COMMISSIE VOOR HYDROLOGISCH ONDERZOEK TNO

COMMITTEE FOR HYDROLOGICAL RESEARCH TNO

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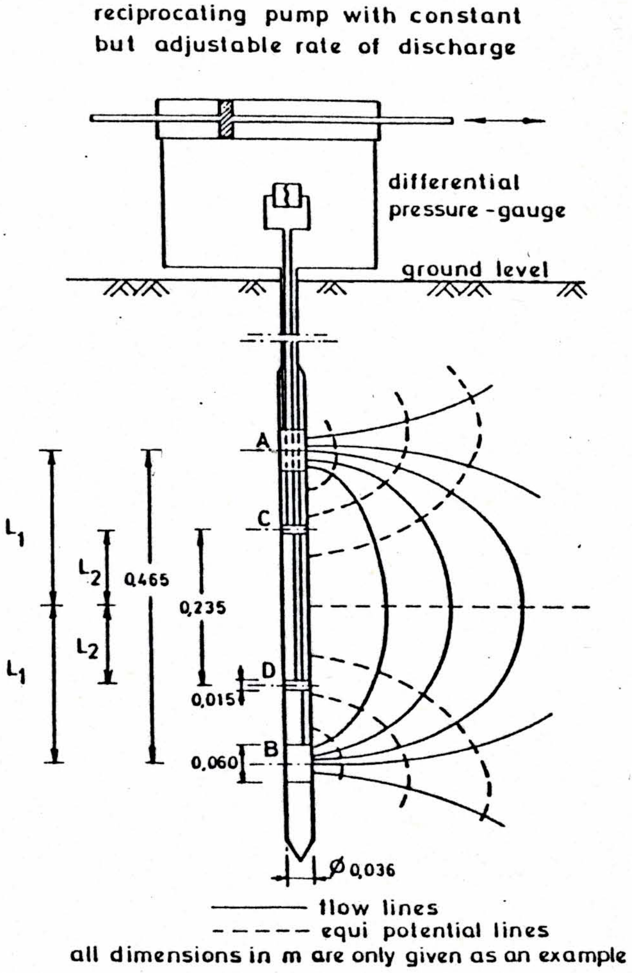

Fig. 1 Schematic drawing of dipole-probe

PREFACE

Inasmuch as the significance of thorough investigations of groundwater as an important natural resource has been generally recognized, several symposia have been held in recent years dealing with its quantitative and qualitative aspects. Much less attention has been paid to the very first phase of these investigations. The beginning is always the collection of data: which data, how much and by what method. It is here that a special problem is encountered. Indeed, there exists at present a great number of highly varied methods and related instruments, but it remains to be indicated which of the numerous methods and instruments are the most appropriate ones in each particular case. Decisive factors in this respect are the objectives of the investigations, the geological environment, the required accuracy and the funds available. It is precisely this problem on which this symposium has been focused. It has also been defined the three main themes of the symposium has been focused, with three main themes:

(i) criteria for optimal sampling strategies;
(ii) methods and techniques for the determination of geohydrological parameters and variables;
(iii) instrumentation for the acquisition and processing of geohydrological data, including accuracy, cost and management aspects.

I wish to thank gratefully TNO, IAHS and IAH for the support given to the activities of Unesco and of the secretariat of the Netherlands National Committee for the IHP.

A. Volker.
Chairman of the Netherlands'
National Committee for the
International Hydrological
Programme.

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THEME 1:

GENERAL INTRODUCTION

INTRODUCTION GENERALE

SIGNIFICANCE AND MAIN CHARACTERISTICS OF GROUNDWATER SYSTEMS

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Abstract

There are many areas all over the world, where groundwater resources provide the most important source of water supply. The interrelation between surface- and groundwaters is inevitable, their conjunctive use is, therefore, the most reasonable approach of the utilization of water. The planning and operation of groundwater exploitation requires the simulation of groundwater systems.
Independently of the fact, whether the simulation is carried out by hydrodynamic modelling of the systems or other approximations are applied to describe the regime of groundwaters, information is required on:
- the structure of the systems (their scale in space and time),
- the parameters characterizing the geometry, the hydraulic behaviour and the internal conditions of the systems, and
- the connection of the systems with the other parts of the hydrological cycle (boundary conditions).

The purpose of the symposium is to survey the recent development of the methods and instruments suitable for the determination of the parameters characterizing the groundwater systems. This introductory paper summarizes some principles, which should be considered and raises some questions, which should be answered when the most appropriate methods of groundwater studies are investigated.

1 Determination of available groundwater resources

1.1 The importance and the role of groundwater resources

A developing economy requires more and more water to be supplied to communities, industry and agriculture in every part of the world. At the same time the effluents released by the users are collected and discharged into the natural system, polluting in this way the recipient waters. This fact hampers the re-use and the multiple utilization of water resources.

There are many areas, where groundwater provides the most important source to meet the ever-increasing demand. A comprehensive quantitative and qualitative management of groundwater basins, therefore, is an inevitable requirement for the rational utilization of water resources.

The amount of water stored below the surface in the form of groundwater and soil moisture exceeds 20 percent of the total fresh water resources of the Earth (Figure 1). This water is available only to a limited extent for direct use, because exploitation of water stored in clay or at great depths is not yet economical. Neither is it permissible, to consume large quantities of this stored water, because it is one of the basic elements of the biosphere and some serious changes would be caused by its absence (e.g. the heat balance would be disturbed by the decrease of evaporation, or the vegetation would be destroyed by lowering the water table over large areas, where the contribution of shallow groundwater is important to meet the water demand of the plants).

The stored groundwater - a so-called static resource - has to be considered, therefore, as a reservoir, from which water may be exploited on a limited scale only. To determine the utilizable amount of groundwater resources in a basin, the expected natural recharge - the amount of water recharging continuously the aquifers by infiltrating rainwater - should be known. The over-exploitation of the groundwater (water mining) can also be avoided by recharging the aquifers artificially. In this case the natural storage capacity of the layers is utilized to change the distribution of runoff in time according to the demand of the users.

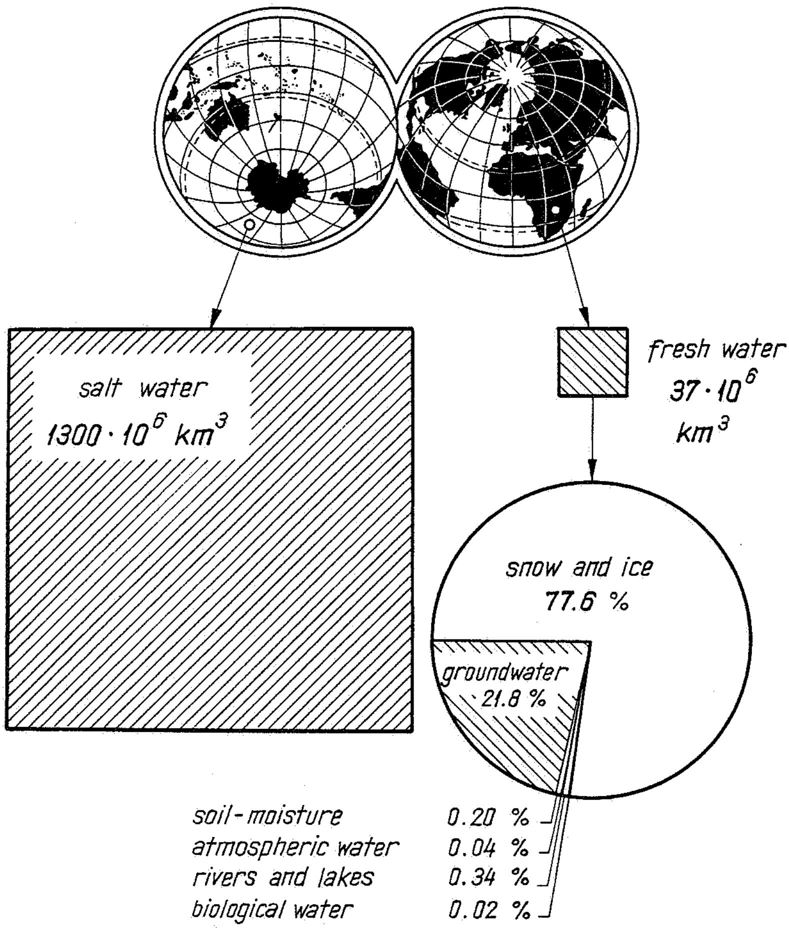

Figure 1 Distribution of fresh and salt water of the earth

1.2 Conjunctive use of surface- and groundwaters

Under natural conditions the groundwater aquifers are in balanced condition, i.e. the multi-annual averages of recharge and discharge respectively are equal. In a closed basin the infiltration of precipitation is the most important source of recharge, while the aquifers are drained by evapotranspiration and by the network of surface water courses. The infiltration is practically not modified by discharging the systems artificially. Hence, the percolation from the aquifers to the rivers and the water withdrawal caused by evaporation is always decreased by the exploitation of the groundwater to reach a new equilibrium.
A further amount of water becomes also available, when the water table is lowered by exploitation. The decrease of the stored amount of water due to pumping is composed of two parts, i.e.

- the pore-volume between original water table and the lowered surface is drained and
- the lowering of pressure causes consolidation.

The result of the latter is the decrease of both the pore-size and the volume of water stored in deeper strata (Figure 2). The emptied pores can be refilled by recharging the aquifer, but the water gained by consolidation can only be partly replenished since the layers are not completely elastic. The environmental consequence of the utilization of this resource is land subsidence due to the decrease of the bulk volume of the affected layers.

The close connection between surface- and groundwaters indicates that the latter cannot be regarded as an independent source providing the users with water. Indeed, utilization of groundwater actually means, that a considerable amount is taken from surface waters by decreasing their base flow. Only a relatively small amount of the water exploited from the aquifers is gained by lowering the water table. Yet the environmental impacts of its utilization should be considered when the use of this resource is planned (decrease of evapotranspiration, land subsidence).
Groundwater utilization has many advantages. This resource is available over large areas (especially in sedimentary basins) and not only along rivers and lakes. The aquifers may be utilized as natural reservoirs.

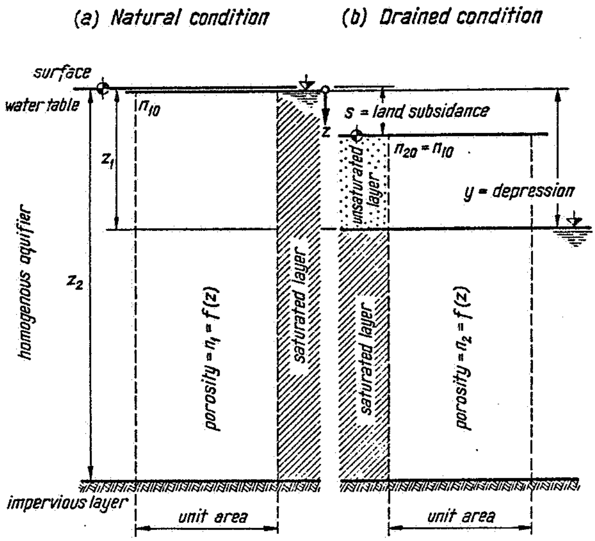

Figure 2 Groundwater abstraction with drawdown of the phreatic table and land subsidence

The quality of groundwater is generally suitable for human consumption without special treatment. It is necessary to consider however, that the available groundwater resources are limited and their exploitation modifies the water regime not only in the subsoil but also that of the surface waters. The environmental consequences of the expected changes have to be considered also when the utilization of groundwater is investigated.

There are two important conclusions, which can be drawn up from the basic aspects of groundwater exploitation listed in the previous paragraphs

a) The groundwater provides a well protected source, therefore, the water supply for direct human consumption should have preference among the various users. Since this source is limited and its utilization influences also the regime of surface waters, the use of the two sources should be investigated jointly. The final purpose of this investigation should be always the determination of the optimal combination of the use of surface- and groundwaters (conjunctive use).

b) The best allocation of the two types of water in case of conjunctive use can be determined only, when the expected modifications in the natural hydrological cycle and in the environment due to groundwater exploitation are predicted with sufficient accuracy, and the changes are carefully observed and evaluated during the operation. Both prediction and monitoring require the knowledge of the storage and transport processes in the groundwater system influenced by exploitation. The precise characterization of these systems is, therefore, an important prerequisite of the utilization of groundwater resources.

1.3 Simulation of groundwater systems

As a brief summary of the previous reasoning it can be stated, that the water stored in aquifers becomes available only by exploitation. The type, the location and even the way of operation of the discharging structures may influence the available amount of water. The other large group of the influencing factors is composed of the natural processes initiating and maintaining water exchange between the aquifer discharged artificially and its surrounding (surface waters, neighbouring aquifers, atmospheric waters through the soil moisture zone). The investigation of a groundwater system must not be limited, therefore, to an aquifer, but it should be always considered, that the water bearing layer is only a part of the whole system of the hydrological cycle. Hence, the groundwater systems - the size of which depends on the rate of exploitation and which may also change in time - must include always those effects which substitute the continuity of the hydrological cycle (natural boundary conditions) and the parameters describing the way of exploitation (the construction and the operation of the discharging devices as artificial boundary conditions).

It follows from this character of groundwater resources, that not only the methods applied in the hydrological investigation of groundwaters, but the basic concept used for the assessment of the available resources differs considerably from the determination of the available surface waters. The calculation of the latter is carried out independently of the demand and after knowing this parameter it is compared to the requirement in the form of a balance. On the contrary the assessment of groundwater resources should start by estimating the rate of exploitation and the consequences of the planned human activities should be predicted.

The upper limit of the available water can be determined only by the economic evaluation of the expected changes due to various exploitation. The damages caused by the modification of some environmental factors, or the investment and operational cost of measures needed to prevent the development of undesirable changes should be compared with the value of the utilized water in this economic studies.

The interpretation of the assessment of groundwater resources indicates clearly that there is only one way to meet the practical requirements. The average depression due to the operation of discharging devices planned to be constructed should be estimated in the vicinity of the exploitation and it should be investigated, what kind of changes are expected in the groundwater system due to this depression. The objective of such studies is always the determination of the hydraulic parameters of the modified system describing the new flow regime as well as the storage and transport of water within the system and around its boundaries. The changes are usually characterized by such parameters as the lowering of the water table or piezometric surface, the decrease of base-flow and that of the yield of springs, the amount of water transported by groundwater flow from neighbouring aquifers, basins or rivers towards the formation discharged by exploitation, etc.
Several different methods can be used to simulate the groundwater system and to predict the expected changes. It is the general opinion, that the regional hydrodynamic modelling of the groundwater systems is the most accurate and theoretically firmly based method which can meet all the practical requirements (Figure 3). The accuracy needed in such studies depends, however, on many factors (e.g. the space- en time-scale of the investigations, the amount of water exploited from the system related to the total available quantity, the knowledge of the hydraulic behaviour and the processes influencing the system, etc.) Hence in many cases somewhat simplified approaches are also acceptable.
There is, however, a common requirement, which should be fulfilled independently of the methods applied for the investigation i.e. the parameters characterizing the properties and the condition of the groundwater systems should be known. The purpose of this symposium is to investigate the methods and instrumentation suitable for the determination of these parameters.

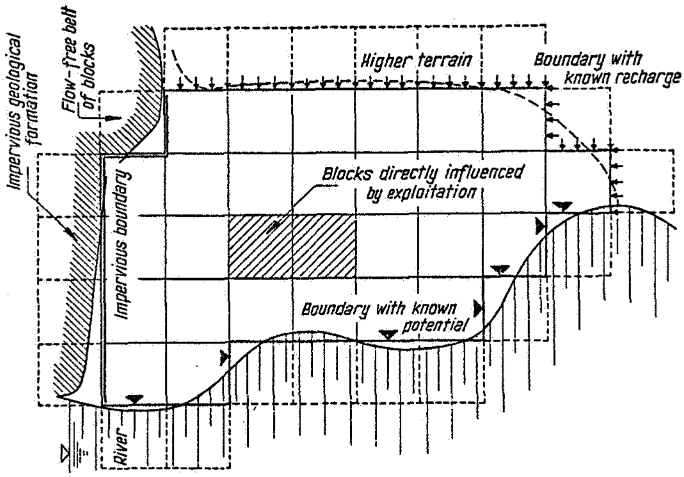

Figure 3 Regional hydrodynamic modelling of a groundwater system

2 Parameters to be determined for the characterization of groundwater systems

2.1 The influence of the use of computers on the development of groundwater studies

Seepage hydraulics has reached a new stage of its development in the last few decades. The application of numerical methods and the use of large computers became a generally accepted way of solving groundwater problems. Although the computer is essentially only a tool used for the rapid determination of the results of various relationships expressed in a mathematical form, it has basically modified both the possibilities of the investigation of groundwater systems and the direction of research in this field.

Previously there was a contradiction between the accurate characterization of the processes and the solution of the equations describing them. The first requirement needed the correct assessment of the influencing processes and their exact description. At the same time it was necessary to satisfy the second requirement too, by reducing the number of variables being taken into account and by simplifying the equations expressing the relationships between the variables, because, in practice, only the most simple analytic solutions were applicable. Hence the purpose of research was usually to find an acceptable compromise between the exactness and the mathematically accessible solution. In most cases there was no other choice than to neglect some of the influencing factors, although the basic relationships describing the physical processes were generally known since long time.

