one third of the capacity will be discharged during the first run of the test.
- Start the pump.
- Measure the drawdown in the pumping borehole with the dip-meter as per the above prescribed time intervals.
- Measure the discharge from the pump at the beginning as well as at the end of the first run of the test.
- Stop the pump after the prescribed time has passed.
- Measure the recovery in the borehole as per the prescribed time intervals.
- Allow the water level to recover to more than 90 percent of the original static water level.
- Repeat the test for two-thirds as well as full capacity of the pump.
- Using this information, the contractor must decide on what pumping rates to use for the stepped drawdown as well as the constant rate tests.

The chart on the next page ***(Fig. A12)*** is an example of a calibration test done prior to a pumping test. From this figure it is clear that the drawdown of about 10 m obtained in 15 min. with the full pumping rate was much too high and this would have caused the water level to drop below the pump intake within a few hours. The second and third pumping rates produced drawdowns of between 3 and 5 m and because of this, a pumping rate of 3600 L/h was chosen for this particular pumping test.

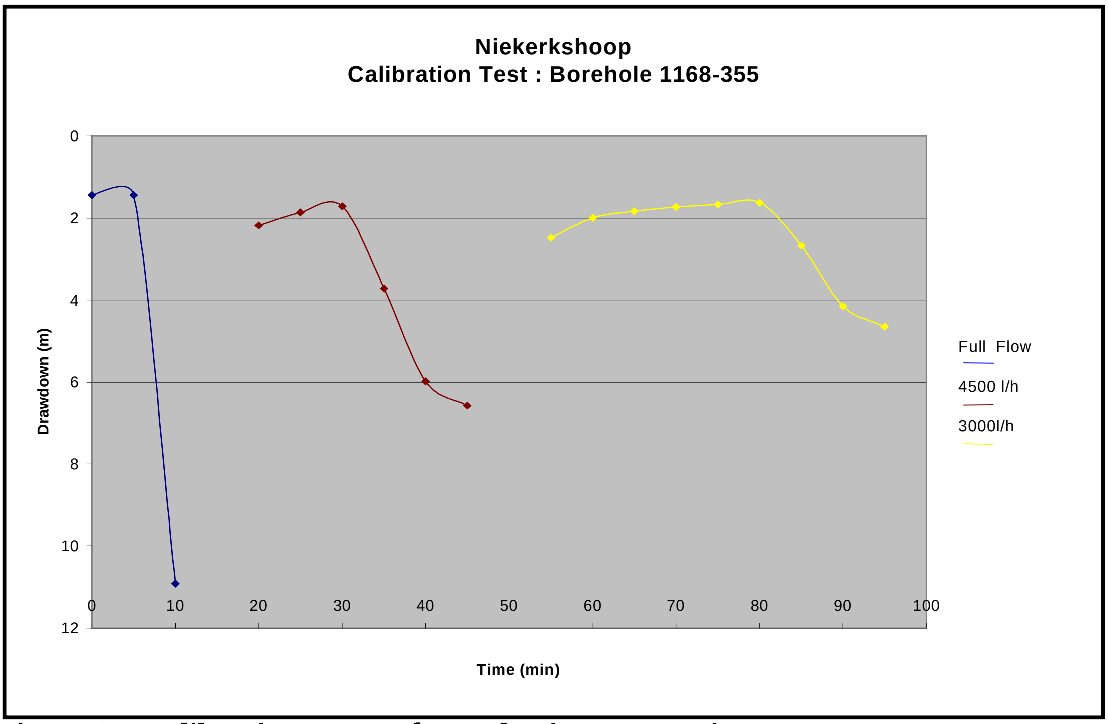

Fig. A12 Calibration test performed prior to pumping test

3.2.13 STEP DRAWDOWN TEST

The step drawdown test is a single-well test and it is performed to evaluate the productivity of a borehole. It also gives an indication of the optimum yield at which the borehole can be subjected to constant discharge testing, if required. The results of a step drawdown test will indicate whether further pump testing in the form of a constant discharge test is warranted or whether the borehole is judged to be sufficiently weak (potential yield less than 0.5 L/s) to make a utilisation recommendation without further testing. If the result of the stepped discharge test is positive, then a constant rate pumping test must be performed.

In performing a step discharge test, the borehole is subjected to three or more sequentially higher pumping rates, which is maintained for an equal length of time. The test is done by pumping the borehole at a low constant discharge rate until the drawdown stabilises. The constant discharge rate is then increased and the borehole is pumped until the drawdown stabilises again. The pumping rate is then increased again and the process is repeated. The time per pumping rate should be between 60 and 120 min. (Hobbs and Marais, 1997). A step length of 100 min. is recommended for the test. The drawdown in the borehole in response to each of the pumping rates must be measured and recorded in accordance with a prescribed time schedule.

The time schedule for a drawdown as well as a recovery step is given in the table ***(Table A5)*** below. A complete step drawdown data sheet will be supplied in ***Appendix D.*** The actual pumping rate maintained during each step of the test should also be measured and recorded. At the end of the pumping steps, the recovery inside the borehole should also be measured for the same period (if pumping lasted for 300 min., then recovery should be measured for 300 min.).

Table 5 Time schedule for each step of step drawdown test

Time (minutes)	Drawdown (meters)	Time (minutes)	Recovery (meters)
1		1	
2		2	
3		3	
4		4	
5		5	
6		6	
7		7	
8		8	
9		9	
10		10	
11		11	
12		12	
13		13	
14		14	
15		15	
16		16	
17		17	
18		18	
19		19	
20		20	
30		30	
40		40	
50		50	
60		60	
70		70	
80		80	
90		90	
100		100	
110		110	
120		120	
		150	

To perform the actual step drawdown test, the pump test contractor must:

- Insert the dip-meter into the plastic tube inside the pumping borehole and determine the static water level. Record the information on the data sheets.
- If no plastic tube is installed, insert the dip-meter into the borehole and determine the static water level.
- Make sure that the stopwatch is zeroed and working properly.
- Make sure that the data logger had been reset and that it is working properly.
- In order to determine parameters such as effective radius of the borehole and storativity of the aquifer the contractor must take water-level readings in the observation boreholes.

Therefore make sure that measurements are taken in the observation boreholes at the prescribed times.