The use of numerical methods combined with their rapid handling facilitated by computers opens the way in practice to consider many variables and also to solve equations not being manageable analytically. Hence, by the new technique, the previous contradiction was eliminated (or it was decreased considerably). Relatively complicated relationships can be solved and more versatile processes can be taken into account than previously.
This analysis draws the attention to the need to find a new compromise, now between the exactness of the model and the accuracy of the parameters instead of the previous one, which aimed at the elimination of the contradiction that existed between the characterization of the processes and the solution of the equations describing them. The reliability of the simulation of a system - which is in general the objective of any study dealing with groundwater systems independently of both its direct purpose (the task of the investigation is research, planning or operation) and the method used to describe the processes (which is called a model without making difference between a set of differential equations and a practical rule of thumb) - depends on the model, on the way of its solution and on the parameters describing the behaviour of the system. It is quite evident, that the accuracy of an investigation is determined basically by the less precise factor. Hence, it is an important requirement, that the reliability of the components in a simulation should be developed in a consistent way.

The application of more exact kinematic equations to describe the physical processes and more precise numerical methods to solve the set of equations requires the concentration of research for the determination of the parameters characterizing the hydraulic behaviour of the water transporting system and the boundary conditions describing the interaction between the system and its surroundings. The task of this symposium is, therefore,

- to survey the methods suitable for the determination of these parameters and the instrumentation needed for their observation;
- to analyse the accuracy of parameters achieved by applying various devices; and
- to harmonize the required accuracy depending on the character of the system as well as on the time- and space-scale of the investigation with the reliability of the observations by indicating measuring techniques and instruments suited to the specific type of the study.

2.2 Parameters characterizing the properties and the condition of the groundwater systems

The basic information needed for a groundwater system and which is included in the models in the form of various parameters can be divided into three main groups

- the geometry of the system
- the hydraulic behaviour of the aquifers conveying the water and the aquitards separating the water transporting formations, and
- the instantaneous or predicted condition of the system influenced by the hydrodynamic processes prevailing there.

The geometry of the system is basically determined by the position of its boundaries and by its internal structure describing the subsequent layers, their thickness and extension. The various methods of exploration aiming at recognizing the stratigraphy and the geological structure of an area are the tools to determine this information. It is necessary to consider that - although the geometrical data are in most cases time-invariant - the external boundaries of the system are sometimes selected arbitrarily (the extension of the influenced field) instead of using an actual well defined surface (land surface or interface with an impervious bed). The position of such boundaries depends also on the development of the processes investigated and, therefore, it changes also in time (moving boundary problem). The delineation of recharge and drainage areas may assist in the determination of the extension of the system, and it provides, therefore, also information on the geometry of the flow field. In nature the water transporting formations are composed in most cases of several partly separated aquifers. The required details of stratigraphy depends also on the space-scale of the investigation. Hydrological observations (vertical pressure distribution, fluctuation of water table and piezometric level), the chemical composition and the temperature of water stored in different formations may help to determine the interconnected aquifers.

The two most important parameters characterizing the hydraulic behaviour of layers are hydraulic conductivity and storage capacity. In the case of water transporting formations the conductivity parallel to the bedding plan is usually more important than that in perpendicular direction. This value is replaced many times by the product of the conductivity and the thickness of the layer (transmissibility). When the aquitards are described the conductivity perpendicular to the bedding plane (or de leakage coefficient, i.e. thickness divided by permeability) is the parameter characterizing primarily the hydraulic behaviour. The determination of storage capacity depends on the fact whether a phreatic or a confined aquifer is investigated. In the first case the knowledge of the specific yield is sufficient, while in the second case the calculation of storage coefficient depending on the consolidation of the whole formation and on the compressibility of the water is needed. To simplify the investigation usually both hydraulic conductivity and storage coefficient are regarded as time-invariant parameters. There are cases, however, when their changes in time should be considered (deformable porous medium). To facilitate such calculation and in general to assist in the estimation of the two basic parameters the determination of some supplementary physical data of the layer (porosity, grain-size distribution, shape coefficient of the particles, fracture-size distribution, compressibility) is also advisable.

The condition inside the investigated area can be described by the spatial distribution of two parameters i.e. potential and seepage velocity (or flux). The potential (which is characterized by the potential head, i.e.

the sum of the vertical distance from the reference level and the pressure prevailing in the water related to the specific weight of the latter) can be determined as a scalar function depending on the space co-ordinates of the point investigated, while velocity can be characterized by a vector field. The parameters describing the instantaneous condition should be determined from direct observations. Since there exists a close relationship between potential and velocity (the latter is proportional to the gradient of the former), the measurement of one of them gives sufficient information on the other as well. Naturally the special behaviour of the transport process (e.g. non laminar character of flow, anisotropy or inhomogeneity of the layer) should be also considered, when such transformation is executed. The prediction of the expected changes in the potential and velocity fields due to the planned human activities is in most cases the task of groundwater investigation.

As a summary of the foregoing it can be stated that the topics of the symposium should include the analysis of the methods and instruments which are suitable to determine both the time-dependent and time-independent parameters of the groundwater systems. The determination of the parameters that will be discussed, can be grouped in the following way:
- geometry (stratigraphy and structural geology)
- hydraulic behaviour (hydraulic conductivity, storage coefficient and supplementary physical data)
- hydraulic conditions (distribution of potential and velocity).

2.3 Determination of boundary conditions

As it was already mentioned, the groundwater systems are only subsystems within the global system of the hydrological cycle. It is quite evident, that the continuity of the cycle should be replaced by appropriate boundary conditions. In fact the process of water transport was divided when the borders of the investigated groundwater system were selected. The character of the conditions to be taken into account depends on the type of the boundary.
At the interface of surface- and groundwaters the water exchange between the two types of water resources should be used as boundary condition.

This can be done either in the form of known flux through the boundary (when the discharging or the recharging effects of the surface water are estimated), or - and it is the more common and more natural way - the potential of the surface water is given around the perimeter of the field and the flux is calculated.
The processes draining or recharging the groundwater systems through the land surface are distributed over the basin. Infiltration originating from precipitation is a positive factor of the groundwater balance, while a part of evapotranspiration - that taking water from subsurface reservoirs (soil moisture and groundwater) - is a discharging effect. Although their specific value (the amount of water crossing a unit area during a time-unit) is usually small, the overall influence is considerable, because the effects have to be integrated over the horizontal surface of the system of a considerable areal extent.
The role of the hydrological processes developing on or over the land surface does not change when the investigation is limited to the saturated zone (the water table is used as the upper boundary of the system), only the numerical values of these boundary conditions have to be calculated in a different way. In this case the water exchange between groundwater and soil moisture (groundwater recharge) should be determined, considering the storage and transport in the soil moisture zone separated from the atmospheric hydrological processes.

There are always attempts to select the subterranean boundaries of the investigated system along the interface of the water transporting formation with an impervious layer. This very simple condition cannot be applied in every case. Sometimes it is not necessary to extend the investigation until a remote impervious boundary (limited space scale of the system), in other cases it should be considered, that the imperviousness of the bed bordering the water transporting formation is only a relative matter depending on its threshold gradient and leakage coefficient of as well as on the potential gradient developing through the underlying layer. Both in the case of a semi-impervious bordering bed (external boundary conditions) and when the aquifers are separated within a water transporting formation by aquitards (interval boundary conditions) the cross-flow should be considered also as a horizontally distributed process (similar to recharge). Its numerical value depends on the potential difference between the waters in adjacent water bearing layers.

Concentrated boundary conditions (not distributed over the area) should be considered not only around the perimeter of the systems, but also along structural lines (deeply eroded valleys, faulting zones) crossing the investigated area, where considerable recharge or discharge may influence the water balance. Sometimes the artificial effects are taken into account also as boundary conditions concentrated at given places of the system.

The numerical determination of the boundary conditions requires the combined investigation and observation of hydrological processes developing over, on and below the land surface. Since the influence of the accuracy of boundary conditions on the reliability of the hydrological characterization of a groundwater system may be at least so strong as that of parameters describing the geometry, the behaviour and the internal condition of the system, it would be advisable to analyse the determination of both parameters and boundary conditions in a combined form. Unfortunately the scope of this symposium did not make it possible to include the problems related to the boundary conditions of groundwater systems into the programme now. It is justified, however, to recommend for the international organizations sponsering this symposium:

- to convene a workshop in the forthcoming years for the detailed investigation of the boundary conditions of groundwater systems; and
- to ensure the interdisciplinary character of this meeting by inviting hydrologists and geologists as well as scientists dealing with theoretical research and experts engaged mostly in practical work.

3 New trends in hydrological research of groundwater

3.1 The random character of both the structure of systems and the influencing factors

Ten, fifteen years ago the development in hydrology was characterized by efforts to apply in our scientific field the methods of system analysis. Sharp debates were heard whether the models describing the systems should be based on some predetermined physical concept or on a statistical analysis of the time-series of observed data which better suits the random character of hydrological events. Since that time system analysis became generally accepted in hydrology and both the deterministic and the stochastic approaches are widely used depending not only on the character of the processes and on the types of data available, but also on the preference of the scientists engaged in the research. A new phase of development can be observed recently in many fields of hydrology trying to combine the advantages of both methods. The expected characteristics of a given phenomenon is determined from deterministic models utilizing the knowledge of the physical processes governing its development; the influence of random factors is analysed by stochastic models providing the variance of the same characteristics at different confidence levels. The hydrological investigation of groundwater systems provides a good example of this double approach. The basic seepage law (which may be linear or not depending on the character of flow) and the kinematic equations describing transport through and storage in porous media are well known and relatively easily applicable since the common application of numerical methods. They provide the deterministic part of the models. The stochastic character of the processes is caused by two different groups of influencing factors:

(i) the random variability of the water transporting channels building up the internal structure of porous media, and

(ii) the hydrological events influencing groundwater systems as boundary conditions and changing randomly in time. The consideration of their influence provides the stochastic elements of models.

The investigation of the influence of structural variability has started just recently. It was already proved that the local point values of hydraulic parameters (e.g. potential gradient, flux, concentration originating from point source pollution) have large scattering and their relative variance is closely related to the relative variance of pore size. This fact is especially important in cases, when the local value has a great practical role (e.g. dispersion of pollutants). It can be stated that also as a result of resent research, that usually the expected point values of the hydraulic parameters are equal to the parameters calculated by supposing homogeneous average structure, but that the flux is an exception. Its expected value is always smaller than that belonging to an average field, and depends not only on the variance of pore-size but also on the form of the zone of influence. This effect is incorporated, however, into the value of hydraulic conductivity and is often masked by the uncertainty of the latter.

The time series of time-dependent data observed in a groundwater system and characterizing the change of its conditions are the same as any other similar hydrological parameter. Their analysis does not differ, therefore, from the traditional methods of hydrology. Hence questions related to the random character of data and to be discussed in the framework of the symposium can be raised mainly in connection with the time-independent parameters:

- what is the probable measuring error of devices used or proposed to determine various parameters;
- how this error is related to the stochastic variation of the same parameter and how can the influence of the two random factors be distinguished;
- how many repetitions of measurements are needed to arrive at a reliable estimate of both the expected value and the variance of a given parameter and how this repetition can be executed under field conditions; and
- what is the range of scattering of a parameter in question, within which the variance can be regarded as the result of random variation of the structure and what is the limit above which the differences indicate the inhomogeneity of the system.

3.2 The applicability of continuum approach

The water transport through a conduit is usually described by two differential equations considering the conservation of energy and mass respectively. This set supplemented with the boundary conditions can be solved directly only in cases of very simple forms of the water conveying elements (straight tubes or prismatic channels). The complicated network composed of the pores or fractures of aquifers excludes, however, the direct application of the kinematic equations. The micro-structure of the channels is substituted, therefore, by a macroscopic characterization of the flow field characterizing the continuous field with average parameters (hydraulic conductivity and storage capacity). The kinematic equations are transformed to facilitate the calculation of the average flow characteristics, i.e. distribution of both potential (pressure) and seepage velocity (flux) within the whole field or at its special points or lines. This is done by considering the boundary conditions only at the perimeter of the field.

The seepage laws derived in this way which give the relationship between the gradient of potential and the seepage velocity can be applied only if the continuum approach is acceptable for the characterization of the field. It is known, that the numerical characteristics of any property of a porous medium depends to a large extent on the position of the point the close vicinity of which is investigated. The range of the probable scattering decreases, when the size of the investigated sample increases. A ceiling can be reached (representative elementary size or unit) above which the calculated parameter has only random variation and is independent of the position of the sample within the layer. The applicability of continuum approach requires, that the flow domain described as a continuous field should be always several times larger in each direction, than the representative elementary size.

Considering the interpretation of the representative elementary size, the type of the porous medium and the character of the process investigated three main cases can be mentioned where the continuum approach (and the seepage laws derived by supposing its validity) cannot be accepted and applied:

a) The field is smaller than several representative elementary units. The average flow characteristics cannot be calculated on the basis of a seepage law, but the water conveyance through individual channels should be investigated.
b) The character of the flow depends on the size of the channel e.g. when the difference between the area of the smallest and the largest pores is higher than one order of magnitude laminar, transitional or even turbulent flow may develop simultaneously in the same porous medium in various channels, or in unsaturated medium the small pores are saturated and there are only water films in the large pores). The frequency (or probability) distribution of the number of pores according

to their size should be known in these cases.

c) The characterization of the average of the investigated process does not give sufficient information, but the probable extremes of the point values should be known (e.g. the dispersion of pollutant in aquifers).

In each case the traditional methods applied in seepage hydraulics should be supplemented with the investigation of the structural variation of the water transporting channels in porous media. The analysis of the influence of the randomly changing structure on the hydraulic behaviour of the field is a recently developing topic of groundwater studies. Our knowledge is, however, very limited concerning the structural variability of aquifers. In the case of loose clastic sediments the grain-size distribution is generally used to characterize the structure of the layer instead of the pore-size distribution. This supplementation is acceptable only, when the pores are evenly distributed (e.g. in coarse-grained sediments). The aggregation of fine grains may create a secondary porosity and in this case the grain-size distribution does not adequately characterize the actual pores. The knowledge concerning the structure of fractures is even more limited. It would be advisable, therefore, to discuss the following problems:

- what kind of methods or instruments can be proposed to measure pore-size distribution in loose clastic sediments;
- is there any idea about the type of the probability distribution suitable to approximate the interrelation between the number of pores and their size; and
- how can the structure of fractured rocks be characterized for hydrogeological studies.

3.3 The propagation of pollutants in groundwater systems

Sofar, in this paper, the quantitative aspects of groundwater studies have mostly been analysed. It was mentioned that one of the advantages of utilizing groundwater is its better quality and its natural protection against pollution. The ever increasing amount of solid and liquid wastes deposited on and below the land surface, the increase of the pollution of surface waters being interconnected with groundwater reservoirs and the chemicals applied in agriculture and percolating through the soil moisture zone indicate the necessity to investigate the quality of subsurface waters more carefully. The theoretical research to characterize the propagation of pollutants in porous media, as well as the practical measures to be applied to increase the protection of aquifers against pollution and to observe the changes in quality, can be regarded also as a new trend of the hydrological investigation of groundwater systems. Among the theoretical problems the determination of the propagation time and the dispersion of pollutants can be mentioned as an area where research is needed. There are even efforts made to supplement the regional hydrodynamic models of groundwater systems with the simulation of chemical transport. When the effects of non-point source pollution (originating mostly from the fertilizers and other chemicals used in agriculture but including also e.g. the influence of acid rain on groundwater) are investigated the propagation of pollutants through an unsaturated layer should be determined. The change of the concentration due to processes other than dispersion (e.g. filtering, chemical decomposition, dissolving salts from the solid matrix, radioactive decay) are forming also very rapidly an important group of theoretical research developing.

The investigation of the filtering ability of natural strata leads from theoretical to practical research. It is necessary to know how the concentration of various chemicals decreases as a function of the length of propagation, the type of the layer and the biological processes which may develop in the layer (e.g. iron bacteriae). Considering these aspects and also the probable direction and velocity of flow developing due to exploitation, the aquifers can be classified according to their natural protection against pollution. Since this property is an important parameter of groundwater systems the methods and observations needed for its numerical characterization could also be discussed in the framework of this symposium.

There are, however, several further practical questions which could be raised in connection with the investigation of the qualitative problems of groundwater utilization, e.g.:

- what kind of qualitative parameters should be measured in a hydrological network serving the regular observation of groundwaters, in the framework of a groundwater exploration or as a part of the operation of water supply schemes;

- which devices are suitable for monitoring the data needed for the characterization of groundwater quality;

- what is the necessary density in space and time of sampling depending on the structure of the groundwater system and on the way of the utilization of the water.

4 Closing remarks

The purpose of the paper was to emphasize the importance of the groundwater resources in water management as an introduction to the symposium dealing with the methods and instruments applied for the observation of processes characterizing groundwater systems. Efforts were made at the same time to define the scope of the symposium by giving the interpretation of groundwater system, by summarizing the time-dependent and constant parameters to be determined, by indicating the interrelation between the accuracy of the parameters and the time- and space-scales of the investigations and finally by referring to the dependency of the methods applied for the exploration of the systems on the purpose of studies. References were also made to the new development of the research in the field of groundwater hydrology to indicate those areas where new types of data collection or the improvement of traditional methods are required.

It is hoped, that this introduction and some questions raised in connection with some special aspects of the characterization of groundwater systems will assist the development of a lively discussion aiming at the clarification of several problems and the indication of further research needs.

GOALS AND CRITERIA FOR THE DEVELOPMENT OF FIELDWORK PROGRAMMES

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Abstract

A framework for the planning of groundwater fieldwork programmes is described. Specific reference is made to the financial, technical, organisational and socio-environmental factors influencing programme design and execution.

Résumé

La structure d'ensemble ayant trait à la planification des programmes de travail sur le terrain est commentée en détail. Il est fait tout spécialement référence aux différents facteurs d'ordre financier, technique, organisationnel et socio-environnemental, lesquels ont leur rôle dans l'élaboration et la mise à exécution de ces programmes.

1 Introduction

It is instructive to begin a consideration of groundwater investigation planning from an historical point of view. It is recorded (Wilkinson, 1977) that in 1849 evidence was given in a court case in England regarding pollution of a domestic well that

> "the laws of the existence and progress of percolating water cannot be known or regulated. It rises to great

> heights and moves collateraly, by influences beyond our apprehension. These influences are so secret, changeable and uncontrollable that we cannot subject them to the regulations of law nor build them a system of rules."

Only seven years later Henri Darcy was to demonstrate the falseness of this statement and to provide the theoretical basis for the study of the flow of groundwater and its management as a resource. Despite the fact that Darcy's work was based on public water supply, the scientific study he started was slow to establish its practical relevance and for a long time a gulf persisted between practitioner and theoretician. Much of the rapid increase in municipal water supply in the latter part of the nineteenth century and the early years of this century was undertaken without any systematic measurement of the groundwater system that was being exploited. The rates of abstraction were usually modest in relation to the available resource, and thus the perturbation of the natural groundwater system was small, with only limited and local implications for other abstractors or the environment. Man's activity as a producer of waste was less, thus reducing the need to have regard to water quality constraints. There was also less public awareness and concern about adverse environmental effects, no doubt due to both the fact that the number of incidents were few and scattered, and because any disbenefits were weighed against the considerable social benefit of first-time water supplies. For all these reasons groundwater exploitation in developed countries has a long history of success, and a short experience of systematic field evaluation, data collection and monitoring.

The situation has changed rapidly in recent years. Worldwide the exploitation of groundwater resources is being undertaken on a hitherto unprecedented scale. In many regions groundwater resources are being used at rates close to or above the rate of natural replenishment. It is beyond question that development on this scale should only be undertaken with closest attention to hydrogeological controls and with the fullest possible understanding of the groundwater system in both quantity and quality terms.

There remains however something of a gulf between the theoretician and the practitioner which hinders the most efficient use of new developments in both measurement techniques and data analysis. This arises because in many cases new methods and models are not sufficiently well documented in terms of the limits and constraints upon their use with the result that the practitioner is not readily able to identify the right method or model for his particular investigation. These constraints may be due to the physical characteristics of the system under investigation. They may also arise from organisational factors such as practical limits on the type or amount of data which it is possible for an investigator to collect. This highlights the need for a balance to be struck between the data demands of an increasingly intensive management of groundwater resources and our ability to obtain, and continue to obtain, this data. We are faced with a resource allocation problem whose objective is the most cost-effective field programme to obtain the maximum amount of valid field data relevant to our planning and operational needs.

2 Defining information needs

The factors influencing the design of effective fieldwork programmes are illustrated in the relationship shown in Figure 1. The process begins with the definition of objectives. The objectives will vary widely depending on the type of investigation, as described below, but they are always necessary to ensure that the correct information needs are identified. Groundwater systems are complex and subject to local inhomogeneity, and even the best designed programmes may produce some anomalous results which do not contribute to an overall understanding of the problem. It is therefore important that additional redundancy is not built in because the objectives, and therefore the defined information needs, are imprecise.

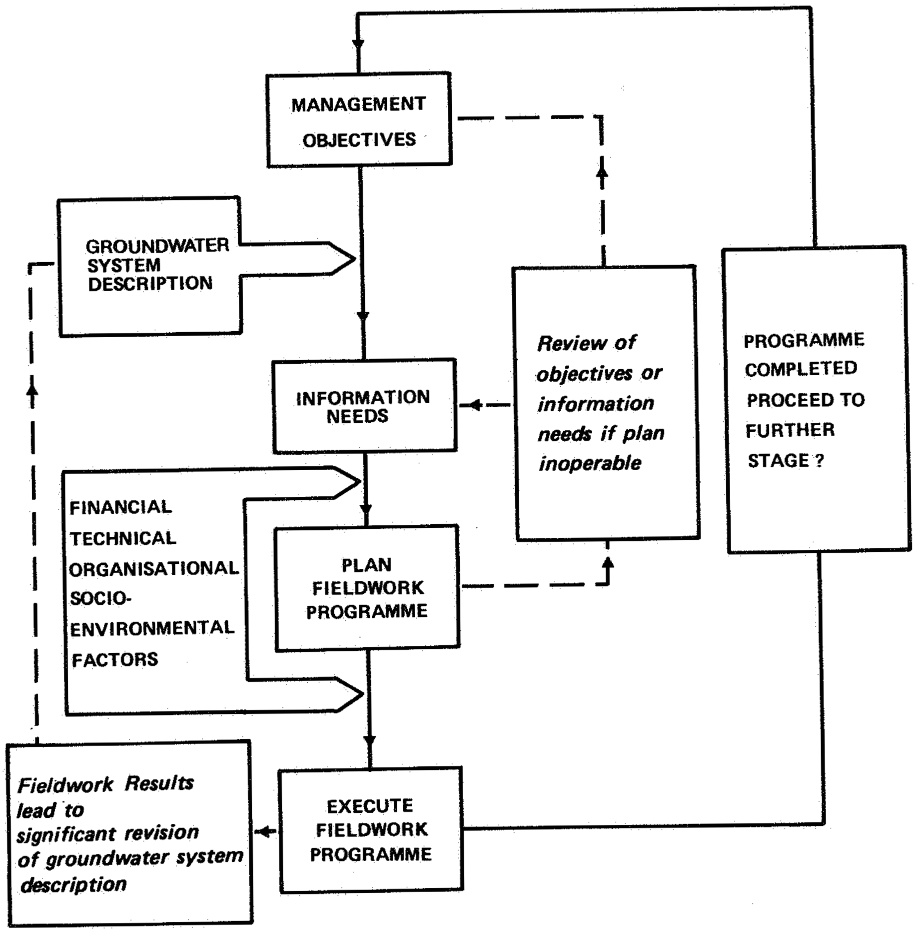

Figure 1 Steps in the development of fieldwork programmes

The information needs will also depend on the characteristics of the groundwater system under consideration. These characteristics need to be identified in terms of the physical, and perhaps also biological properties of the system and the way these are dependent upon changes in scale and time. Of particular importance is the adequate definition of the boundaries of the groundwater system, especially where there are links with other parts of the total hydrological system. Exploitation of groundwater resources can rarely be adequately considered without a knowledge of its potential impact on surface water resources and thus the information needs will often include data on surface water flow and spring discharges. Depending on the initial level of knowledge of the system it is likely that certain characteristics may not emerge until the investigation is in progress. It must be recognised, therefore, that the perception of information needs may change during the progress of a study, and thus require changes in the nature of the investigation programme.

3 Types of investigation

The structure shown in Figure 1 can be applied to many types of investigation, but it is convenient to regard them as falling into three categories. These categories are not exclusive and any one method of measurement or monitoring facility could be used in all three roles. The distinction comes not so much in the way the information is collected but in the way that it is used. The three categories may be briefly described as

- i) research orientated,
- ii) project orientated, or
- iii) operationally orientated

3.1 Research orientated investigations

These are specific field investigations often restricted to individual experimental sites and designed to improve understanding of basic physical or chemical processes in groundwater systems. They are distinguished by the fact that the investigations are not designed to provide information only on to the situation and location in which the measurements are made. The results are intended to have a wide application to groundwater management either directly, or through their use to validate general models of groundwater systems.

3.2 Project orientated investigations

This type of investigation is directly related to the planning of a groundwater project within the area in which the study is undertaken. The object of the investigation is to characterise the groundwater system and obtain the basic information with which to consider the feasibility of a project or to assess the relative merit of alternative strategies. It is very likely that the investigation results will be used to establish and calibrate a model of the system. The model will typically have the dual role of assisting in the understanding of how the system operates and providing an aid to scheme design. The investigation results will also be used to devise controls on the operation of the project which will form the basis of later monitoring.

3.3 Operationally orientated investigations

This type of fieldwork programme is directed towards monitoring the effects of what is happening in the catchment over a period of time. The information collected will often be linked in some way with resource management decisions. A typical example would be the control of the operation of water supply sources by reference to piezometric head or quality changes observed at monitoring points. In this case the monitoring is strictly operational, and is designed to ensure efficient management of the works. Other situations may be regulatory in nature, for example the surveillance of water quality around a landfill site as part of a waste disposal licence condition designed to enforce legal protection of other interests. In both examples the locations which are monitored, and the controls which are set, will have been determined from the results of an earlier project-orientated investigation. The structure shown in Figure 1 can therefore be thought of as iterative. In some situations a single investigation could develop from a research study, through a project-orientated pilot phase, to an operational phase. The three types of investigation may therefore be thought of as an evolution of a study with time. They may also be thought of as a series of scale changes, in that a research study may be confined to an individual location whereas the operational programme may occupy tens or hundreds of square kilometres. The investigator needs to be aware of the significance of the change of scale as it affects the identification and measurement of boundary conditions and the strategy for data collection. Changes in time and scale lead to new perceptions of information needs and therefore to a change in the content of the fieldwork programme.

4 Issues in programme design

The discussion so far has not considered the critical aspect in investigation design; the problem of translating information needs into a coherent and cost-effective fieldwork programme. Figure 1 identifies four factors which influence programme design and the following discussion deals with these in sequence, identifying some of the more important issues, the way they may be met and the areas where development of new techniques could be expected to improve future fieldwork programmes.

5 Financial factors

5.1 Set realistic budgets

There is very little information on the way in which budgets for investigation programmes are set or on any objective assessment of how objectives may be compared with costs. A number of possible approaches exist. The cost of an investigation programme can be assessed in relation to likely benefits. This has been discussed by Wilkinson and Edworthy (1981) in relation to groundwater quality networks, where the cost of the network is compared with the replacement cost of water supply installations which might otherwise be a risk. Statistical methods of determining the optimal groundwater observation programme based on this approach have been developed in Israel (Bachmar and Ben-Zvi, 1981). For investigation programmes which are likely to lead to a capital scheme such as a new water supply source the cost may be compared with the potential capital investment. Industrial enterprises expect to allow between 5 and 15% of the final investment costs as development expenditure, the wide range reflecting the level of technology and competitiveness of the market. Figures available in the United Kingdom suggest that investigation costs of groundwater schemes are about 10% of total capital cost. This is on the high side compared with other examples in the water industry, such as dam construction, but this is perhaps reasonable in view of the complexity and lack of homogeneity of groundwater systems. In the absence of more objective criteria the investigator must use professional judgement influenced by the standards adopted in similar situations in other areas. Reports of field investigations rarely give attention to these aspects and thus deny other workers the benefit of experience gained in programme design. Richards (1977) quotes average densities of monitoring networks in the U.K.

5.2 Stage the programme

Significant economies can be achieved by attention to the structure of the programme. Investigations carried out in stages, with an opportunity for data analysis between stages will have the benefit that the design of later stages can be based on results gained from the first stage. There may be small cost penalties because of repeat mobilising costs of contractors but these will normally be more than outweighed by the benefits. Where an investigation covers a large area, the work may be segmented by confining the detailed investigation to a limited, pilot area. This technique has been used for a number of United Kingdom studies. To be successful it is essential that the pilot area adequately reflects the conditions in the rest of the area under study, and to ensure that this is so an investigation, albeit at lesser intensity, is required over all of that area. The concept of staging may also be applied to recurring costs such as water quality analysis. Sampling opportunities, once missed, cannot be repeated but costs will be high if sampling is frequent and all the samples taken are presented for analysis. This problem can be met by storing a proportion of the samples and analysing them in a second stage, if relevant trends are evident.

5.3 Examine high cost activities

Examination of project budgets normally identifies borehole drilling as the most expensive single element, and any means of reducing this without unreasonably compromising on the information needs will provide additional resources for other parts of the programme. Full use should be made of existing boreholes and it will often be worthwhile rehabilitating disused boreholes to contribute to the investigation. Where there is uncertainty as to the most appropriate location for test boreholes this can often be resolved by using surface geophysical surveys to identify lithological, structural or water quality variations. One redundant borehole saved will normally pay for a substantial surface geophysical survey.

6 Technical factors

A survey of the methods available for determining the various parameters which define groundwater systems is not appropriate here. Common to all investigations however are a number of basic principles which, if followed, should help to improve the efficiency of the study and the quality of the result.

6.1 Strategy for monitoring of groundwater systems

The majority of measurements made on a groundwater system are at individual point sources. They therefore suffer from the same limitations that exist for all sampling systems. The distribution and density of sampling points must be influenced by the nature of the dynamics of the groundwater system. A tractable analysis of groundwater flow depends upon a scale of measurement for which the porous medium is sensibly homogeneous and can be seen as a continuum. Aquifers exhibit inhomogeneities at a variety of scales from the microscale dependent on grain fabric to the macroscale dependent on fracture density and both may occur within the same system. The former may be very important in a study of pollutant migration but can be ignored for most purposes in a groundwater resource study where larger scale variations dominate. The concept of representative elemental volume (Long et al, 1982) to define the volume of aquifer which is homogeneous for the relevant purpose can be used in the formulation of a monitoring strategy. Investigation techniques which are not point dependent and give a measure, even if only qualitative, of the influence of the scale factor in characterising the groundwater system are particularly valuable when used early in an investigation programme. Examples of these are aerial remote sensing surveys and, for the depth component, wireline geophysical logging.

6.2 Match technique to need

The choice of technique for a given measurement needs to be given close attention, and the wrong choice may affect the scientific validity or the cost-effectiveness of the exercise. Simple examples of inappropriate techniques are the use of laboratory measurements of dispersivity in regional quality models, the use of destructive sampling methods for variables expected to change with time and water quality sampling in layered aquifers from single unlined boreholes. In these cases consideration of the effects of space, time and system boundaries respectively would have indicated the need for other methods.

A further factor is the need to appreciate the accuracy requirements for different purposes. This can affect not only the scope of the programme but also the type of instrumentation which is used. An example is the determination of the hydraulic conductivity. The simplest method of obtaining the hydraulic conductivity data is probably by using the various empirical relationships with sediment characteristics, such as that due to Hazen. Grain size analyses are easily obtained, and in many situations may be already available from other sources such as engineering site investigations. The Hazen relationship, or the more complex developments of it, cannot provide an accurate estimate of hydraulic conductivity. However for sediments with a common depositional environment and where a limited amount of check data can be obtained by more sophisticated means, it can provide reasonable estimates. For many applications, for example regional modelling or obtaining estimates of travel times to assist in the design of a monitoring network, these data may be of adequate accuracy and are obtained at much less cost than by other means. The most frequently used method of determining hydraulic conductivity is by purpose-designed pumping tests. Freeze and Cherry (1979) have expressed the view that the pumping test approach is often used where it is not justifiable on cost grounds. They recommend reliance on single well short term injection and bailing tests for most applications (for example geotechnical applications, contaminant studies and regional flow analysis) and advise a full scale pumping test only where there is a likelihood that the expensive test facilities can later be used operationally. In contrast pumping tests, although expensive, provide information other than on hydraulic conductivity, some of it which can not be obtained by any other means. In this example, as with many other hydrogeological parameters, it is necessary to choose a method where the scope and accuracy are appropriate to the need.

6.3 Integrate quality and quantity studies

There is a danger that investigations can be identified as groundwater development problems or groundwater contamination problems with the result that quality or quantity aspects unreasonably dominate the programme. For a full description to be made of the groundwater system it is necessary to look equally at quality and quantity, preferably using common sampling points. Ideally the aquifer should be characterised for piezometric head, hydraulic properties and quality throughout its full thickness and the long term monitoring facilities planned with these data. The most common methods of groundwater quality sampling, from pumped samples and by bailer or depth samples are inadequate to define groundwater quality changes with depth (Wilkinson and Edworthy, 1981). Permanent *in situ* samplers can be used to take samples from individual layers but the borehole then becomes committed to their use. These constraints are beginning to be overcome now that light, small diameter (down to 50 mm) sampling pumps are available that can be used without heavy lifting equipment and thus can be operated by a field scientific team. Pumping between double packers at successive depth horizons means that both hydraulic conductivity and groundwater quality determinations can be made from reliably identified horizons. Permanent groundwater chemistry and piezometric measuring points can be established at horizons of interest by means of nested open piezometers, thus ensuring common measurement points for quality and quantity.