- See to it that the equipment to measure the discharge from the pump is in place and that it is ready.
- By using information from the slug as well as the calibration tests, work out what the different steps for the step drawdown test will be. Strain the pump so that a low first constant discharge rate increment will be delivered.
- Start the pump.
- Measure the drawdown in the pumping borehole with the dip-meter as per the prescribed time intervals. Record this information on the data sheets.
- Measure the discharge from the pump at the beginning of the discharge increment. After that, check the discharge after 7, 15 and 60 min. Then take a final reading just before the end of the step.
- After about 10 min. into the test take a water sample. The borehole number, date and time must be indicated on the bottle. Take another sample just before pumping is stopped.
- By now, the drawdown will have stabilised and the chosen time interval (say 100 min.) will have passed.
- After the prescribed time, increase the delivery of the pump to the second constant discharge rate increment.
- Measure the second constant discharge rate increment from the pump. Check the discharge at the same intervals as prescribed above.
- Measure the drawdown at the prescribed time intervals. The drawdown will eventually stabilise.
- Increase the discharge rate to the third constant rate increment after the prescribed time. Follow the same procedure as described above.
- Just before the pump is stopped, take another water sample.
- After the prescribed time, stop the pump and start the recovery. Measure and record the recovery.
- Allow the water level to recover for the same period of time as the length of pumping.

The chart below ***(Fig. A13)*** is an example of a step drawdown test performed on a borehole. The effect on the water level with the increased pumping rate can clearly be seen.

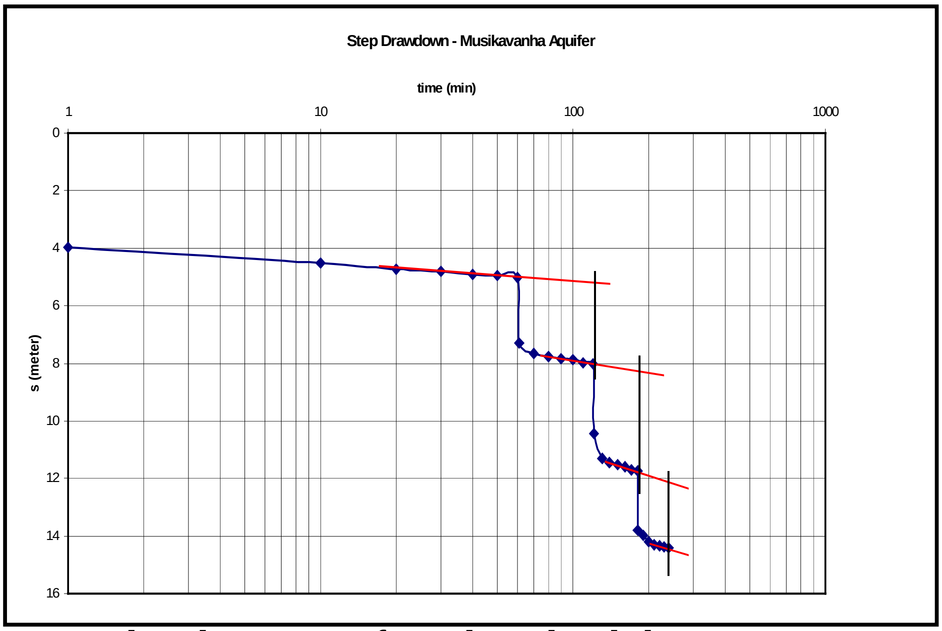

Fig. A13 Step drawdown test performed on a borehole

3.2.14 MULTIRATE TEST

This test is very similar to the step drawdown test, but there are some differences in the method of performing the test. This test will indicate whether further pumping tests are warranted or whether the borehole is judged to be sufficiently weak to make utilisation recommendations without further testing. This test will give an indication of whether the borehole is well-developed or not. With this method, the borehole efficiencies for the different abstraction rates can be determined. The main difference between this test and the step discharge test is that in determining the drawdown values (s), no extrapolation is necessary; this means that no guesswork takes place. The pumping rates may also be increased as well as decreased during the test, something not possible during the step discharge test. Additional transmissivity and storativity values can also be obtained while doing this test, also not possible with the step discharge test.

In performing a multirate test, the borehole is subjected to three or more sequentially higher pumping rates, which is maintained for an equal length of time. The test is done by pumping the borehole at about one-third of the expected operational yield of the borehole. After a certain time, pumping is stopped and recovery takes place. After recovery, increase the pumping rate to approximately two-thirds of the expected operational yield and pump the borehole for the same time interval as in the first case. Recover again after pumping. Repeat the procedure for a pumping rate equal to that of the expected operational yield of the borehole. Increase the pumping rate to approximately 1.25 or 1.5 the expected operational yield of the borehole and repeat the procedure. A step length of 60 min. is recommended for the length of the pumping times. The drawdown in the borehole in response to each of the pumping rates must be measured and recorded in accordance with a prescribed time schedule.

In performing the actual multirate test, the contractor will:

- Insert the dip-meter into the plastic tube inside the pumping borehole and determine the static water level. Record the information on the data sheets.
- If no plastic tube is installed, insert the dip-meter into the borehole and determine the static water level.
- Make sure that the stopwatch is zeroed and working properly.
- Make sure that the data logger had been reset and that it is working properly.
- In order to determine parameters such as the transmissivity and storativity of the aquifer, the contractor must take water-level readings in the observation boreholes. Therefore make sure that measurements are taken in the observation boreholes at the prescribed times.
- See to it that the equipment to measure the discharge from the pump is in place and that it is ready.
- By using information from the slug as well as the calibration tests, work out what the expected operational yield of the borehole will be. For the first step in the multirate test, the pumping rate should be approximately one-third of the expected operational yield. Strain the pump so that this discharge rate increment will be delivered.
- Start the pump.
- Measure the drawdown in the pumping as well as the observation boreholes with the dip-meter as per the prescribed time intervals. Record this information on the data sheets.
- Measure the discharge from the pump at the beginning of the discharge increment. After that, check the discharge after 7, 15 and 60 min.
- After about 10 min., take a water sample. The borehole number, date and time should be indicated on the bottle.
- After the prescribed time, stop the pump and measure the recovery. Recovery must be measured for the same time interval as used for pumping.
- Increase the pumping rate to approximately two-thirds of the expected operational yield of the borehole and proceed with the pumping for the same time interval as in the first instance. Check the discharge at the same intervals as prescribed above.
- Measure the drawdown at the prescribed time intervals. Remember to measure the drawdown in the observation boreholes as well.
- For the third interval, increase the discharge rate to approximately the expected operational yield of the borehole. Repeat the procedure and after the prescribed time, stop the pump. Measure the recovery again.
- Take another water sample.
- For the next interval, increase the discharge rate to approximately 1.25 or 1.5 the expected operational yield of the borehole. Repeat the procedure and after the prescribed time, stop the pump. Measure the recovery again. During this increment, care should be taken that the water-level does not reach the intake of the pump.

The chart below ***(Fig. A14)*** is an example of a multirate test done on the Campus Test Pumping Terrain at the University. The blue line represents the drawdown in the borehole, while the red line indicates the pumping rates.