6.4 Anticipate and review

A major constraint in many investigations is time, which prevents adequate base line data from being collected and means that conclusions often have to be drawn without the benefit of information on long term changes affecting the groundwater system. Attempts should be made to anticipate the study and to establish a few monitoring points early on so that the investigation data can be viewed against long term trends. Equally it is important to review all data once collected to ensure that unnecessary data collection does not continue.

7 Organisational factors

A high, and often ignored cost, of field investigation programmes is the operational cost. A large part of this cost is in manpower time. Any methods which can be used to improve the productivity of manpower, particularly if at the same time they can reduce human error should be exploited. This is an area where significant developments can be expected in groundwater investigations in the next few years.

7.1 Execution of field work programmes

The method of execution of field work programmes needs to be determined at an early stage. Although major work, like borehole drilling, will normally be done by specialist contractors, the option exists for much of the specialist field work in an investigation programme to be done either by the members of the investigation team or by a service company. In the oil industry the emphasis is very much towards the use of service companies. In the water industry, especially now that much of the necessary equipment is available in a robust form, not needing special facilities to handle or install it, a different trend has developed. Many agencies now perform most, if not all, of the geophysical logging, closed circuit television and specialist borehole sampling surveys. With the advent of easy to handle packer systems and sampling pumps the range of this work can be expected to extend. The advantages of this practice are seen both in reduced costs and increased flexibility. Work programmes can be easily modified in the light of results and can be mobilised quickly to fit in with the progress of drilling work to minimise unproductive drilling time. This argument is less compelling for reconaissance type tests, for example surface geophysical surveys, which normally take place at the beginning of a programme and do not link with other parts of the work. In this situation, and also for rarely used specialist skills, external contractors may provide the best service.

7.2 Data capture

There are a number of aspects of electronic data capture which can be expected to aid hydrogeological investigations in the future. The advent of the 'intelligent' recorder means that it is possible to depart from the collection of uniform serial time data and either move to non-uniform time-based data or to event recording (Walker, 1982). Both of these will reduce the volume and increase the value of data collected. The former has obvious application to pumping tests where data is required on an expanding time base. The second has a application to a number of situations when trigger information is required such as movement of saline interfaces or tracer breakthrough or merely to reduce data volume when range of fluctuation is small. A related development is the use of programmable calculators and micro-computers to perform data processing and preliminary analysis of data at a pumping test, with multichannel loggers handling data input from a number of test boreholes. This both reduces the manpower involvement necessary and gives a quicker interpretable data return which enables the programme of pumping to be modified in the light of results.

Where for various reasons data are best collected by field visit electronic data capture can still play a part. The use of a microprocessor based 'electronic notepad', such as are now commonplace in supermarket stock control, can be used to prompt the operator to collect the right data, provide immediate quality control by comparing with previous readings at the same site and provide an error free means of data transfer to computer-based archive. The major limitation on the application of these techniques is that developments in data storage and recording have outstripped developments in sensors, particularly of water level. It has proved cumbersome to take electrical signals from float systems up to now. Floats cannot reliably be used in very small boreholes and pressure transducers suffer from limitations of drift, range and precision. This is an area where there is scope for development work to enable new technology to be exploited.

8 Socio-Environmental factors

8.1 Environmental effects on the groundwater system

Changes in the environment can affect both the quality and quantity of the groundwater resource and any study directed at long term development programmes must take account of future trends. Problems of point source pollution are now met in most countries by the various national systems for groundwater protection but diffuse pollution is less well regulated. Systems with thick unsaturated zones may pose a problem of 'latent' pollution, irreversible but not yet manifest in the saturated system. This can be assessed by special unsaturated zone monitoring. Land use changes, changes in surface drainage and mineral workings can all reduce the quantity of recharge, although rarely to a significant degree. Where this is a matter of concern appropriate investigation is necessary.

8.2 Effects of groundwater development on the environment

This is increasingly becoming a significant aspect of many groundwater investigations which, before they can lead to a groundwater exploitation programme, will have to provide information to satisfy some form of public scrutiny. This development is most advanced in the United States where groundwater development programmes fall within the scope of the legislation requiring environmental impact statements. Although less formalised, procedures in many European countries have a similar effect and the issue is currently under study within the European Economic Community. The environmental effects of groundwater development are normally understood to embrace the following:

a) diminution of water levels in private, generally shallow, boreholes and wells;
b) change in water quality of private sources by change in groundwater flow patterns or by recharge of different quality water;
c) effects upon stream flows to the detriment of water supply, fisheries and general amenity;
d) effects upon flows from springs and on the water levels of lakes and ponds;
e) effects upon wetlands of native conservation value;
f) effect on the moisture content of the soil and on natural vegetation and crops dependent on it;
g) where discharge of groundwater is made to augment flows, the effects of those discharges on the ecology and fisheries of the receiving streams;

h) settlement of land due to dewatering of unconsolidated strata.

Many of these features are subject to considerable natural change under influences unrelated to the exploitation of groundwater resources. In order to identify those changes which may be due to the effects of abstraction it is necessary to have reliable base line data and thus the sooner that any environmental implications are perceived and monitoring undertaken the better . The types of monitoring will vary widely and require a variety of specialist skills to carry out. Modelling systems are available to deal with many of these problems and data collection will normally be best directed towards information for model calibration with aquifer water level measurements always having a dominant role.

The investigator should have regard to likely public reaction and where the issue is likely to be the cause of public concern it may be wise to take account of it in investigation design even if an objective assessment shows the issue to be of minimal significance. The characteristics of groundwater systems are not widely understood by the general public and an even limited amount of field data is valuable if there is to be public debate.

9 Conclusion

The importance of well-conceived and practically achievable field investigation programmes, although not much in evidence in the early development of groundwater resources, is now generally recognised. A common framework can be applied to investigations, whether they be for research, project planning or operational monitoring purposes. Many of the methods used are long-established and well-proven but technological changes, particularly in the field of data collection and handling, present new opportunities which should be pursued. It is important that fieldwork programmes are planned and executed with the information needs clearly in mind and in recognition of the limits placed on methods of analysis by the availability and reliability of the data collected. The cause of better investigation design would be well served by better documentation of the benefits and limits upon various methods, critical assessments of the various techniques available for different parameters and more published information on the criteria and costs of measurement programmes in case studies.

References

Bachmat, Y. and M. Ben-Zvi, 1981. Optimisation of a groundwater quality sampling programme. In: Quality of Groundwater, W. van Duijvenbooden, P. Glasbergen and H. van Lelyveld (Eds.), Elsevier, Amsterdam, pp. 607-625.

Freeze, R.A. and J.A. Cherry, 1979. Groundwater. Prentice-Hall Inc, New Yersey, 604pp.

Long, J.C.S., J.S. Remer, C.R. Wilson, and P.A. Witherspoon, 1982. Porous media equivalents for networks of discontinuous fractures. Wat. Res. Res., 18, 645-658.

Richards, H.J., 1977. Groundwater Quality Networks. In: Groundwater Quality - Measurement, Prediction and Protection, Water Research Centre, Medmenham, England, pp.132-206.

Walker, S.T., 1982. Hydrometric data capture using intelligent solid state logging systems. In: Advances in Hydrometry (Proc. Exeter Symposium, July 1982). IAHS Publ. No. 134, pp. 229-237.

Wilkinson, W.B., 1977. Estimation of groundwater resources. In: Irrigation development planning. J.R. Rydzewski (Ed.) Southampton University, Southampton, UK. pp 35-62.

Wilkinson, W.B. and K.J. Edworthy, 1981. Groundwater quality monitoring systems - money wasted? In: Quality of Groundwater, W. van Duijvenbooden, P. Glasbergen and H. van Lelyveld (Eds.), Elsevier, Amsterdam, pp. 629-642.

THEME 2:

CRITERIA FOR OPTIMAL SAMPLING STRATEGIES

CRITERES RELATIFS A DES STRATEGIES OPTIMALES D'ECHANTILLONNAGE

GROUNDWATER QUALITY STRATIFICATION -
ITS RELEVANCE TO SAMPLING STRATEGY

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Abstract

In many groundwater systems the adequate characterisation of groundwater quality, or chemistry, cannot only involve sampling to define areal and temporal variations. At any given time, variations in groundwater quality, often on a grand scale, are likely to be present with depth in the saturated zone, particularly in unconfined aquifers. This groundwater stratification will be controlled by, and will reflect, the present and historic groundwater flow regimes, both natural and artificially induced.

This topic will be illustrated with results from several detailed studies of diffuse groundwater pollution in the unconfined zones of major British aquifers. A research, rather than routine, level of investigation is giving a clearer insight into the distribution of solutes in these groundwater systems and posing numerous questions about the adequacy of traditional approaches to groundwater quality monitoring.

1 Introduction

The unconfined zones of the Chalk and Triassic Sandstone aquifers are important sources of public water supply and yet are particularly vulnerable to diffuse pollution originating at the surface in the aquifer intake/recharge areas. Such pollution is increasing and arises from numerous processes, in particular, modern farming practices (Foster et al.,1982) and fallout of industrial airborne pollutants.

In-depth research has been carried out in several catchments and the results from three are presented to illustrate the occurrence of groundwater stratification. While extensive data has also been collected on the unsaturated zone, the present discussion will be restricted to the saturated zone. In each catchment an integrated and phased research programme including centrifugal extraction of interstitial water from cored rock samples (Edmunds and Bath, 1976), pumped sampling from purpose-designed observation boreholes and borehole depth sampling controlled by flow logging (Tate et al., 1970; Foster and Robertson, 1977), has been undertaken.

2 Research catchments

2.1 Triassic Sandstone of South Yorkshire (Carlton)

In this research catchment (Figure 1a), low permeability Drift covers and confines the aquifer except for a limited area of sandstone outcrop. In this latter area, a major public-supply source has been abstracting groundwater since 1968, from two fully-penetrating, adjacent, production boreholes. The sandstone has high porosity. Groundwater movement is dominated by intergranular flow although occasional fissures exert an influence. Natural hydraulic gradients are very low, such that most flow is induced towards production boreholes.

Soon after pumping commenced, the concentration of solutes in the pumped discharge increased; notably, nitrate rose from 2 mg N/l to 12 mg N/l. Pore-water extracted from several cored boreholes, drilled during 1979-1981, in the outcrop area show a marked vertical stratification of water quality in the saturated zone. The pore-water chemistry is illustrated by the profiles of sulphate and nitrate (Figure 2), although the profiles of calcium, magnesium, chloride and various minor elements have a similar form. Other ions, such as bicarbonate, iron and manganese, display an independent distribution. The same is true of vertically stratified groundwater in other research catchments. At Carlton, very high solute concentrations occur at shallow depth and elevated levels of certain solutes and post-1953 pore-water tritium extend to at least 120m. Pumped samples from different depths within the aquifer broadly

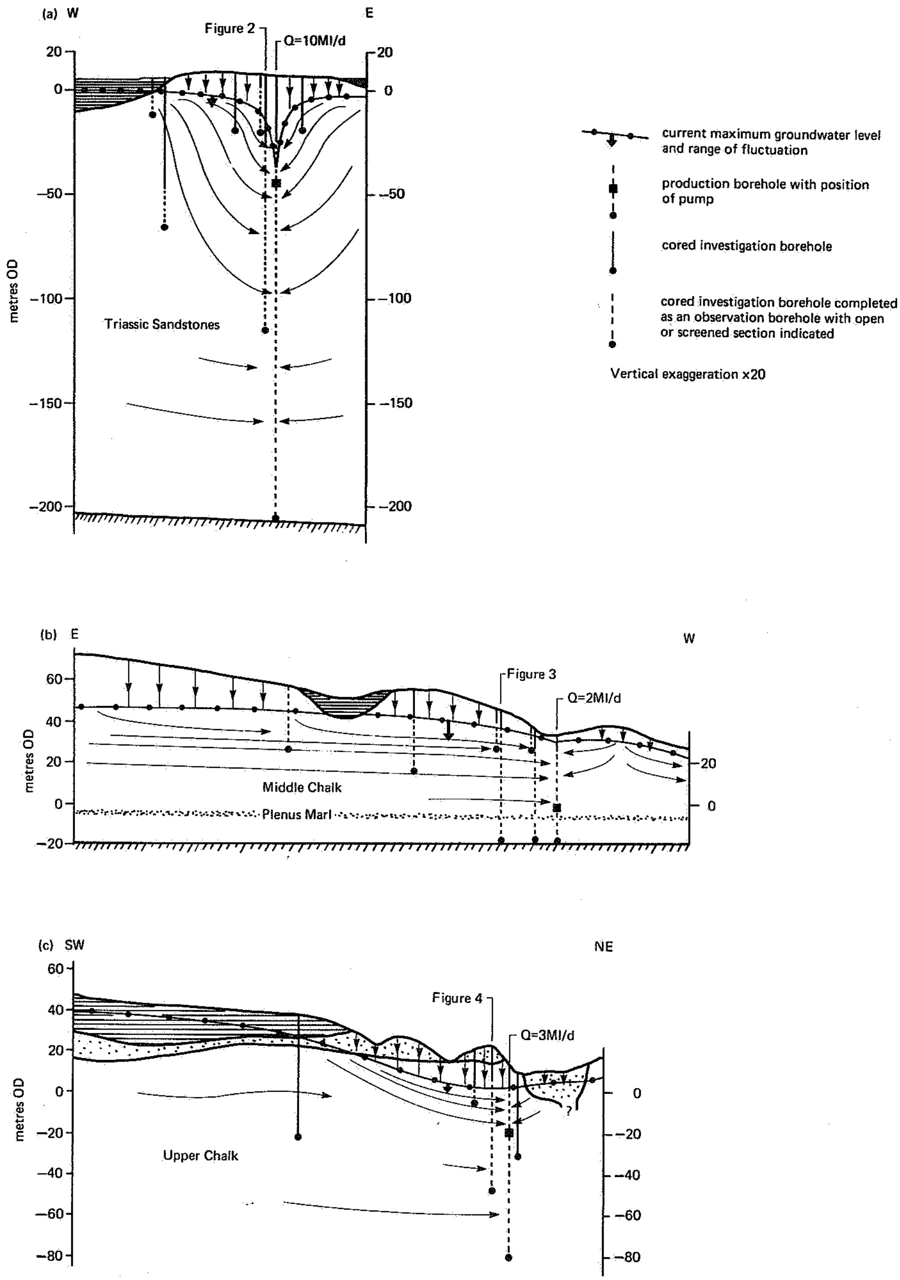

Figure 1 Diagrammatic hydrogeological sections of (a) Triassic Sand-Stone of south Yorkshire, (b) Chalk of west Norfolk and (c) Chalk of central Norfolk

demonstrate the vertical variation in groundwater chemistry (Figure 2).

Depth samples taken in the production boreholes under both static and dynamic conditions provide a complex and very variable set of results. From these it can be deduced that there is a degree of vertical variability within the aquifer but they give no indication of the form or magnitude of the stratification.

The deep penetration of modern recharge at Carlton and the vertical stratification of groundwater quality appear to result, primarily, from the construction of the production boreholes and their irregular and intermittent pumping regime. Pumping from deep boreholes has increased the vertical component of flow whilst the lateral throughflow has been

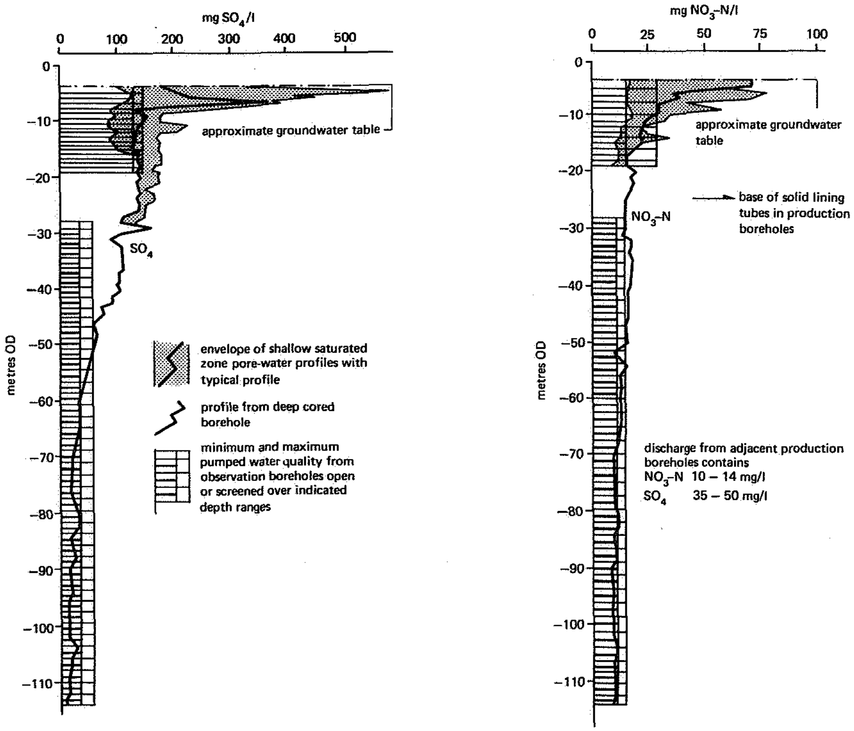

Figure 2 Chemical profiles of saturated zone pore-waters from the outcrop of the Triassic Sandstone in south Yorkshire and composition of pumped samples from observation boreholes

concentrated in the upper levels of the aquifer by preferentially high horizontal permeabilities. Diffuse pollutants entering the aquifer by direct infiltration through the outcrop of the sandstone move relatively rapidly through the system at shallow depths where the influence of recently increasing diffuse pollution is most evident. The induced groundwater flow regime is such that some of the water that penetrates to greater depths has a very much longer residence time in the aquifer and reflects diffuse pollution to a lesser degree.