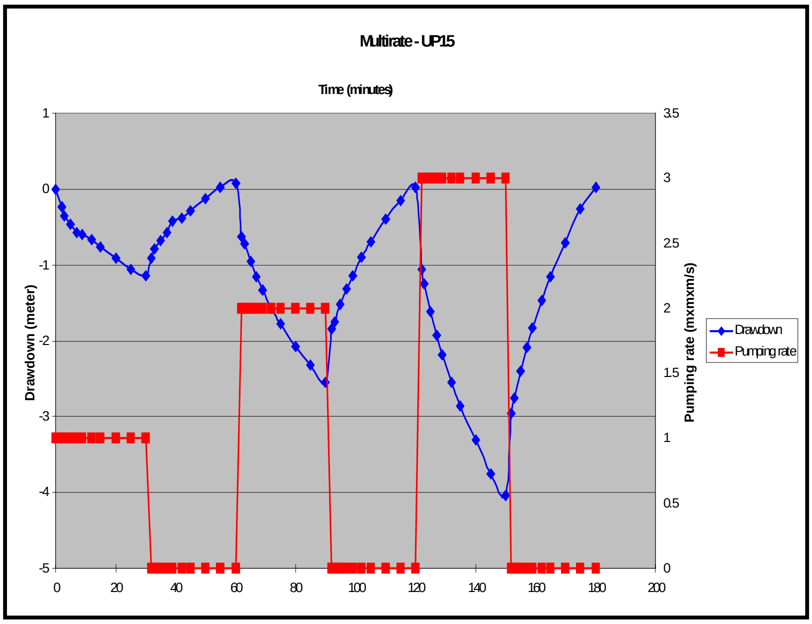

Fig. A14 Multirate test done on borehole UP15

3.2.15 ONE-HOUR STEP DRAWDOWN TEST

The proposed step drawdown test of a minimum of one hour, which is set as one of the minimum requirements for sustainable yield estimation, is performed exactly in the same way than the step drawdown test discussed earlier. The only difference is that the length of the steps could differ (e.g. rate could be changed after 5, 10, 25 or 40 min.). If a well loss coefficient must be estimated, use can be made of the Helweg method (Helweg, 1994).

Helweg wrote the step drawndown equation as:

$$s = AQ + B' LogtQ + C' LogtQ^{p}$$

If t is constant and p=2, this equation simplifies to the well-known Jacob well loss equation. The unknown coefficients A, B', C' and p are estimated with a least square method.

3.2.16 CONSTANT RATE TEST

This test is performed to assess the productivity of the aquifer according to its response to the abstraction of water. This response can be analysed to provide information in regard to the hydraulic properties of the groundwater system. With this information, we can determine an optimum yield for the long- and medium-term utilisation of the borehole.

A constant rate pumping test is performed by pumping water from a borehole at a single pumping rate which is kept constant for an extended period of time. It is critical that the pumping rate during the entire duration of the test be kept as constant as possible. The pumping rate should be set at a yield, which will be able to be maintained for the duration of the pumping test, and in the process utilising more than 70 percent of the available drawdown. The available drawdown, however, should not be exhausted. The drawdown in the water-level in the borehole as well as the observation boreholes is measured according to the prescribed time schedule. Before starting with the constant rate pumping test, the borehole should be allowed to recover completely from the stepped discharge or multirate tests. If the borehole is recovered overnight, the water level will return to within a few cm of the original water level.

The time schedule for a constant rate test is in the table ***(Table A6)*** below. A complete constant rate data sheet is supplied in ***Appendix D.***

Table A6 Constant rate test prescribed time schedule

Time (minutes)	Time (minutes)	Time (minutes)	Time (minutes)	Time (minutes)
1	11	25	240 (4h)	1800 (30h)
2	12	30	300 (5h)	2160 (36h)
3	13	40	360 (6h)	2520 (42h)
4	14	50	420 (7h)	2880 (48h)
5	15	60	480 (8h)	3600 (60h)
6	16	75	600 (10h)	4320 (72h)
7	17	90	720 (12h)	
8	18	120	900 (15h)	
9	19	150	1080 (18h)	
10	20	180 (3h)	1440 (24h)	

The actual pumping rate maintained during the test should be measured and recorded regularly. The pumping rate must be checked and adjusted, if necessary, after 7, 15, 60, 120 and 180 min. From then on, the pumping rate should be checked whenever the water-level measurements are taken. At the end of the constant rate test, the recovery inside the borehole should be measured.

In performing the actual constant rate pumping test, the contractor will:

- Insert the dip-meter into the plastic tube inside the pumping borehole and determine the static water level. Record the information on the data sheets.
- If no plastic tube is installed, insert the dip-meter into the borehole and determine the static water level.
- Make sure that the stopwatch is zeroed and working properly.
- Make sure that the data logger had been reset and that it is working properly.
- In order to determine parameters such as the transmissivity and storativity of the aquifer, the contractor must take water-level readings in the observation boreholes. Therefore make sure that measurements are taken in the observation boreholes at the prescribed times.

- See to it that the equipment to measure the discharge from the pump is in place and that it is ready.
- By using information from the slug, calibration, step discharge and multirate tests, work out what the pumping rate for the constant rate pumping test should be. The pumping rate should be such that the water level inside the borehole does not drop to below the intake of the pump at any time during the pumping test. Strain the pump so that this desired discharge would be delivered by the pump.
- Start the pump.
- Measure the drawdown in the pumping as well as the observation boreholes with the dip-meter as per the prescribed time intervals. Record this information on the data sheets.
- Measure the discharge rate from the pump at the beginning of the test. After that, check the discharge after 7, 15, 60, 120 and 180 min. From then on, the pumping rate should be checked whenever the water-level measurements are taken.
- A water sample must be taken after about 10 min. of pumping and another one just before the end of the test. The borehole number, date and time should be indicated on the sample bottle.
- After the prescribed time, stop the pump and measure the recovery. Recovery must be measured for the same time interval as used for pumping.

The chart below ***(Fig. A15)*** is an example of a constant rate test that was done on borehole UO5 on the Campus Test Site. It can clearly be seen that there was a rapid drawdown at the beginning of the test and that it decreased towards the end of the test.

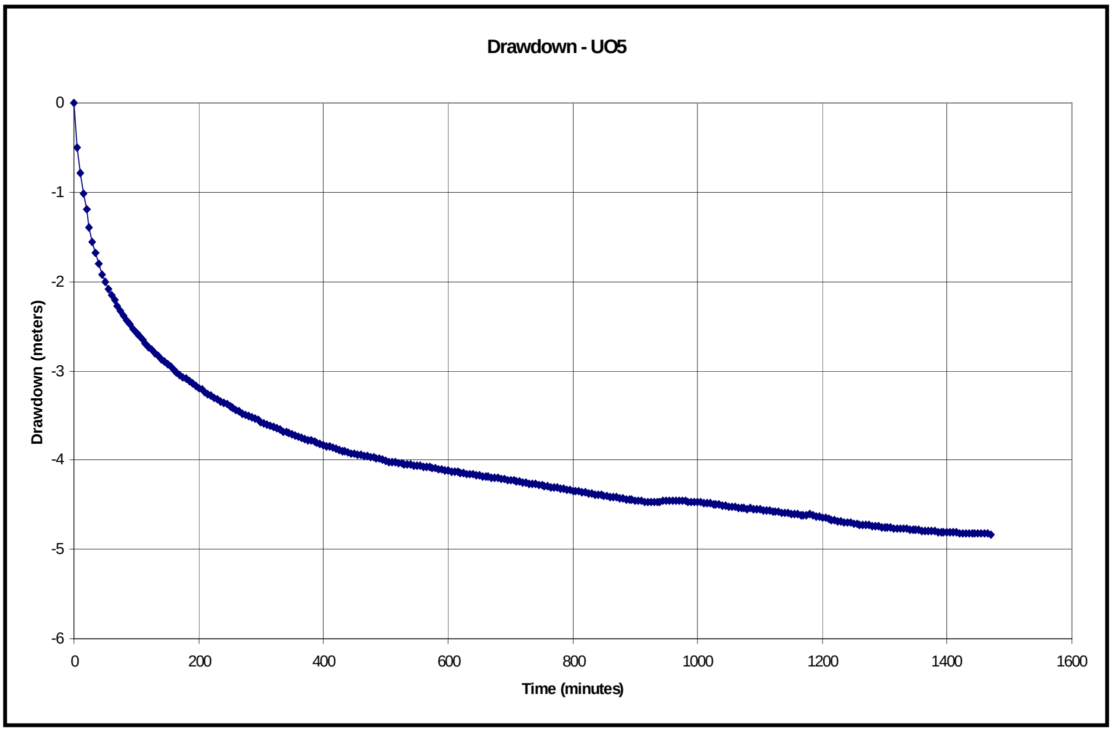

Fig. A15 Constant rate pumping test done on UO5

3.2.17 RECOVERY TEST

When the pump is shut down, the water levels in the pumping borehole as well as the observation boreholes will start to rise. It is very important to measure the recovery because parameters, such as the transmissivity of the aquifer, can be determined. This will act as an independent check on the results obtained with the pumping test and at a fraction of the cost of the pumping test.

In some instances, recovery data are more reliable than pumping test data because the recovery takes place at a constant rate, whereas the constant discharge during pumping is sometimes very difficult to obtain in the field. By making use of the information gathered with a recovery test, the number of hours that a borehole should be pumped at a tested rate can be determined.

The recovery of the water-level should be measured for a period equal to the duration of the constant rate pumping test or until the water level has recovered fully, whichever occurs first. Water-level measurements should be taken at the same intervals as during the constant rate pumping test.

The chart below ***(Fig. A16)*** is an example of a recovery test that was performed on borehole UO5 on the Campus Test Site. The last section of the chart represents the recovery test.

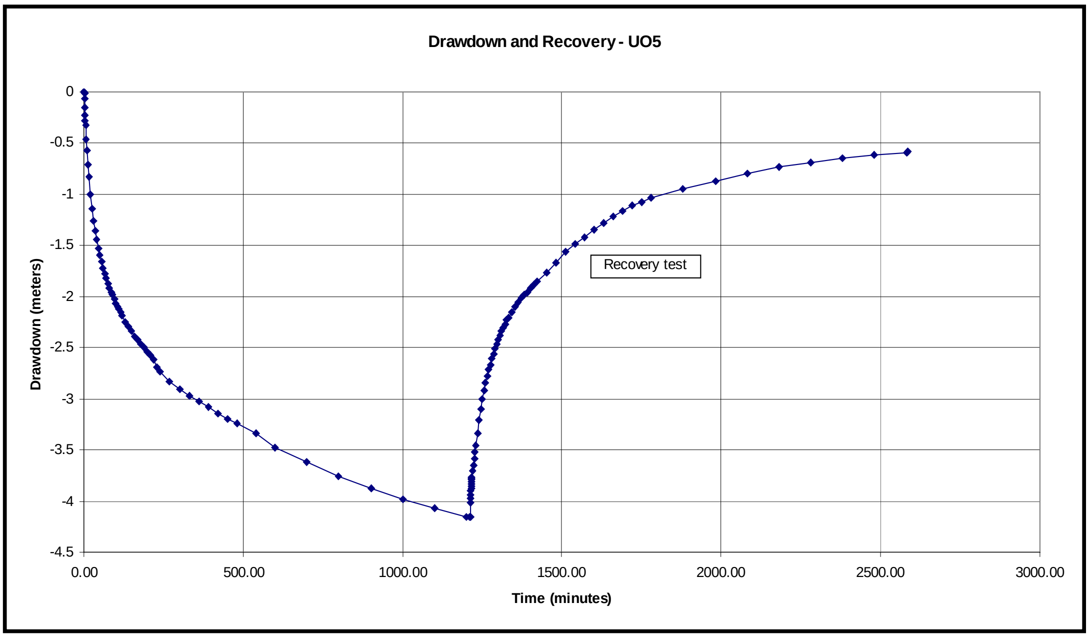

Fig. A16 Recovery test on UO5

3.2.18 DURATION OF THE CONSTANT RATE PUMPING TEST

The duration of the test will depend on the objective of the test. If the objective is parameter estimation, the test must be long enough such that a radial phase flow could be identified. If, however, the objective is only to estimate the sustainable yield of the borehole, the borehole must be stressed in such a way that the position of the main water strike is reached after about 8 hours.

The duration of the constant rate pumping test shall not be less than 12 h and, in some instances, it might even last for 72 h or more if the objective is the estimation of aquifer parameters for well field management. This criterion is set according to the Reconstruction and Development Program Rules of South Africa.

A good practice is to, while the pumping test is in progress, plot the water levels obtained during a constant rate pumping test on semi-log paper (the time is plotted on the logarithmic scale). If considerable changes in the gradient of the curve are noticed, it may be considered to extend the duration of the test so that more measurements will point out the new gradient and in effect new parameters. In many circles, the duration of constant rate pumping tests remains a controversial issue. Different opinions exist about this issue and it will be investigated and discussed in more detail later in this report. Continued pumping without achieving significant drawdown does not make any sense. The pump test contractor should also not just continue with a constant rate pumping for several hours without obtaining information that makes sense, because he is paid by the hour.

3.3 ABORTING TESTING

It is inevitable that, in some instances, pumping tests will be interrupted. These interruptions might be planned or, in some cases, it might be due to breakdowns or other problems.