2.2 Chalk or West Norfolk (Bircham)

This small groundwater catchment, situated on a subdued Chalk escarpment, supports a smaller public supply source (Figure 1b). The unconfined Chalk aquifer, deeply weathered at the surface, is structurally simple, although locally covered by a variable suite of permeable Drift deposits. Chalk groundwater flow is through fissures which are particularly well developed in the zone of water-table fluctuation and at the top of the permanently saturated zone. This high transmissivity layer is some 20 m thick and carries the bulk of the flow to the pumping station.

The land of the catchment is predominently used for arable farming and this is reflected in the groundwater chemistry. Surface derived, diffuse, pollutants in the pore-water (Figure 3a) exhibit peak concentrations in the zone of water-table fluctuation and relatively high concentrations through a considerable depth of the saturated zone but are much attenuated below a low permeability marl band. A long-term programme of sampling (depth and pumped) in open observation boreholes completed over restricted depth ranges clearly demonstrates a vertical stratification of mobile (fissure) water quality also and shows that the pumped supply has a composition between the extremes of the upper and lower levels in the aquifer (Figure 3b).

2.3 Chalk of Central Norfolk (Colney)

The Colney catchment, although on the Chalk dip slope, has a ground-

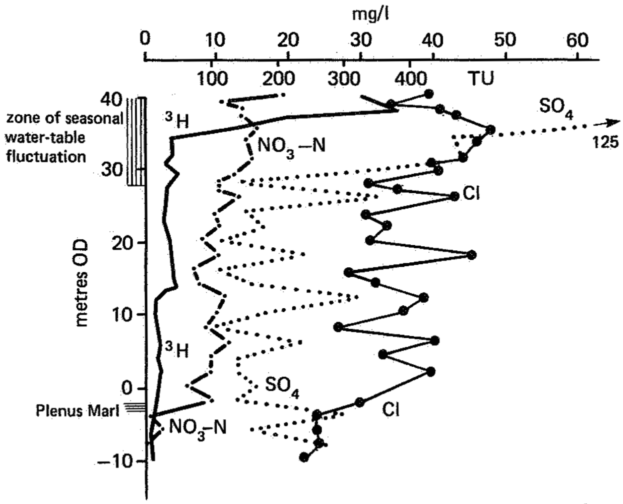

Figure 3a Chemical and isotopic profiles of saturated zone pore-waters from the unconfined Chalk aquifer in west Norfolk

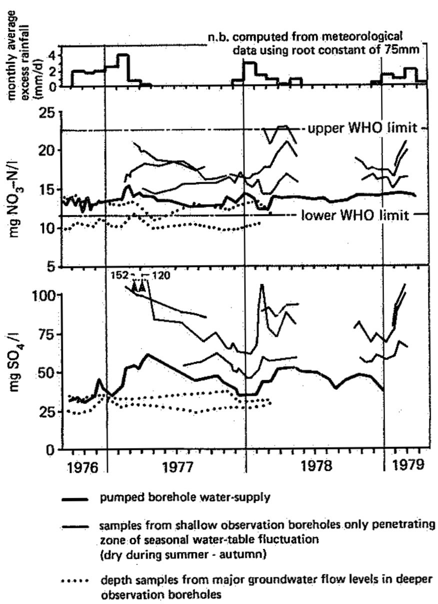

Figure 3b Variation of NO_3-N and SO_4 concentrations of Chalk groundwater with depth and time in west Norfolk research catchment

water flow regime in many ways similar to that at Bircham. The transmissivity of the Chalk is very high and, again, the most important fissure flows are in the upper 20 m of the saturated zone. The higher ground of the catchment, however, has a thick cover of Boulder Clay which restricts or eliminates infiltration(Figure 1c). Consequently, the production borehole derives the majority of its supply by interception of rapid groundwater flow in the high transmissivity zone which is recharged by infiltrating rainfall in an outcrop area largely dedicated to arable agriculture. A smaller component of the supply is groundwater of different quality which originates in the riparian zone to the north of the production borehole.

Pore-water quality varies widely, both areally and vertically. A cored borehole south of the pumping station revealed a marked interface in the pore-water chemistry, illustrated by the pore-water profiles of nitrate and sulphate (Figure 4). Above the interface, solutes in the fissure water have diffused into the pore-water creating a zone in which pore-water and fissure water alike are in equilibrium with solute inputs from modern recharge. The dominance of shallow horizontal flow towards the pumping station has generally restricted penetration of diffuse pollutants below the interface, the position of which must be related to a combination of the stratification of permeability in the aquifer and the pump setting in the production borehole.

Depth sampling in conjunction with borehole flow and quality logging has demonstrated that some 85% of the supply enters the production borehole above the level of the pump and contains recently derived diffuse pollutants and large amounts of dissolved oxygen. The remainder of the supply is from an inflow zone at much greater depth which, by contrast, is depleted in oxygen and contains low levels of pollutants such as nitrate. This groundwater probably represents a regional flow of much longer residence time in the aquifer.

3 Limitations of traditional groundwater quality monitoring approach

There are several different approaches to routine groundwater quality

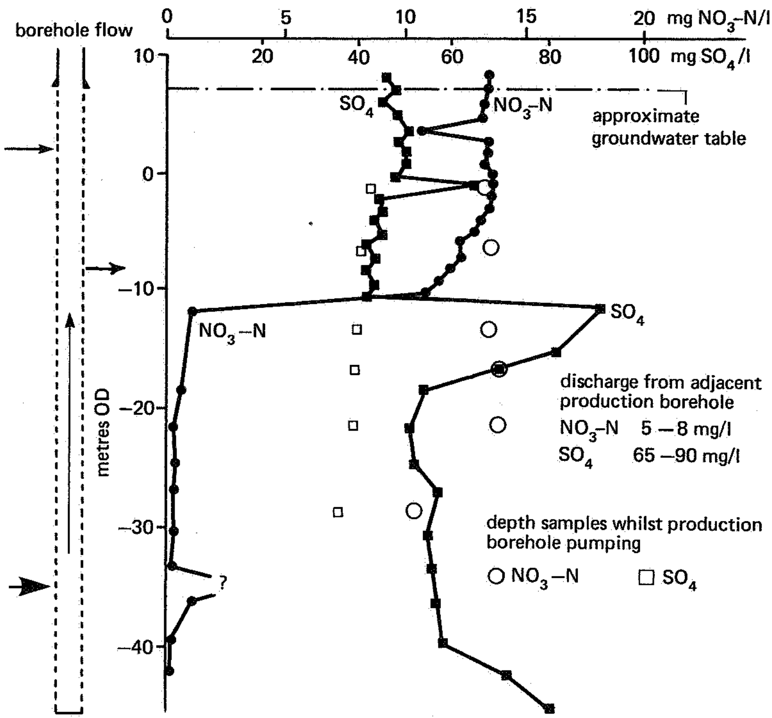

Figure 4 Saturated zone pore-water chemical profiles from the unconfined Chalk aquifer in central Norfolk and composition of depth samples from the same borehole

monitoring but in Britain it is usual for a considerable reliance to be placed on discharge samples from fully-penetrating pumping boreholes, supplemented by depth sampling in open observation boreholes. The limitations of this approach are often recognised and where possible, depth sampling is controlled by borehole flow logging. These traditional methods of groundwater sampling are reviewed.

3.1 Pumped samples

In all three research catchments the discharge from the production boreholes is of better quality than groundwater which is known to exist to variable depths, over large areas of the surrounding aquifer. Major abstraction boreholes such as these, and observation boreholes drilled primarily for groundwater level measurements, typically penetrate a considerable depth of the aquifer and are open or screened throughout most of their length. Pumped samples from such boreholes will have a composition which is some mixture of the groundwater present in the aquifer over that depth interval, thus masking any vertical variations in groundwater chemistry. In reality, where there is groundwater stratification, the composition of the pumped sample may be affected by the borehole depth, length of solid casing, the pumping regime, the intersection of fissures or zones of high permeability, and, under some circumstances, the depth of the pump.

In unconfined aquifers, it is not uncommon for boreholes of apparently similar construction, in close proximity, to produce water of quite dissimilar quality due to differences in the proportion of inflow from various depths in the aquifer. Similarly, pumped samples from widely distributed observation boreholes may differ in quality as a result of vertical rather than areal variations within the aquifer. Where groundwater quality is stratified, it is also common for pumped discharge quality to vary with pumping time, especially after periods of pump shutdown (Nightingale and Bianchi, 1980). If these possibilities are not considered then pumped water quality data will be misinterpreted.

3.2 Depth sampling

Observation boreholes are commonly too narrow to be pumped and depth sampled simultaneously, consequently, static depth sampling is still often employed. In a static borehole, water within the cased section is of uncertain origin and stagnation effects can radically modify the chemistry of the water (Marsh and Lloyd, 1980). Depth samples from the open or screened section are preferable, but still of questionable value. Logging in open observation boreholes has frequently demonstrated strong natural flow in the borehole column. The flow is generated by vertical head differentials within the aquifer and is usually directed upwards, especially in aquifer natural discharge areas and around major production boreholes. Where such conditions occur, and there is also a vertical variation in groundwater chemistry, depth samples are not necessarily representative of groundwater at comparable depth in the aquifer. This phenomenon is clearly illustrated in one of the boreholes in central Norfolk (Figure 4) in which the water quality in the borehole column is dictated by a dominant fissure inflow near the base of the borehole. This fissure contains anomalously high nitrate concentration at this depth in the aquifer.

In larger diameter boreholes it is possible for dynamic depth samples to be collected, but such sampling is not without problems. The borehole's existence disturbs the natural groundwater system and during pumping a complex flow regime may be set up in and around a borehole and short-circuit flows develop. Where groundwater quality is stratified, such complexities in the flow regime can render depth samples very misleading.

4 Concluding summary

(a) A research level of investigation has demonstrated that gross vertical stratification of groundwater quality is a common occurrence in the unconfined zones of British aquifers subject to diffuse pollution and that variations with depth are often more significant than spatial variations. An optimal sampling strategy for water quality monitoring in these types of aquifer must include methods designed to detect vertical as well as areal variations.

(b) If the objectives of groundwater quality monitoring are to provide an early warning of pollution, then, in most cases, sampling from shallow depths in the saturated zone will be the most sensitive. This strategy is, however, not applicable where density effects or complex hydraulic properties override.

(c) Discharge samples from fully-penetrating production boreholes are of much less value in that context since mixed samples from a considerable depth range will tend to mask the impact of pollution.

(d) Any depth sampling must be controlled and interpreted using borehole flow logging. The detail of vertical stratification cannot readily be determined from depth samples.

(e) Analysis of pore-water from cored boreholes is the best method of detecting groundwater quality stratification. Another method, particularly for fissured aquifers, is the use of inflatable packers to obtain pumped samples from isolated depth intervals within a single borehole. Neither method, however, is currently practical on a routine basis. Observation boreholes screened over narrow depth ranges and periodically pumped or, in certain circumstances, "in-situ" samplers installed in backfilled boreholes, may be used to monitor groundwater quality variations with depth.

Notes

[1] Static conditions are defined as those where there is no pump in the borehole but natural flows or those induced by adjacent pumping boreholes, may occur. Dynamic conditions are designated as referring exclusively to cases where water is actually being removed from the borehole itself.

Acknowledgements

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References

Edmunds, W.M. and A.H. Bath. 1976. Centrifuge extraction and chemical analysis of interstitial waters. Environ. Sci. Tech., 10: 467-472.

Foster, S.S.D. and A.S. Robertson. 1977. Evaluation of a semi-confined Chalk aquifer in East Anglia. Proc. Inst. Civil Eng., 63: 803-817.

Foster, S.S.D., A.C. Cripps and A.K. Smith-Carington. 1982. Nitrate leaching in groundwater. Phil. Trans. R. Soc. Lond. 296: 477-489.

Marsh, J.M. and J.W. Lloyd. 1980. Details of hydrochemical variations in flowing wells. Groundwater, 18: 366-373.

Nightingale, H.I. and W.C. Bianchi. 1980. Well-water quality changes correlated with well pumping time and aquifer parameters - Fresno, California. Groundwater, 18: 274-280.

Tate, T.K., A.S. Robertson, and D.A. Gray. 1970. The hydrogeological investigation of fissure flow by borehole logging techniques. Q.J. Eng. Geol., 2: 196-215.

STATISTICAL TREATMENT OF SEDIMENTARY STRUCTURES IN THE UNDERGROUND AND ITS USE FOR GEOHYDROLOGICAL SAMPLING PROCEDURES

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Abstract

In geohydrological survey several sampling schemes can be used. For optimal adjustment of the sampling procedures to a particular sedimentary substratum at various scales sedimentary structures should be discerned. Knowledge of the depositional circumstances can be very helpfull for this geological analysis. Besides qualitative descriptions of heterogeneities quantitative analysis is possible through application of various statistical tools, such as the semi-variogram. An improved sampling procedure is proposed by application of repeatedly statistical testing of a structural hypothesis by means of analysis of variance and the subsequent adjustment of the sampling scheme. Also some remarks are made concerning the cost benefit ratio of sampling procedures.

Resumé

Dans la reconnaissance géohydrologique on peut appliquer des schèmes divers d'échantillonnage.
Pour l'adaptation optimal de la procédure d'échantillonnage à un certain souterrain sédimentair, c'est recommendable de discerner des structures sédimentaires sur des échelles différentes. La connaissance des conditions sédimentaires peut être serviable pour cet analyse géologique. Excepté les descriptions qualitatives des hétérogénéités, c'est possible d'obtenir une analyse quantitative au moyen de l'application des instru-

ments statistiques divers, comme le semi-variogramme.
Une procédure améliorée d'échantillonnage est proposé par l'application des tests statistiques à plusieurs reprises, d'une hypothèse structurelle à moyen d'une analyse de variance et ensuite de l'adaptation du schème d'échantillonnage.
Aussi quelques observations sont fait, touchant la relation de l'évaluation de frais et des bénéfits des procédures d'échantillonnage.

1 Introduction

Is there any problem in executing a geohydrological reconnaissance of a particular area by means of sampling?
At first sight there seems to be no problem at all!
A sampling scheme is chosen, suited to the purpose and the sampling is carried out, using the kind of sampling technique that reveals the detailes wanted.

At closer view however, doubts may rise about this approach, regarding for instance the different sampling schemes that are available. A few of the most important ones are the regular scheme and the random and stratified random schemes (Webster, 1977). In general, the stratified random scheme is preferred. It stands for a fixed number of sampling points at random chosen within the cells of a regular network, covering the area to be sampled. The fully random scheme has the disadvantage of a possibly uneven covering of the area, whereas the regular scheme is not to be preferred in an unknown area because of a possible spatial cyclicity in the parameters, characterizing the subsoil.

The possible spatial behaviour of the geohydrological parameters brings us to the following question: Can a priori knowledge of the sedimentary structure of the subsoil contribute to a more sophisticated sampling procedure? In order to try to answer this question, some notes on sedimentary structures will be made. Independent of a possibly satisfactory answer to the geological question practical questions remain:
The most important ones concern items like the spatial density of the sampling scheme, optimal refinement of the scheme and the cost-benefit

ratio.
Before proposals will be made for improvement of the sampling procedure a few practical statistical tools will be discussed briefly.
For classical geohydrological sampling problems reference is made to Willardson and Hurst, (1965) and to Dylla and Guitjens (1970).

2 Sedimentary processes, can they influence geohydrological parameters?

If the relics of sedimentary structures, such as a cross bedding have been preserved more than fragmentaric in the underground, the lithological structure results in a predictable distribution of hydraulic permeability and porosity (e.g. Weber et al, 1972).
This spatial distribution is the result of processes during the deposition of the sediment and possibly also due to processes after sedimentation has taken place, such as bioturbation, soil genesis, etc. The processes governing the sedimentation depend on three main factors, viz. the sedimentary environment (fluvial, marine, etc.), the energy of this environment, available for sediment transport and the abundance of, and the quality of the sediment available.
The primairy factor controlling the character of a sediment body is the environment in which it is deposited. This is particularly illustrated in the geometrical extension of shale intercalations (Figure 1).

The second important factor, the energy of the (sub-)environment determines the bedforms, given the environment and the kind and abundance of sediment.
In Figure 2 an illustration is given of the variety of bedforms as a function of the energy of the transporting medium and the mean fall diameter of the sediment.

As energy is one of the main factors governing the bedforms, regarding the energy fluctuations within for instance a river in time and space, it is to be expected that different kinds of bedforms are formed in relatively short distance and time (Figure 3).

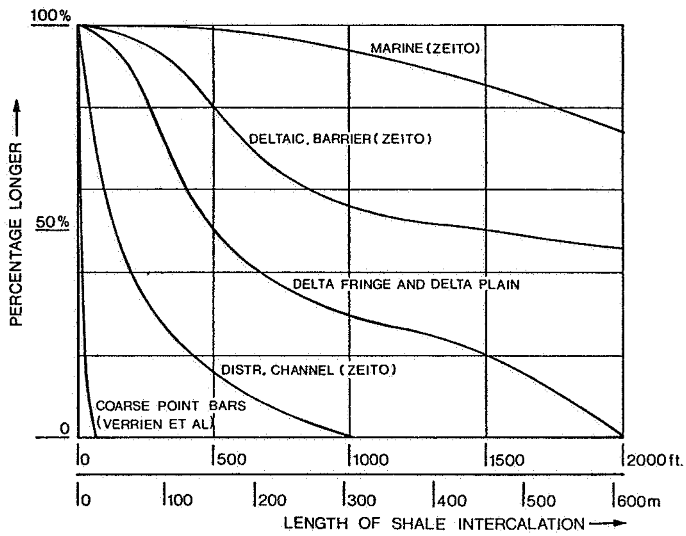

Figure 1 Continuity of shale (silt) intercalations as a function of depositional environment, after Weber (1980)

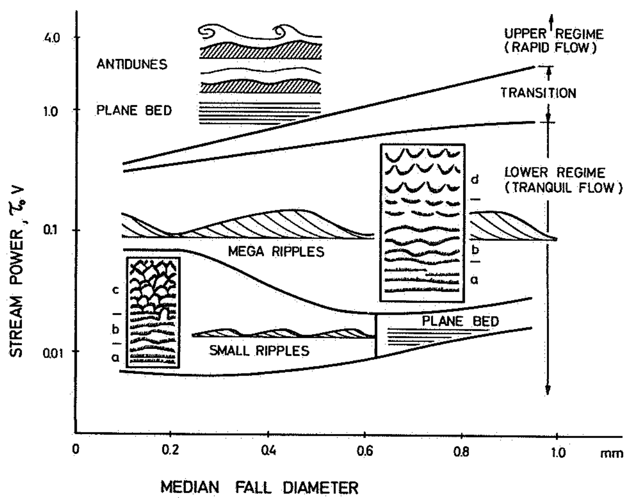

Figure 2 Schematic representation of various bedforms and their relationship to grain size and stream power, after Reineck and Singh (1980)

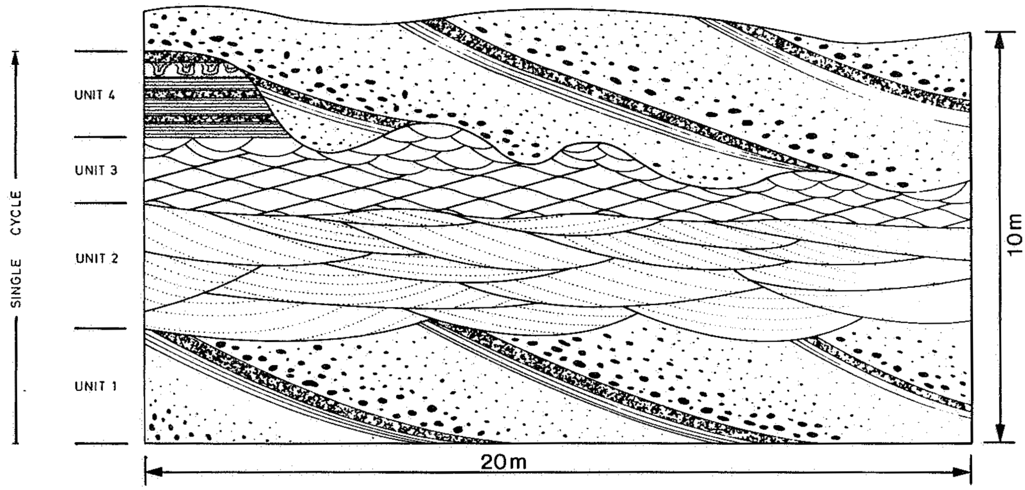

Figure 3 Schematic vertical sequence of a braided river deposit, after Reineck and Singh (1980)

Not seldomly bedforms are parts of other larger bedforms, to form so-called nested structures. Besides this kind of scale effects, the scales in sediment structures are generally formed in climbing dimensions in fluvial environment for instance by the remnants of river bars, river levees, channel fills of cut-off river bends, a meandering beld, a flood plain, etc.

Returning to the question, which influcence sedimentary processes may have on the spatial distribution and quality of geohydrological parameters, given the type of environment, the energy properties of the sediment transporting medium turn out to be the controlling factor. In figure 3 for instance diminishing energy leads to a fining upward of the grain size, from unit 1 to unit 4 and within the cross bedding of unit 1. The permeability contrast is obvious in unit 1, as the coarse bottom of the next oblique layer (seen from left to right) immediately follows the fine grained top layer of the previously sedimentated layer. The same structural forms influence other parameters like the porosity. In general, the values of geohydrological parameters vary within the structural sedimentary unit and between several units. In addition this "behaviour of variation" repeats itself for each scale of heterogeneity.

The spatial distribution of the hydraulic permeability and the porosity is well illustrated by the article of Pryor (1973) for different sedimentary environments. An overview of the origin and extension of sandstone bodies can be found in the articles of Le Blanc (1977) (unconsolidated sand layers are also regarded as a "sandstone"). In general, for information about sedimentary processes and bedforms reference is made to Reading (1978), and Reineck and Singh (1980).

3 Manipulation and interpretation of spatial data

The main problem one encounters when the science of sedimentology is consulted for sampling problems is the translation of the often qualitative data and descriptions of e.g. bedforms to quantitative data. In literature many reports can be found of the reversed case: sedimentological interpretations of data from intensively sampled areas. See for instance Smith (1981), Dowd and Royle (1977), Bennion and Griffiths (1966), Royle (1977), Dijkstra and Kubik (1974) and David and Dagbert (1974). These data interpretations are all made, using statistical tools. The main tools, used are: the (auto)correlogram, the semivariogram, and the frequency distribution (Figure 4).

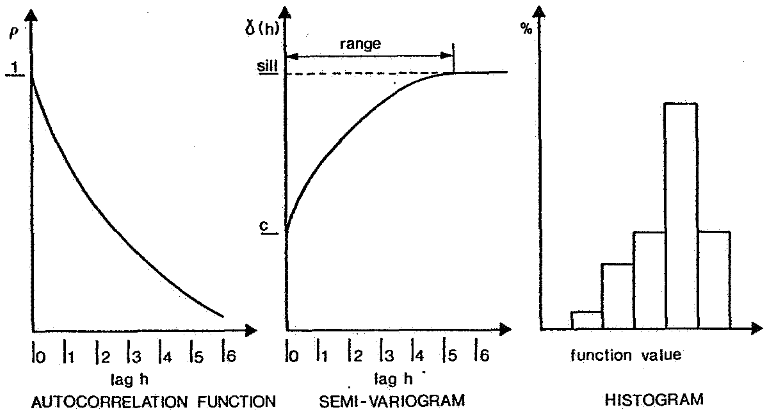

Figure 4 Some statistical functions often used in sampling data interpretation

Other statistical tools such as the frequency analysis and the power spectrum are also used.
The first two functions depend on the relative distance between the sampling locations, often entitled as "lag". The first function shows the correlation of a certain spatial variable, e.g. a geohydrological parameter as a function of the lag. The second function, the semi-variogram stands for the variance of the differences between values of the spatial variable at different sampling locations, with the same lag, as a function of this lag. (Journel and Huijbregts (1978), Delhomme (1978) and Clark (1979)). Besides these spatial statistical functions a pure statistical function is also used: the frequency distribution. This is often expressed in a so-called histogram.

A relation between the correlogram $\rho(h)$ and the semi-variogram $\gamma(h)$ exists, as can be seen in the following:

$$\gamma(h) = \frac{1}{2N(h)} \sum_{i}^{N(h)} (F(r_i) - F(r_i + h))^2 \qquad (1)$$

in which $N(h)$ = number of pairs, at relative distance h
F = spatial variable
r_i = the i^{th} position of the sampled spatial variable (one position can be used more than one time!).

As the mathematical expectation, denoted by $E(\cdot\cdot)$ stands for $\frac{1}{N}\Sigma(\cdot\cdot)$, (1) can be written as:

$$\gamma(h) = \tfrac{1}{2}E\,(F(r) - F(r + h))^2 \qquad (2)$$

After some elaboration (2) turns into:

$$\gamma(h) = \mathrm{var}\,(F) - \mathrm{covar}\,(F(h)) \qquad (3)$$

Herein: $\mathrm{var}\,(F) = E\left[(F(r) - \bar{F})^2\right]$, the variance of F. (4)

$\mathrm{covar}\,(F(h)) = E\left[(F(r) - \bar{F})(F(r + h) - \bar{F})\right]$, the covariance of F for h. (5)

As the auto-correlation function $\rho(h) = \text{covar}(F(h))/\text{var}(F)$, (3) can be turned into: (6)

$$\gamma(h)/\text{var}(F) = 1 - \rho(h) \tag{7}$$

Formula (3) and (7) apply for no trend conditions. So, if for augmenting h, covar (F(h)) goes to zero, formula (3) shows that $\gamma(h)$ approaches to var (F) in a no trend condition.
In case formula (3) applies, the covariance function can be plotted against the lag h in the same way as the semi-variogram.
Formula (3) shows us that the covariance function is, for no trend condition, the complement of the semi-variogram with respect to var (F).
For the semi-variogram some additional characteristics should be mentioned. Viz.: the range, the nugget effect C and the sill. The range is the lag value at which for increasing h values, exceeding the range, no correlation between F(r) and F(r + h) are to be expected any more. The range can sometimes be interpreted as the geometric dimension of a structural unit in the subsoil (see for instance Journel and Huijbregts (1978 p. 160)). The second characterization is the value of $\gamma(h)$, often expressed by the symbol "C", indicating the nugget effect. This can be considered as a mixture of measuring variance and $\gamma(h)$-values for lag values smaller than the smallest lag value used in the semi-variogram. For the maximum value of $\gamma(h)$ the notion sill is used. The semi-variogram as well as the covariance function can (also) be used to express the influence of scale effects to the distribution of variance of the parameters.

4 Statistical analysis of variance at different scales

For sedimentary reasons different scales are to be expected in the structures that characterize the subsoil. The question can, however, be put whether these scaled structures significantly influence the behaviour of the variance of the sampled parameter value.
To put it more practically: In case on sedimentological grounds, a subdivision can be made of a sampled area, how can the decision be made whether this hypothetical subdivision can be accepted or should be

rejected?
To answer this question a proper statistical procedure is available, viz. the analysis of variance, extended with the calculation of variance components of different scales. (Webster, (1977), p. 77-98)).
Table 1 can serve as a working guideline. In this table the existence of four nested scales of heterogeneities are supposed. Each unit of scale level i-1 contains N_i units of scale level i; so the total number of samples is $N_1 \times N_2 \times N_3 \times N_4$. For reasons of simplicity at one and the same scale level i the same number N_{i+1} of sample units of level i+1 are taken, all over the area.

Starting with the highest level number (at the largest scale) the sum of squares are calculated, corrected for the mean parameter value at this local scale level, as is indicated at Table 1 for level 4. Easily the next step can be executed by calculating the mean of squares. So, for each level the sum of squares and the mean of squares can be calculated. The question concerning the significance of a chosen subdivision can be answered by executing the F-test on the mean of squares of these subdivided units and the mean of squares of the lower level that contains these units.
The F- or Fisher test can be used to test the hypothesis that two variances differ significantly.
Important elements of the analysis of variance are the components of variance, that characterize the different scales. They can also be calculated from Table 1.
As an illustration to this analysis of variance an example of three groups, each consisting of six permeability values is treated in Table 2 and Table 3. For this example the conclusion can be drawn that the proposed subdivision is statistically confirmed.

Table 1 Analysis of variance table

Level of subdivision	Degrees of freedom (DGF)	Sum of squares	Mean of squares (optimal estimates)	The components of variance, estimated by the mean of squares
1	$N_1 - 1$	$S_1 = N_2 \cdot N_3 \cdot N_4 \sum_i (\overline{X}_i - \overline{X})^2$	$V_1 = S_1/(N_1-1)$	$\sigma_4^2 + N_4\sigma_3^2 + N_3 \cdot N_4 \cdot \sigma_2^2 + N_2N_3N_4\sigma_1^2$
2	$N_1 \ (N_2 - 1)$	$S_2 = N_3 \cdot N_4 \cdot \sum_i \sum_j (\overline{X}_{ij} - \overline{X}_i)^2$	$V_2 = S_2/(N_1 \cdot (N_2 - 1))$	$\sigma_4^2 + N_4\sigma_3^2 + N_3N_4\sigma_2^2$
3	$N_1 \cdot N_2 \cdot (N_3 - 1)$	$S_3 = N_4 \sum_i \sum_j \sum_k (\overline{X}_{ijk} - \overline{X}_{ij})^2$	$V_3 = S_3/(N_1 \cdot N_2 \cdot (N_3-1))$	$\sigma_4^2 + N_4\sigma_3^2$
4	$N_1 \cdot N_2 \cdot N_3 (N_4-1)$	$S_4 = \sum_i^{N_1} \sum_j^{N_2} \sum_k^{N_3} \sum_l^{N_4} (X_{ijkl} - \overline{X}_{ijk})^2$	$V_4 = S_4/(N_1 \cdot N_2 \cdot N_3 (N_4-1))$	σ_4^2
	$N_1N_2N_3N_4-1$	$S = \sum_i \sum_j \sum_k \sum_l (X_{ijkl} - \overline{X})^2$	$S/(N_1 \cdot N_2 \cdot N_3 \cdot N_4-1)$	

Test on significance of V_i to V_{i+1}: $V_i/V_{i+1} < F_{1-\alpha}\left[DGF_i, DGF_{i+1}\right]$, in case $V_i > V_{i+1}$; for $V_{i+1} > V_i$ test V_{i+1} to V_i. Variance total $= \sigma_{tot}^2 = \sigma_1^2 + \sigma_2^2 + \sigma_3^2 + \sigma_4^2$ (Webster (1977, p. 77-98)).

Explanation of the symbols of Table 1.

X_{ijkl} = the l^{th} value of the spatial variable at level 4 of subdivision.

$\overline{X}_{ijk}$ = the k^{th} mean of the spatial variable X_{ijkl} at level 4. Other means can be explained in the same way.

N_4 = the number of observations at level 4, per unit of the 3^{rd} level; of this last level N_3 units exist. Other numbers can be explained in the same way.

σ_4^2 = the theoretical variance of the spatial variable at level 4, <u>within</u> units of the 3^{rd} level.

σ_3^2 = the theoretical variance <u>between</u> units of the 3^{rd} level, <u>within</u> units of the 2^{nd} level. Other variances can be explained likewise.

$F_{1-\alpha}\left[DGF_i, DGF_{i+1}\right]$ = the Fisher F-distribution for $(1-\alpha) \times 100\%$ of reliability of the test, and DGF_i and DGF_{i+1} degrees of freedom of respectively the variance with subscript i and the variance with subscript i+1. (See for instance Sachs, (1978)).

Table 2 Example of two levels of subdivision: three groups of six permeability values: $N_1 = 3$; $N_2 = 6$

	field 1	field 2	field 3	formulas from table 1
	$X_{1,j}$	$X_{2,j}$	$X_{3,j}$	
k values in m/s times 100 000	105.01	30.53	15.74	
	110.81	56.89	23.85	
	73.75	23.65	19.61	
	91.02	45.37	11.09	
	88.81	43.74	25.57	
	83.57	35.23	16.29	
mean $\overline{X}_i$	92.16	39.235	18.69	$\overline{X}_i = \frac{1}{N_2}\sum_{i=1}^{N_2} X_{ij}$; $\overline{X} = 50.03$
sum of squares corrected for mean, V_i	938.18	704.34	147.03	$V_i = \sum_{j=1}^{N_2} (X_{ij}-\overline{X}_i)^2$

Table 3 Analysis of variance table of the data of Table 2.

level of sub-division	degrees of freedom DGF	sum of squares	mean of squares	components of variance
1	$N_1 - 1 = 2$	$S_1 = N_2 \sum_{i=1}^{N_1} (\overline{X}_i - \overline{X})^2 =$ 17241.99	$V_1 = S_1/(N_1-1) =$ 8621.00	$\sigma_2^2 + N_2 . \sigma_1^2$
2	$N_1 . (N_2-1) = 15$	$S_2 = \sum_{i=1}^{N_1} \sum_{j=1}^{N_2} (X_{ij} - \overline{X}_i)^2$ = 1789.55	$V_2 = S_2/(N_1 . (N_2-1))$ = 119.30	σ_2^2

Conclusion: the components of variance can be calculated as follows: $\sigma_2^2 = 119.30$; this makes $\sigma_1^2 = (8621.00 - 119.30)/6 = 1416.95$. $\sigma_{tot}^2 = 1416.95 + 119.30 = 1536.25$. As $V_1/V_2 = 8621.00/119.30 = 72.26$ is much greater than $F_{0.95}\,[2,15] = 3.68$ the assumption that the variance between the fields is not significant, compared with the variance within the field is rejected.