If the data collected during a pumping test are evaluated during the test and it is found that sufficient information had been collected for an adequate scientific evaluation thereof, then the consultant can order the pumping test contractor to stop with the test.

If the pumping test is not performed according to the prescribed criteria as set out before in the contract, the consultant can order the pumping test contractor to stop the test and to do the test over. In this case, the pumping test contractor should allow the water level in the borehole to recover to the original rest water level or to within five percent thereof. Then the next attempt to do the pumping test can start.

During a pumping test, pumping can be interrupted due to mechanical failure or breakdowns. If something like this takes place, there are certain steps that need to be taken to correct the situation. For each of the different pumping tests, there is a different procedure that needs to be followed.

3.3.1 ABORTING CALIBRATION TEST

After the stoppage or breakdown in the test, start to record the recovery of the water level as per the prescribed time intervals. The water level must be recovered to the initial or rest water level or to a level within five percent of the initial water level. Fix the breakdown and then start the calibration test over again. Do not take the information gathered prior to the breakdown into account, but start the test as if it is the first attempt.

3.3.2 ABORTING STEP DISCHARGE TEST

After the stoppage or breakdown, record the time and start to measure the recovery at the prescribed time intervals. If the breakdown occurred during the first or second step of the stepped discharge test, recover the borehole to the initial water level. After the recovery, restart the step discharge test as if is the first attempt at the test. If the breakdown occurs during the third step of the step discharge test and the stoppage can be fixed within five minutes, continue at the same pumping rate prior to the breakdown and finish the test. Only one breakdown like this is allowed.

If a second breakdown occurs during the test, proceed as described for the first step breakdown and recover and restart the test from the beginning after the recovery. If the breakdown occurs during the fourth step of the test, proceed in the same manner as a breakdown in the third step. If the breakdown at this stage cannot be fixed within the five minutes, continue and measure the recovery as per the prescribed time intervals as if the test had been fully completed.

3.3.3 ABORTING MULTIRATE TEST

If the breakdown occurs during the first step of the multi-rate test, recover as prescribed and restart the test as if it is the first time. If the breakdown occurs during the next steps of the test, recover to the initial water level and do the step over as if has not been performed before.

3.3.4 ABORTING CONSTANT RATE PUMPING TEST

Note the time at which the breakdown occurred and start the recovery of the water level as per the prescribed time. If the breakdown occurred within two hours after the test was started, the test must be aborted. A restart is possible after full recovery.

If the breakdown occurs later than two hours into the test and the breakdown can be fixed within the time spans given in the table below ***(Table A7)***, continue with the test at the same pumping rate as before the breakdown.

Table A7 Period allowed for breakdown and continuation of testing

TIME BREAKDOWN OCCURRED	PERIOD ALLOWED FOR REPAIR
2 – 4 hours	6 minutes
4 – 6 hours	12 minutes
6 – 8 hours	18 minutes
8 – 10 hours	24 minutes
10 – 12 hours	30 minutes
12 – 14 hours	36 minutes
14 – 16 hours	42 minutes
16 – 18 hours	48 minutes
18 – 20 hours	54 minutes
Longer than 20 hours	60 minutes

If the breakdown cannot be fixed and if the pump cannot be started within one hour of the breakdown occurring, recover the water level to the initial water level or within five percent thereof. Then restart the pumping test as if it is the first attempt. If the breakdown occurs after approximately 80 percent of the planned duration of the constant rate test, continue with the recovery as per prescribed time intervals. The allowable elapsed time in regard to selected constant rate test durations in order for this specification to be acceptable is given in the table below ***(Table A8).***

Table A8 Period after which constant rate test can be considered completed

CONSTANT RATE TEST DURATION	ALLOWABLE TIME ELAPSED
24 hours	20 hours (80 % of total time)
36 hours	30 hours (83 % of total time)
48 hours	38 hours (79 % of total time)
72 hours	60 hours (77 % of total time)

3.4 OTHER IMPORTANT MEASUREMENTS TO BE TAKEN

As much as possible information should be gathered during the pumping test, because anything of importance left out during the pumping test might result in the test being a failure. At the time, it might not seem important to include the information in the data sheet, but at a later stage, it will be impossible to obtain that specific piece of information. A golden rule is never to guess at any information that has not been measured. The integrity of the pump test contractor might be on the line. So when in doubt, measure.

The diameters of all the boreholes, including the observation boreholes, must be measured. The diameter of the pumping borehole is important when doing a slug test. The theory for slug tests was developed with information from 165 mm boreholes. The theory for slug tests can therefore not be used on other diameter boreholes. The diameters of boreholes are required in most of the software packages.

The distances to observation boreholes from the pumping borehole must be measured. Some parameters such as storativity are distance dependent, so this information can be evaluated against the distances that observation boreholes are away from the pumping borehole. The straight line distance should be recorded and it is recommended that the distances between the observation boreholes be measured as well.

For all the boreholes used in the pumping test, the collar heights must be measured. This is the height from the ground to the top of the concrete column, housing the casing of the borehole. This measurement is important because all water-level measurements are taken with the concrete column as reference. This measurement allows the user to relate the information inside the borehole to ground level.

3.5 ADDITIONAL INFORMATION OF AREA

As much as possible information, how irrelevant it might seem at the time, should be collected during the pumping test. This information might come in handy at a later stage and it might even clear up uncertainties that may come out during the pumping tests. The pumping test contractor must therefore try to obtain information on the following.

3.5.1 BOUNDARIES

In practice, no aquifer extends infinitely and they are bounded laterally in one way or another. At some point, a boundary will start to interfere with the abstraction of water from that aquifer. A boundary may consist of a geological barrier that delineates the lateral extent of the aquifer or it may consist of a geological formation with lower permeability or even zones within a formation of lower permeability. In some cases, these boundaries may even show in the geology protruding from the ground. The pump test contractor must keep his eyes open for these formations and he must make a note about all such possible boundaries.

Groundwater will normally follow the natural gradient of the topography of the surface of an area. The pump test contractor should take note of the general topography of the area surrounding the borehole being pump tested. A borehole situated next to a mountain range may be influenced by that mountain range. The mountain range may act as a boundary on the aquifer. Therefore it is important that the pump test contractor make notes or even a schetch of the area showing all mountains, koppies and hills. Distances from the borehole to these protrusions should also be noted.

3.5.2 WATER ABSTRACTION

Before the pumping test is performed, all pumping activities should be stopped in the immediate area of the borehole to be tested. This should be arranged beforehand and pumping should be stopped at least 72 hours before the pumping test starts. The groundwater level should have enough time to recover to its natural static or rest water level position. The pump test contractor must make enquiries into whether pumping activities were stopped in time for the aquifer to recover. If somebody kept on abstracting water from the aquifer within this recovery period prior to the pumping test, it must be noted. The abstraction rates, as well as the length of time that water was abstracted, must be written down.