An other important conclusion is that some 8 percent of the total variance is contributed by the component of the variance within the fields, whereas 92 percent is contributed bij the component of the variance between the fields.

At this point the question concerning the relation between the variances at different scale levels, the semi-variogram and the co-variogram can be answered as follows:
For an increasing relative distance between sampling points, growing from local, areal to regional proportions, the influence of the co-variance of these respective scales diminishes subsequently so, that at last only the regional aspect remains, as can be seen in Figure 5.

In a complementary way a semi-variogram can be thought to be built up when the lag is scaled from points to regions.
For argumentation of the summable properties of the co-variance function and the semi-variogram reference is made to Dijkstra and Kubik (1974) to Miesch (1975) and to Journel and Huijbregts (1978, p. 148).

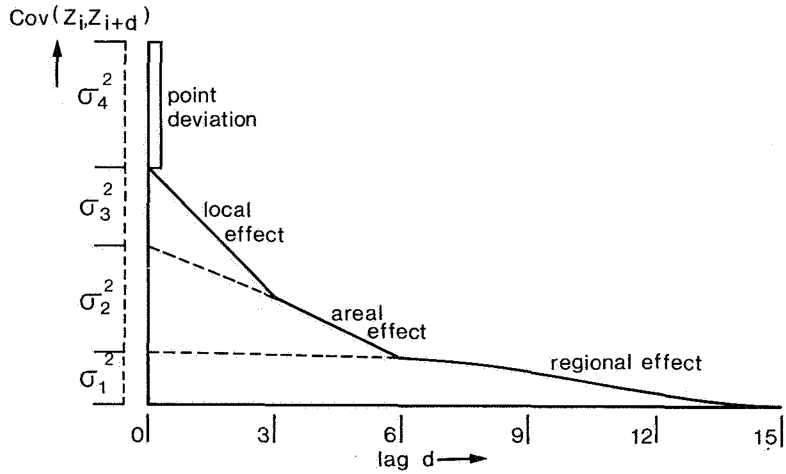

Figure 5 Illustration of a co-variance function showing scaling effects, after Dijkstra and Kubik (1974)

To illustrate the similar behaviour of the semi-variogram an example is given. It concerns a semi-variogram of the phreatic groundwater heads in the Sleen hydrological test area in the eastern part of the Netherlands at August 28, 1978 (Figure 6).

As indicated by Figure 5 a change in contribution to the co-variance function due to exceedance of h of a certain scale may affect the co-variance function by a bend or an inflexion point in the curve. This indicates a change in the diminishing of the co-variance with increasing lag.
Regarding now Figure 6 two changes in the augmentation of the semi-variogram with lag can be discerned: A change between 400 and 800 m and a "change" lying at about 8 km (analysed is a range of $\sqrt{3}$x5.11 = 8.85 km), representing the regional effect.

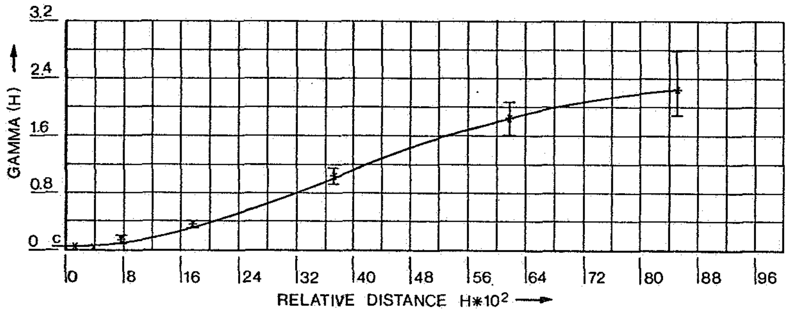

Figure 6 Semi-variogram of phreatic groundwater heads of the Sleen test area at August 28, 1978

Theoretical type of semi-variogram is Gaussian:

$\gamma(h) = w.(1. - \exp(-(h/a)^2) + c$

with: a = 5110 m, w = 2.366 m² ($\gamma(h)$ is in m²!)

Standard deviation (observed-calculated) is: 0,03122 m².

The percentage of the confidence region is 95.

The optimization is made by the RID computerprogram VAREST.

Keeping in mind the similarity between $\gamma(h)$ and a summation of variances, contributed by different scales, an explanation for the behaviour of the semi-variogram can be searched for in a scale dependent semi-variogram contribution of the phreatic groundwater heads. Indeed two scales of dewatering canals, fed by groundwater exist in the Sleen test area. One scale of dewatering canals consists of ditches with a relative distance of about 500 m; the other scale has the dimension of the relative brook distance of 5 to 8 km.

These scales of dewatering canals fit well the change in augmentation of the semi-variogram with lag h, mentioned above.

In case of a semi-variogram of porosity values or hydraulic permeability values the same analysis can be made, this time to be explained by a sedimentologist instead of a hydrologist.

5 Proposal for a sampling procedure in which sedimentological interpretation and statistical analysis of data are incorporated

In order to improve the general sampling schemes, mentioned in the introduction, such as the stratified random sampling scheme,good use can be made of the science of sedimentology and of a statistical analysis of data.

The way for improvement, however, contains an important internal contradiction:

To be able to improve spatial sampling a priori knowledge must be available at a scale level that is not sampled yet!

Consequently, the conclusion to be drawn is that necessarily one or more cycles of a sampling procedure have to be executed.

In these cycles gathering of information and interpretation of this information to set up an additional sampling scheme are alternating.

The essential sampling cycle consists of the following steps:

a. gathering of information from archives, to be interpreted structurally
b. structural analysis of the sampling area by geophysical prospecting, when usefull
c. formation of a structural hypothesis by a geologist for the scale level to be sampled next; when possible an indication should be given on the expected variances within and between the structural units, possibly obtained from sampling experience
d. construction of the sampling scheme
e. execution of the sampling scheme
f. analysis of the variance from which the conclusion can be drawn whether or not the structural subdivision at the sampled level can be proved to be statistically significant.

Using the geological information, extracted from the samples obtained from this last cycle the next cycle can start at step c.

In short this sampling cycle contains the trio: structural hypothesis, formation and execution of the sampling scheme and the analysis of

variance.
Of course in case of a multiscale structural hypothesis some sampling cycles can be combined to one cycle, whereafter the analysis of variance can be executed as given in Table 1.

To this cyclic sampling procedure the following remarks can be made:
Between step c and step d a relation should exist between sampling costs, the variances estimated and the sampling scheme to be constructed.
When total costs for a sampling cycle, denoted c, is split up in costs for groups of samples (= C_1) and the individual sample costs (= C_2) the next expression can be formed: $C = C_1 N_1 + C_2 . N_1 . N_2$. Herein N_1 and N_2 are respectively the number of sample groups and the number samples within one group or the number subgroups within one main group.
Taking the cost benefit ratio as a starting point a relation can be formulated between the costs, expressed as above and the amount of information to be gained, which can be expressed by the variance of the mean value of the main group of sampled values, estimated in advance:

$$\mathrm{var}\ \bar{X}_1 = \mathrm{var}_1/N_1 = \frac{\sigma_2^2}{N_1 . N_2} + \frac{\sigma_1^2}{N_1}$$

In order to optimize the costs per information gain the next expression F is to be minimized for N_2 and N_1:

$$F = (C_1 . N_1 + C_2 . N_1 . N_2)\left(\frac{\sigma_1^2}{N_1} + \frac{\sigma_2^2}{N_1 . N_2}\right) = (C_1 + C_2 . N_2)(\sigma_1^2 + \sigma_2^2/N_2)$$

(F is independent of N_1!)

This results in $N_2 = \frac{\sigma_2}{\sigma_1} \sqrt{\frac{C_1}{C_2}}$ and $N_1 = \frac{C}{C_1 + C_2 . N_2} = \frac{C}{C_1 + \frac{\sigma_2}{\sigma_1}\sqrt{C_2 . C_1}}$

(see also Webster (1977)).

In practice it is not uncommon to execute double sampling: two samples taken at very short distance. This can be usefull as well for averaging measuring errors as for detecting short distance variation. Reworked into a semi-variogram the contribution to the semi-variance at this short distance will approach the nugget effect.

As a result of the analysis of variance a sophisticated semi-variogram can be constructed, which expresses the summed influences of variances characterizing the scale levels at which samples are taken.

References

Bennion, D.W. and J.C. Griffiths, 1966. A stochastic model for predicting variations in reservoir rock properties. Trans A.I.M.E. vol. 237, part 2, pp. 9-16.

Clark, I., 1979. Practical Geostatistics. Applied Science Publishers, Ltd. London, pp. 129.

David, M. and M. Dagbert, 1974. Lakeview revisited: variograms and correspondence analysis - new tools for the understanding of geochemical data. In: Developments in Economic Geology I: Geochemical Exploration, I.L. Eliot and W.K. Fletcher (eds.), Elsevier, Amsterdam.

Delhomme, J.P., 1978. Kriging in Hydroscience, Advances in Water Resources, vol. 1, no. 5, pp. 251-266.

Dowd, P.A. and A.G. Royle, 1977. Geostatistical applications in the Athabasca tar sands. In: Proceedings of the 15th APCOM Symposium, Brisbane, Australia, pp. 235-242.

Dijkstra, S. and K. Kubik, 1974. Autocorrelation studies in the analysis of stream sediment data. In: Developments in Economic Geology I: Geochemical Exploration, I.L. Eliot and W.K. Fletcher (eds.), Elsevier, Amsterdam.

Dylla, A.S. and J.C. Guijtjens, 1970. Hydraulic conductivity sampling for confidence. Transactions of the ASAE, vol. 13, pp. 485-488.

Journel, A.G. and Ch.J. Huijbregts, 1978. Mining Geostatistics, Academic Press, London, 600 pp.

Le Blanc, R.J., 1977. Distribution and continuity of sandstone reservoirs, Journal of Petroleum Technology, July 1977, part 1: pp. 776-792, part 2: pp. 793-804.

Miesch, A.T., 1975. Variograms and variance components in geochemistry and ore evaluation. In: Quantitative studies in the geological sciences, E.H.T. Whitten, (ed.), Mem. Geol. Soc. Am., 142, pp. 333-340.

Pryor, W.A., 1973. Permeability - porosity patterns and variations in some Holocene sand bodies, The Am. Soc. of Petroleum Geologists; vol. 57, no. 1, pp. 162-189.

Reading, H.G. (ed.), 1978. Sedimentary Environments and Facies, Blackwell Scientific Publ., Oxford, pp. 556.

Reineck, H.E. and I.B. Singh, 1980. Depositional Sedimentary Environments, Springer Verlag, Berlin; 548 pp.

Royle, A.G., 1977. Geostatistical recognition of structure in beach sand. In: Proceeding of the 15th APCOM Symposium, Brisbane, Australia, pp. 231-234.

Sachs, L., 1978. Angewandte Statistik, Springer Verlag, Berlin, 552 pp.

Smith, L., 1981. Spatial variability of flow parameters in a stratified sand, Mathematical Geology, vol. 13, no. 1, pp. 1-21.

Weber, K.J., R. Eijpe, D. Leijnse and C. Moens, 1972. Permeability distribution in a Holocene distributary channel-fill near Leerdam (The Netherlands), Geologie en Mijnbouw, vol. 51 (1), pp. 53-62.

Weber, K.J., 1980. Influence on fluid flow of common sedimentary structures in sand bodies, Society of Petroleum Eng. of AIME, SPE 9247, 12 pp.

Webster, R., 1977. Quantitative and numerical methods in soil classification and survey, Clarendon Press, Oxford.

Willardson, L.S. and R.L. Hurst, 1965. Sample size estimation in permeability studies, Journal of the Irrigation and Drainage Division of the Am. Soc. of Civil Eng., March 1965, pp. 1-9.

APPLICATION OF THE OBSERVER THEORY IN OPTIMIZATION OF GROUNDWATER MONITORING NETWORK

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Abstract

The method proposed is based on three concepts already existing in control theory: state equations, observability of the system and observer systems. A canonical form of state equations for an arbitrary groundwater system is derived and a simple observability criterion is proposed. For an observable groundwater system the observer system is constructed. The state reconstruction error is used as a criterion in optimizing a groundwater measuring network.

1 Introduction

Optimization of groundwater monitoring networks must always be considerd a multipurpose decision making procedure. It should combine internal and external conditions that determine measurements of any hydrogeological process. By internal conditions all physical relationships (processes, parameters, boundary conditions etc.) are understood. External conditions are those introduced by monitoring activities of men (instrumentation, sampling frequency, spatial distribution of measuring points etc.). In order to cope with the complexity of the monitoring problem the same mathematical description for both internal and external conditions must be used. State space approach, already existing in control theory, seems to be an ideal means for the purpose.

State equations for a given groundwater system together with the observation equation for an existing or planned monitoring network can easily be used to check an observability criterion. The latter is an algebraic criterion on whether the state (e.g. piezometric head) of the groundwater system can be completely reconstructed, providing measurements of the state at some discrete points of the groundwater system are available. If an answer to this question is negative the observability criterion can be used for planning locations of new measuring points. This should then ensure observability of the system from the enlarged monitoring network. If the answer to the observability question is positive, the Luenberger's observer can be constructed for a given groundwater system and measuring network under consideration. The observer is an abstract dynamic system having the state convergent to the state of physical system. Having the observer for the groundwater system built the unknown coordinates of the state vector can be reconstructed. Especially those which have become unknown after removing one or more measuring points. For such points the reconstruction error is defined as the mean square departure of reconstructed values from actual ones. The method proposed suggests to decision makers to remove those measuring points for which the reconstruction error is small.

Before applying the optimization procedure the following four assumptions must be fulfilled. Assumption one: a groundwater monitoring network does already exist and periodic measurements of groundwater level in some discrete points are available. Assumption two: boundaries and physical parameters of the groundwater system are knwon. Assumption three: boundary conditions, infiltration and pumping rates are all known over the observation period. Assumption four: the groundwater system is "noise-free", i.e. the stochastic factor plays a minor role in the system's evaluation in time. The last assumption is usually valid for deep aquifers for which all interactions with their environment are smoothed down. For phreatic aquifers the assumption must be carefully examined.

2 State representation of the Boussinesq's equation

The Boussinesq's equation represents well physically based mathematical description of piezometric head dynamics in an arbitrary groundwater system. For 2-dimensional horizontal flow in a domain Ω the equation together with initial and boundary conditions can be written as follows:

$$\begin{cases} \mu \dfrac{\partial h}{\partial t} = \dfrac{\partial}{\partial x} (T_x \dfrac{\partial h}{\partial x}) + \dfrac{\partial}{\partial y} (T_y \dfrac{\partial h}{\partial y}) + S, \text{ for } (x,y) \in \Omega \text{ , } t > o & (1) \\ h(x,y,o) = h_o(x,y) \text{ , for } (x,y) \in \Omega & (2) \\ h(x,y,t) = g(x,y,t) \text{ , for } (x,y) \in \Gamma_1 \quad \partial\Omega \text{ , } t > o & (3) \\ (- T_x \dfrac{\partial h}{\partial x} \text{ , } - T_y \dfrac{\partial h}{\partial y}) = \underline{f} \text{ , for } (x,y) \in \Gamma_2 = \partial\Omega \; \Gamma_1 \text{ , } t > o & (4) \end{cases}$$

where

h, h_o = piezometric head (m)
T_x,T_y = transmissivity (m^2/day)
S = source/sink term (m^3/m^2.day)
μ = storativity (-)
g = prescribed piezometric head along Γ_1 boundary (m)
$\underline{f}$ = prescribed flow through Γ_2 boundary (m^3/m.day)

Analytical solutions to the Boussinesq's equation are restricted to simple cases. For arbitrary hydrogeological conditions a number of numerical methods have been developed to calculate approximate solutions. In the following the finite element method has been applied in order to get an approximate solution of the Boussinesq's equation $\underline{h} = (h_1,h_2,\ldots,h_n)^T$ in a finite number of nodal points. The resulting ordinary differential equations have the form of state equations:

$$D\dot{\underline{h}} + A\underline{h} = B\underline{u} \tag{5}$$

where

$\underline{h}$ = state vector (nx1)

D,A = parameter matrices (nxn)

$\underline{u}$ = input vector (source/sink terms and boundary conditions) (kx1)

B = input distribution matrix (nxk)

State equations (5) can be solved numerically by any stable integration scheme (e.g. Crank-Nicholson scheme) but as it was observed by many authors (e.g. S.P. Kjaran, 1977; Nawalany, 1980) such schemes are not suitable for qualitative analysis of the system dynamics. J. Eliasson et al. (1976) have proposed the mode superposition approach which seems to be very useful from this point of view. Firstly, a stationary equation has been solved, i.e.:

$$A\bar{\underline{h}} = B\bar{\underline{u}} \tag{6}$$

where

$\bar{\underline{u}}$ = average input vector

$\bar{\underline{h}}$ = stationary solution

Then by substracting the stationary equation (6) from the state equation (5) the state equation for the transient part $\underline{h}_1$ of the state vector $\underline{h}$ has been obtained:

$$D\dot{\underline{h}}_1 = - A\underline{h}_1 + \underline{u}_1 \tag{7}$$

where

$\underline{h}_1(t) = \underline{h}(t) - \bar{\underline{h}}$

$\underline{u}_1(t) = B(\underline{u}(t) - \bar{\underline{u}})$

It has been shown by S.P. Kjaran (1977) that in order to find a contineous-time solution to the equation (7) it is convenient to solve a generalized eigenproblem for matrices D and A, i.e.

$$\lambda_i D\underline{\phi}_i = A\underline{\phi}_i \quad , \quad (i = 1,\ldots,n) \tag{8}$$

and represent the transition solution $\underline{h}_1$ in an eigenvectors expansion.