The pumping test contractor should also keep an eye open to see whether any pumping activity takes place during the pumping test. The pumping activities must be monitored and pumping rates, as well as pumping times, must be written down. This information can be used to interpret uncertainties in the drawdown curves of the pumping test.

3.5.3 RECHARGE

Although recharge due to rainfall normally has a delayed effect, all rainfall figures during the pumping test period should be written down. Normally, rainfall figures are available (farmers, the public and weather bureau) and the pumping test contractor should include these figures in his report. Rainfall figures prior to the pumping test should also be acquired and it should be included in the report.

Dams and rivers situated close to the pump testing site can cause recharge to the aquifer and it is good practice to mention this in the report that will be drawn up after the pumping test is completed.

3.5.4 IRRIGATION

All irrigation activities prior as well as during the pumping test should be written down by the pump test contractor and it should be included in the report on the pumping test. Sometimes water from an external source such as a river, dam or channel far away from the pump testing site is used for irrigation and this water can act as a recharge to the aquifer. The pumping test contractor should keep an eye open for this and it should be mentioned in his report. This external influence could have a negative effect on the results of the pumping test.

3.5.5 WATER-LEVEL RESPONSE TO SEASONS

It is general knowledge that the water levels in aquifers fluctuate during the different seasons. This is caused by the time of year that recharge normally takes place as well as the amount of abstraction that takes place at different times of the year. Some farmers that make use of groundwater extensively keep record of the groundwater levels in their area. In some areas, like in the Molopo district, the government monitors groundwater levels and the pump test contractor should make enquiries to obtain information in this regard. This information should be supplied in his report. This can help in determining the availability of water in the aquifer at different times of the year and this can influence the determination of the possible yield of a borehole.

3.6 PHOTOGRAPHS

A good set of photographs can yield valuable information. The results of the pumping tests performed by the contractor will, in many cases, be interpreted and evaluated by independent people without any knowledge of the area where the pumping test was performed. Photographs will provide these people with an idea of how the terrain and the surroundings look. Possible boundaries can also be described in a panoramic photograph image of the area.

3.7 PLOTTING POSITIONS OF BOREHOLES ON MAPS AND LOOKING AT AERIAL PHOTOGRAPHS

The pump test contractor should try to obtain a 1:50 000 map of the area where the pumping test is taking place. The positions of the pumping as well as the observation borehole should be plotted on these maps. A map must be included in the report on the pumping test because a map can supply a lot of information about the area surrounding the test area. Detail on a map can include surface contours, hills, mountain ranges, access roads, rivers and boreholes.

Aerial photographs can also supply information that can help to understand the area surrounding the borehole better. A good copy of an aerial photograph with the positions of the different boreholes plotted on it should also be attached to the report of the pump test contractor.

3.8 CHECKING CO-ORDINATES WITH GPS

A Global Positioning System should be used by the pump test contractor to determine the latitude and longitude (x and y co-ordinates) of all the boreholes involved in the pump testing. Although the accuracy is not similar to that of a survey, the method of determining the position is sufficient to plot the positions on the 1 : 50 000 map. Normally, the height measurement (z co-ordinate) of a GPS is not very accurate and it should not be used in any calculations.

3.9 SURVEYS

If it is specified in the contract, the contractor should make use of a land surveyor to determine the accurate latitude, longitude, as well as the height of the boreholes involved in the pumping test. Sophisticated survey equipment is used to tie the boreholes in with the trig beacons with latitude, longitude and height situated in the area. The collar height of each borehole is determined and is used in calculations.

3.10 CHANGE IN WATER COLOUR AND TEMPERATURE

During the pumping test the contractor should regularly look at the colour of the water and any change in the water-colour should be noted. The time at which the change in colour took place must also be written down and the contractor must describe the color that the water changed to.

Regular temperature readings should also be taken at the discharge point from the pump. The temperature readings should be logged against time and any change in temperature should be mentioned.

3.11 REMOVAL OF THE EQUIPMENT

After all the pumping tests had been conducted and satisfactory results were obtained, the pumping test contractor can now start to remove his equipment from the boreholes. First, the data logging equipment can be removed and after that the pumping equipment.

3.12 REINSTALLATION OF EXISTING EQUIPMENT

After the equipment of the pumping test contractor had been removed from the borehole the equipment that was removed to perform the tests can be reinstalled. All the equipment should be installed to the same condition that it was found in and the contractor should ensure that everything is in working order. The reinstallation of the equipment should be done to the satisfaction of the owner.

3.13 CLEANING UP OF THE TERRAIN

Before the pumping test contractor leaves the terrain, he should make sure that the site is properly cleaned. All papers and loose material should be picked up and it should be removed. The condition of the terrain should meet the approval of the owner of the property.

3.14 EVALUATION OF THE FIELD DATA AND OTHER INFORMATION

The data that were gathered during the pumping test must now be prepared and evaluated to determine the different parameters and characteristics of the borehole and aquifer. This is normally done at the pumping test contractor's office and it is presented in the form of a report to the consultant or the owner.

NOTE: An EXCEL program for data entering of the calibration, variable rate and constant discharge tests are available from IGS.

Appendix B. CHECK-LIST FOR PUMP TEST EQUIPMENT

NO	DESCRIPTION	QUANTITY	PACKED
1	Global Positioning System (GPS)		
2	Survey equipment (Theodilite, range rods)		
3	Compass		
4	Camera		
5	Electric lead		
6	Adapter plugs		
7	Flat screwdriver		
8	Phillips screwdriver		
9	Generator		
10	Jerry cans		
11	Fuel (Petrol or diesel)		
12	Oil		
13	Funnel		
14	Pump – Small capacity		
15	Vertical delivery pipes for small pump		
16	Horizontal delivery pipes for small pump		
17	Hose clamps for small pump		
18	Valve to reduce delivery from small pump		
19	Pump – Large capacity		
20	Vertical delivery pipes for large pump		
21	Horizontal delivery pipes for large pump		
22	Hose clamps for large pump		
23	Valve to reduce delivery from large pump		
24	Twenty litre bucket		
25	Fifty litre bucket		
26	Stopwatch		
27	Flow meter		
28	Flow meter fittings		
29	Tripod frame		
30	Block and tackle		
31	Winch		
32	Additional chain		
33	Slug test cylinder		
34	Bailer		
35	Ski rope (100 meters)		
36	Pressure probe (s)		
37	Data logger		
38	Data cable		
39	Solar panel		
40	Battery for data logging equipment		
41	Dip-meter		
42	Extra batteries for dip-meter		
43	Five meter tape measure		
44	Fifty meter tape measure		

45	Hundred meter tape measure		
46	Shifting spanner		
47	Baboon wrench		
48	Pliers		
49	Hammer		
50	Open-ring spanners		
51	Thin wire		
52	Cable ties		
53	Spade		
54	Tree saw		
55	Water sample bottles		
56	Marker pen		
57	Thread tape		
58	Masking tape		
59	Floodlight		
60	Flashlight		
61	Pen		
62	Data sheets – Constant rate test		
63	Data sheets – Calibration test		
64	Data sheets – Step drawdown test		
65	Data sheets – Multirate test		
66	Writing paper		
67	Lap top computer		
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Appendix C. CHECK-LIST FOR OTHER EQUIPMENT

NO	DESCRIPTION	QUANTITY	PACKED
1	Tent		
2	Tent pegs		
3	Camping beds		
4	Mattress		
5	Matches		
6	Fire wood		
7	Charcoal		
8	Chair		
9	Table		
10	Water container		
11	Water		
12	Gas braai		
13	Umbrella		
14	Hat		
15	Suntan lotion		
16	Bedding		
17	Mosquito repellent		
18	Alarm clock		
19	Pots and pans		
20	Kettle		
21	First aid kit		
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APPENDIX D

PUMP TEST DATA SHEETS

APPENDIX D

PUMP TEST DATA SHEETS

CALIBRATION TEST DATA SHEET

General Information	Pumping Borehole Information	Remarks
Project No:	Depth of Pump (m):	
Borehole No:	Collor Height (m):	
Site Name:	BH diameter (m):	
Farm Name:	Depth of BH (m):	
Coordinates	Static W/L (m):	
Latitude:		
Longitude:		

Discharge rate 1

Date: Time:

Static W/L (m):

Time (min)	Drawdown s (m)	Yield (l/s)	Time (min)	Recovery s' (m)
1			1	
2			2	
3			3	
4			4	
5			5	
6			6	
7			7	
8			8	
9			9	
10			10	
11			11	
12			12	
13			13	
14			14	
15			15	
			20	
			30	
			40	
			50	
			60	
			70	
			80	
			90	
			100	

Discharge rate 2

Date: Time:

Static W/L (m):

Time (min)	Drawdown s (m)	Yield (l/s)	Time (min)	Recovery s' (m)
1			1	
2			2	
3			3	
4			4	
5			5	
6			6	
7			7	
8			8	
9			9	
10			10	
11			11	
12			12	
13			13	
14			14	
15			15	
			20	
			30	
			40	
			50	
			60	
			70	
			80	
			90	
			100	

Discharge rate 3

Date: Time:

Static W/L (m):

Time (min)	Drawdown s (m)	Yield (l/s)	Time (min)	Recovery s' (m)
1			1	
2			2	
3			3	
4			4	
5			5	
6			6	
7			7	
8			8	
9			9	
10			10	
11			11	
12			12	
13			13	
14			14	
15			15	
			20	
			30	
			40	
			50	
			60	
			70	
			80	
			90	
			100	

Discharge rate 4

Date: Time:

Static W/L (m):

Time (min)	Drawdown s (m)	Yield (l/s)	Time (min)	Recovery s' (m)
1			1	
2			2	
3			3	
4			4	
5			5	
6			6	
7			7	
8			8	
9			9	
10			10	
11			11	
12			12	
13			13	
14			14	
15			15	
			20	
			30	
			40	
			50	
			60	
			70	
			80	
			90	
			100	

Discharge rate 5

Date: Time:

Static W/L (m):

Time (min)	Drawdown s (m)	Yield (l/s)	Time (min)	Recovery s' (m)
1			1	
2			2	
3			3	
4			4	
5			5	
6			6	
7			7	
8			8	
9			9	
10			10	
11			11	
12			12	
13			13	
14			14	
15			15	
			20	
			30	
			40	
			50	
			60	
			70	
			80	
			90	
			100	

Discharge rate 6

Date: Time:

Static W/L (m):

Time (min)	Drawdown s (m)	Yield (l/s)	Time (min)	Recovery s' (m)
1			1	
2			2	
3			3	
4			4	
5			5	
6			6	
7			7	
8			8	
9			9	
10			10	
11			11	
12			12	
13			13	
14			14	
15			15	
			20	
			30	
			40	
			50	
			60	
			70	
			80	
			90	
			100	

STEPPED DISCHARGE TEST DATA SHEET

General Information	Pumping Borehole Information	Remarks
Project No:	Depth of Pump (m):	
Borehole No:	Collor Height (m):	
Site Name:	BH diameter (m):	
Farm Name:	Depth of BH (m):	

Discharge rate 1

Date: Time:

Static W/L (m):

Time (min)	Drawdown s (m)	Yield (l/s)	Time (min)	Recovery s' (m)
1			1	
2			2	
3			3	
4			4	
5			5	
6			6	
7			7	
8			8	
9			9	
10			10	
11			11	
12			12	
13			13	
14			14	
15			15	
16			16	
17			17	
18			18	
19			19	
20			20	
30			30	
40			40	
50			50	
60			60	
70			70	
80			80	
90			90	
100			100	
110			110	
120			120	

Discharge rate 2

Date: Time:

Static W/L (m):

Time (min)	Drawdown s (m)	Yield (l/s)	Time (min)	Recovery s' (m)
1			1	
2			2	
3			3	
4			4	
5			5	
6			6	
7			7	
8			8	
9			9	
10			10	
11			11	
12			12	
13			13	
14			14	
15			15	
16			16	
17			17	
18			18	
19			19	
20			20	
30			30	
40			40	
50			50	
60			60	
70			70	
80			80	
90			90	
100			100	
110			110	
120			120	

Discharge rate 3

Date: Time:

Static W/L (m):

Time (min)	Drawdown s (m)	Yield (l/s)	Time (min)	Recovery s' (m)
1			1	
2			2	
3			3	
4			4	
5			5	
6			6	
7			7	
8			8	
9			9	
10			10	
11			11	
12			12	
13			13	
14			14	
15			15	
16			16	
17			17	
18			18	
19			19	
20			20	
30			30	
40			40	
50			50	
60			60	
70			70	
80			80	
90			90	
100			100	
110			110	
120			120	

Discharge rate 4

Date: Time:

Static W/L (m):

Time (min)	Drawdown s (m)	Yield (l/s)	Time (min)	Recovery s' (m)
1			1	
2			2	
3			3	
4			4	
5			5	
6			6	
7			7	
8			8	
9			9	
10			10	
11			11	
12			12	
13			13	
14			14	
15			15	
16			16	
17			17	
18			18	
19			19	
20			20	
30			30	
40			40	
50			50	
60			60	
70			70	
80			80	
90			90	
100			100	
110			110	
120			120	

Discharge rate 5

Date: Time:

Static W/L (m):

Time (min)	Drawdown s (m)	Yield (l/s)	Time (min)	Recovery s' (m)
1			1	
2			2	
3			3	
4			4	
5			5	
6			6	
7			7	
8			8	
9			9	
10			10	
11			11	
12			12	
13			13	
14			14	
15			15	
16			16	
17			17	
18			18	
19			19	
20			20	
30			30	
40			40	
50			50	
60			60	
70			70	
80			80	
90			90	
100			100	
110			110	
120			120	

Discharge rate 6

Date: Time:

Static W/L (m):

Time (min)	Drawdown s (m)	Yield (l/s)	Time (min)	Recovery s' (m)
1			1	
2			2	
3			3	
4			4	
5			5	
6			6	
7			7	
8			8	
9			9	
10			10	
11			11	
12			12	
13			13	
14			14	
15			15	
16			16	
17			17	
18			18	
19			19	
20			20	
30			30	
40			40	
50			50	
60			60	
70			70	
80			80	
90			90	
100			100	
110			110	
120			120	

CONSTANT DISCHARGE TEST DATA SHEET

General Information	Test Started	Remarks
Project No:	Date:	
Borehole No:	Time:	
Site Name:	**Test Ended**	
Farm Name:	Date:	
Latitude:	Time:	
Longitude:	Duration (min):	
Pumping Borehole Information	Planned Yield:	

Pumping Borehole Information	Observation Borehole Information		
	Observation BH 1	**Observation BH 2**	**Observation BH 3**
Depth of Pump (m):			
Collor Height (m):	Number:	Number:	Number:
BH diameter (m):	Distance:	Distance:	Distance:
Depth of BH (m):	Static W/L (m):	Static W/L (m):	Static W/L (m):
Static W/L (m):			

Time (min)	Drawdown s (m)	Yield (l/s)	Time (min)	Recovery s' (m)	Time (min)	Drawdown (m)	Time (min)	Drawdown (m)	Time (min)	Drawdown (m)
1			1		1		1		1	
2			2		2		2		2	
3			3		3		3		3	
4			4		4		4		4	
5			5		5		5		5	
6			6		6		6		6	
7			7		7		7		7	
8			8		8		8		8	
9			9		9		9		9	
10			10		10		10		10	
11			11		11		11		11	
12			12		12		12		12	
13			13		13		13		13	
14			14		14		14		14	
15			15		15		15		15	
16			16		16		16		16	
17			17		17		17		17	
18			18		18		18		18	
19			19		19		19		19	
20			20		20		20		20	
25			25		25		25		25	
30			30		30		30		30	
40			40		40		40		40	
50			50		50		50		50	
60			60		60		60		60	
75			75		75		75		75	
90			90		90		90		90	
120			120		120		120		120	
150			150		150		150		150	
180			180		180		180		180	
240			240		240		240		240	
300			300		300		300		300	
360			360		360		360		360	
420			420		420		420		420	
480			480		480		480		480	
600			600		600		600		600	
720			720		720		720		720	
900			900		900		900		900	
1080			1080		1080		1080		1080	
1440			1440		1440		1440		1440	
1800			1800		1800		1800		1800	
2160			2160		2160		2160		2160	
2520			2520		2520		2520		2520	
2880			2880		2880		2880		2880	
3600			3600		3600		3600		3600	
4320			4320		4320		4320		4320	

MULTI RATE TEST DATA SHEET

General Information	Pumping Borehole Information	Remarks
Project No:	Depth of Pump (m):	
Borehole No:	Collor Height (m):	
Site Name:	BH diameter (m):	
Farm Name:	Depth of BH (m):	

Discharge rate 1

Date: Time:

Static W/L (m):

Time (min)	Drawdown s (m)	Yield (l/s)	Time (min)	Recovery s' (m)
1			1	
2			2	
3			3	
4			4	
5			5	
6			6	
7			7	
8			8	
9			9	
10			10	
11			11	
12			12	
13			13	
14			14	
15			15	
16			16	
17			17	
18			18	
19			19	
20			20	
30			30	
40			40	
50			50	
60			60	
70			70	
80			80	
90			90	
100			100	
110			110	
120			120	

Discharge rate 2

Date: Time:

Static W/L (m):

Time (min)	Drawdown s (m)	Yield (l/s)	Time (min)	Recovery s' (m)
1			1	
2			2	
3			3	
4			4	
5			5	
6			6	
7			7	
8			8	
9			9	
10			10	
11			11	
12			12	
13			13	
14			14	
15			15	
16			16	
17			17	
18			18	
19			19	
20			20	
30			30	
40			40	
50			50	
60			60	
70			70	
80			80	
90			90	
100			100	
110			110	
120			120	

Discharge rate 3

Date: Time:

Static W/L (m):

Time (min)	Drawdown s (m)	Yield (l/s)	Time (min)	Recovery s' (m)
1			1	
2			2	
3			3	
4			4	
5			5	
6			6	
7			7	
8			8	
9			9	
10			10	
11			11	
12			12	
13			13	
14			14	
15			15	
16			16	
17			17	
18			18	
19			19	
20			20	
30			30	
40			40	
50			50	
60			60	
70			70	
80			80	
90			90	
100			100	
110			110	
120			120	

Discharge rate 4

Date: Time:

Static W/L (m):

Time (min)	Drawdown s (m)	Yield (l/s)	Time (min)	Recovery s' (m)
1			1	
2			2	
3			3	
4			4	
5			5	
6			6	
7			7	
8			8	
9			9	
10			10	
11			11	
12			12	
13			13	
14			14	
15			15	
16			16	
17			17	
18			18	
19			19	
20			20	
30			30	
40			40	
50			50	
60			60	
70			70	
80			80	
90			90	
100			100	
110			110	
120			120	

Discharge rate 5

Date: Time:

Static W/L (m):

Time (min)	Drawdown s (m)	Yield (l/s)	Time (min)	Recovery s' (m)
1			1	
2			2	
3			3	
4			4	
5			5	
6			6	
7			7	
8			8	
9			9	
10			10	
11			11	
12			12	
13			13	
14			14	
15			15	
16			16	
17			17	
18			18	
19			19	
20			20	
30			30	
40			40	
50			50	
60			60	
70			70	
80			80	
90			90	
100			100	
110			110	
120			120	

Discharge rate 6

Date: Time:

Static W/L (m):

Time (min)	Drawdown s (m)	Yield (l/s)	Time (min)	Recovery s' (m)
1			1	
2			2	
3			3	
4			4	
5			5	
6			6	
7			7	
8			8	
9			9	
10			10	
11			11	
12			12	
13			13	
14			14	
15			15	
16			16	
17			17	
18			18	
19			19	
20			20	
30			30	
40			40	
50			50	
60			60	
70			70	
80			80	
90			90	
100			100	
110			110	
120			120	