As matrix D is symmetrical and positive definite, while matrix A is symmetrical the generalized eigenproblem (8) is equivalent to the ordinary eigenproblem for matrix $D^{-1}A$ (M. Dryja, 1982; Szmelter, 1980). Once eigenvalues and eigenvectors have been found the transition solution can be written as follows (J. Eliassen et al., 1976):

$$\underline{h}_1(t) = \sum_{i=1}^{n} e^{\lambda_i} \; \underline{\phi}_i^T \, D \, \underline{h}_1(o) \; \underline{\phi}_i + \sum_{i=1}^{n} \int_o^t e^{\lambda_i(t-\tau)} \; \underline{\phi}_i^T \, \underline{u}_1(\tau) d\tau \; \underline{\phi}_i \tag{9}$$

The resulting solution of the state equation is a sum of stationary and transient terms:

$$\underline{h}(t) = \bar{\underline{h}} + \underline{h}_1(t) \tag{10}$$

The eigenvector and eigenvalues found can also be used for transformation of the state equation (7) into canonical (diagonal) form, which will be used in the observability criterion:

$$\begin{cases} \underline{\xi} = \Lambda \underline{\xi} + P^T \underline{u}_1 \\ \underline{\xi}(o) = P^{-1} \underline{h}_1(o) \end{cases} \tag{11}$$

where

$\Lambda = \text{diag}\ (\lambda_1, \lambda_2, \ldots, \lambda_n)$
$P = \{\underline{\phi}_1, \underline{\phi}_2, \ldots, \underline{\phi}_n\}$

Relationship between the old state vector and the new one is linear, i.e.

$$\underline{h}_1 = P\underline{\xi} \tag{12}$$

3 Observability of groundwater systems

The observation equation for an arbitrary groundwater monitoring network has the following form:

$$\underline{y} = H\underline{h} = H(\bar{\underline{h}} + \underline{h}_1) = \bar{\underline{y}} + H\underline{h}_1 \tag{13}$$

in which:

$\bar{\underline{y}},\underline{y}$ = vectors of measurements (mx1)

$\bar{\underline{h}},\underline{h}$ = state vectors (nx1)

H = observation matrix (mxn)

Without losing generality it may be assumed that observation matrix H consists of only 0-s and 1-s. Substituting (12) into (13) we obtain the observation equation for the new state vector $\underline{\xi}$:

$$\underline{y}_1 = \underline{y} - \bar{\underline{y}} = HP\underline{\xi} = \begin{bmatrix} 0\ldots010\ldots\ldots\ldots\ldots0 \\ 0\ldots\ldots010\ldots\ldots\ldots0 \\ \cdot \qquad\qquad\qquad \cdot \\ \cdot \qquad\qquad\qquad \cdot \\ \cdot \qquad\qquad\qquad \cdot \\ 0\ldots\ldots\ldots\ldots010..0 \end{bmatrix} \cdot \{\underline{\phi}_1,\underline{\phi}_2,\ldots,\underline{\phi}_n\} \cdot \underline{\xi} =$$

$$\uparrow\ \uparrow\ \ \uparrow$$
$$k_1\ k_2\ldots\ldots k_m$$

$$= \begin{bmatrix} \phi_1^{k_1} & \phi_2^{k_1} & \ldots\ldots\phi_i^{k_1} & \ldots\ldots\phi_n^{k_1} \\ \cdot & & \cdot & \cdot \\ \cdot & & \cdot & \cdot \\ \cdot & & \cdot & \cdot \\ \phi_1^{k_m} & \phi_2^{k_m} & \ldots\ldots\phi_i^{k_m} & \ldots\ldots\phi_n^{k_m} \end{bmatrix} \cdot \underline{\xi} \tag{14}$$

where

$k_1,k_2,\ldots,k_m$ = numbers of nodal points at which measuring points are located

ϕ_j^i = i-th coordinate of the j-th eigenvector

This form of observation equation is useful for examination of the observability criterion. The latter is an algebraic criterion that confirms or negates the possibility of reconstruction of the (groundwater) system's state from a given monitoring network.

For state equations in canonical form (11) the observability criterion states (D.M. Wiberg, 1971): "a system is totally observable if and only if all colums of the HP matrix are nonzero-columns". Knowing numbers of nodes at which measuring points are located (i.e. $k_1,\ldots,k_m$) one can formulate the observability criterion explicitely:

$$(\phi_i^{k_1},\phi_i^{k_2},\ldots,\phi_i^{k_m})^T \overset{?}{=} (0,0,\ldots,0)^T, \quad (i=i,2,\ldots,n) \tag{15}$$

If i_o-th column of the HP matrix is a zero-column then the i_o-th coordinate of the state vector $\underline{\xi}$ does not influence any of the coordinates of the measuring vector, so the system is not observable. In such a case the monitoring network must be enlarged. An additional measuring point must be placed at such a node k_ν that $\phi_{io}^{k_\nu} \neq 0$.

The enlarged monitoring network guarantees the system's observability, providing the i_o-th column is the only zero-column of the HP matrix. If not, other measuring points have to be added. However, it was observed by M. Nawalany (1980) that groundwater systems encountered in practice are always observable although some columns of HP matrix might be "almost" zero-columns. Hence, "practical observability of the system" may be formulated as follows: "a system is practically observable if and only if for any $i,(i=1,\ldots,n)$, there exists such j_o $(j_o=1,\ldots,m)$ that

$$|\phi_i^{k_{jo}} \cdot \delta\xi_i| > \delta y \tag{16}$$

where:

δy = an accuracy of measurements

$\delta\xi_i$ = "typical" length of the state vector coordinate ξ_i

It means that each coordinate of the state vector $\underline{\xi}$ is detectable at least at one measuring point. The criterion (16) has only theoretical value as long as the "typical" length of ξ_i is not defined. To do so one may use relationship (12) to express the $\underline{\xi}$ vector as

$$\underline{\xi} = \{\underline{\phi}_1,\ldots,\underline{\phi}_n\}^{-1}\ \underline{h}_1 = P^{-1}\underline{h}_1 \tag{17}$$

An inverse of matrix P can easily be derived from generalized orthogonality of eigenvectors $\underline{\phi}_i$, $(i=1,\ldots,n)$, i.e. $P^T DP = I$.

After some calculations it gives

$$\xi_i = \sum_{\mu=1}^{n} \phi_i^{\mu} \sum_{\omega=1}^{n} D_{\mu\omega} h_{1\omega} \tag{18}$$

where:

$D_{\mu\omega}$ = elements of D matrix

Now, the "typical" length of ξ is defined as the square root of its statistical variance, i.e.

$$\delta\xi_i = \sqrt{\sum_{\mu=1}^{n} (\phi_i^{\mu})^2 \sum_{\omega=1}^{n} D_{\mu\omega}^2 \quad \mathrm{var}(h_{\omega})} \tag{19}$$

Here $\mathrm{var}(h_{1\omega})$ has been replaced by $\mathrm{var}(h_{\omega})$, for both are equal to each other. Of course, values of $\mathrm{var}(h_{\omega})$ can be calculated only for existing measuring points while for the rest of the nodal points a reasonable guess ought to be done.

4 Reconstruction of the state vector

The reconstruction problem can be formulated as follows: when we have a groundwater system together with a specified monitoring network we are interested in the evolution in time of non-measurable state coordinates (e.g. piezometric head at arbitrary points of a given aquifer). Simulation - formulas (9) and (10) - is in principle possible but an initial value of $\underline{h}_1$ must be approximated with reasonable accuracy. Otherwise the simulated values of the state coordinates may significantly depart from the actual ones. And in practice they often do. To overcome this problem D.G. Luenberger (1966) has introduced a hypothetical dynamical system, called observer, which has the following properties:

- it is stable
- its state vector converges to the state vector of the physical system
- its input is a linear combination of input $\underline{u}_1$ and measuring vector $\underline{y}_1$

The basic Luenberger's theorem (D.G. Luenberger, 1966) states that "the observer system can be constructed if and only if the system is observable". Consequently all the coordinates of the state vector can be reconstructed if and only if the observability criterion has been positively checked. State equations for the observer system is given by:

$$\dot{\hat{\underline{q}}} = A_L \hat{\underline{q}} + B_{yL} \underline{y}_1 + B_{uL} \underline{u}_1 \tag{20}$$

where:

$\hat{\underline{q}}$ = an approximate of vector $\underline{q}$ ($\underline{q}$ is an unknwon part of the state vector $\underline{h}_1$)

Luenberger (1966) has assumed a linear relationship between the state vector itself and its unknown part $\underline{q}$, i.e. $\underline{q} = L\underline{h}_1$. The resulting approximation error equation has the following form (A. Niderlinski, 1974):

$$\Delta\dot{\underline{q}} = A_L \, \Delta\underline{q} + (A_L.L - LA - B_{yL}.H)\, \underline{h}_1 + (B_{uL} - L)\, \underline{u}_1 \tag{21}$$

where:

$\Delta\underline{q} = \hat{\underline{q}} - \underline{q}$ = approximation error

To achieve a convergence of $\Delta\underline{q}$ vector to zero (which is equivalent to convergence $\hat{\underline{q}}$ vector to $\underline{q}$ vector) it is sufficient to evaluate a stable matrix A_L and unspecified matrices L, B_{yL} and B_{uL} in such a way that expressions in brackets in (21) vanish, i.e.:

$$\begin{cases} A_L . L - L . A = B_{yL} . H \\ B_{uL} = L \end{cases} \tag{22}$$

The following theorem (R. Bellman, 1960) gives the condition for the existence of such matrices: "if the A and A_L matrices have no common eigenvalues there exists such an L that satisfies equations (22) for arbitrary A, H, A_L and B_{yL}". Efficient algorithm of finding A_L, L and B_{yL} matrices has been given by Lenberger (1966). Below, Figure 1, it is shown how the observer system is used for the state reconstruction.

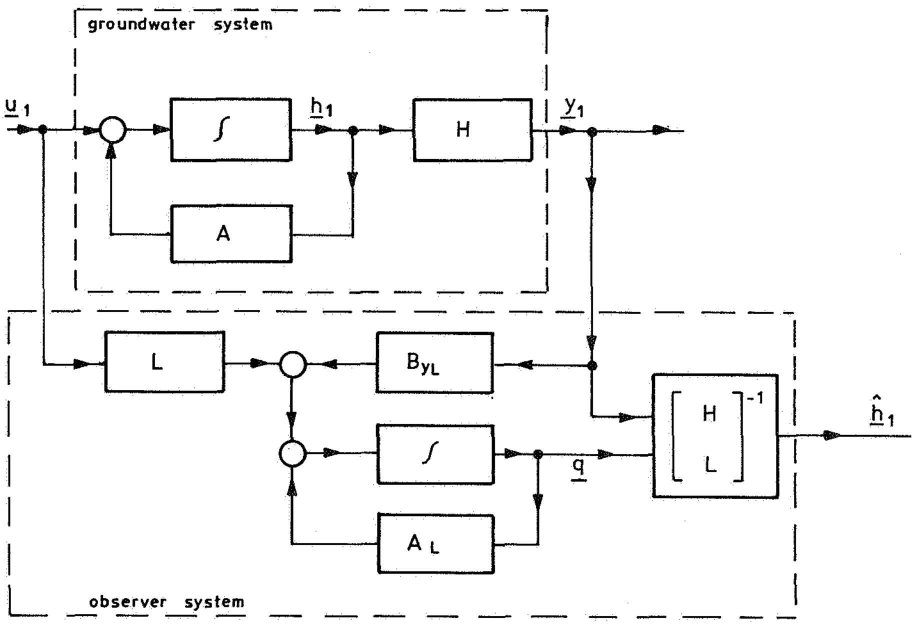

Figure 1 Groundwater system and observer systems

Once the vector q is computed from the state equation of the observer system, i.e. from

$$\begin{cases} \dot{\hat{\underline{q}}} = A_L\hat{\underline{q}} + B_{yL}\underline{y}_1 + L\underline{u}_1 \\ \hat{\underline{q}}(o) = \hat{\underline{q}}_o \end{cases} \tag{23}$$

the state vector can be estimated as

$$\hat{\underline{h}}_1 = \begin{bmatrix} H \\ \hline L \end{bmatrix}^{-1} \begin{bmatrix} \underline{y}_1 \\ \hline \hat{\underline{q}} \end{bmatrix} \tag{24}$$

5 Reduction of the existing measuring network

To reduce an existing monitoring network one has to answer two questions:

- how many measuring points must be cancelled?
- which points must be removed from the network?

Answering the first question is determined by economic, technical or sometimes human factors, rather then those of hydrogeological nature. Usually, a departure point for reduction is a statement like: "we must remove, M measuring points". In such a case, the just introduced observability concept may help in accepting or opposing the specified number M. If reduced network could still be practically observable the only task is to find an optimal residual network (i.e. to answer the second question). But if reduction of an existing network by M points makes the system non-observable decision makers must be warned for the consequences of keeping residual network which is not capable to monitor the groundwater system completely. In this case the observability criterion applied to different reduction numbers M' < M and to different configurations of residual networks can answer the question how many measuring points can be removed at the utmost. Before answering the second question the reconstruction error has to be defined. By reconstruction error δ_{io} at i_o-th measuring point the mean square departure of reconstructed value $\hat{h}_{io}$ from that measured is understood, i.e.

$$\delta_{io} = \sqrt{\frac{1}{m_h} \sum_{j=1}^{m_h} \{h_{io}(j) - \hat{h}_{io}(j)\}^2} \tag{25}$$

where:

m_h = number of historical measurements

$h_{io}(j)$, $\hat{h}_{io}(j)$ = measured and reconstructed values of i_o-th coordinates of the state vector $\underline{h}$ at j-th instant of time, $(j=1,\ldots,m_h)$

Now, the optimal reduction of an existing monitoring network will be presented in the form of algorithm. Let a primary network consists of N_o measuring points. In order to reduce the network by one, the observability criterion must be checked for all subnetworks containing N_o-1 points. Also appropriate observer systems has to be constructed for those subnetworks which are observable. For such networks a reconstruction error must be computed according to the definition (25). The measuring point having the least reconstruction error δ_{io} may be removed first. The procedure can be repeated for residual network and the next measuring point can be chosen. Obviously observer systems at the second stage are based on N_o-2 measuring points. Following the alogrithm one may remove M points from groundwater monitoring network keeping the residual network observable and reconstruction error minimal.

6 Conclusions

The optimization method presented has several properties that seem to be promising for future applications:

- it is based on the physical description of a groundwater system
- it takes advantage from existing data records
- it answers the question whether the system dynamics can be reconstructed
- it contains a mathematical procedure (the observer system) that accomplishes reconstruction of the system's state, providing specified measurements are available

- it offers a measure (the state reconstruction error) that can be used as a criterion in designing of a groundwater monitoring network

Some questions must still be answered, namely:
- how to deal with measurement errors?
- how to incorporate other sources of a groundwater system's uncertainty?

Despite these question marks the application of observer systems gives some insight into a groundwater system - measuring network relationship.

References

Bellman, R. (1960). Introduction to Matrix Analysis. Ed. McGraw-Hill Book Company, N.Y.

Dryja M. and J.M. Jankowscy (1982). Survey of numerical methods and algorithms. Part 2. Ed. Wydawnictwa Naukowo-Techniczne, Warszawa, (in Polish).

Eliasson, J. and S.P. Kjaran (1976). Reservoir mechanism analysed by the method of Galerkin and orthogonal eigenfunctions, Nordic Hydrology, 7, 1976, pp. 33-42.

Kjaran, S.P. (1977). Theoretical and numerical models of groundwater reservoir mechanism. NEA Hydraulic Laboratory Report, University of Iceland, Reykjavik, pp. 85-109.

Luenberger, D.G. (1966). Observers for multivariable systems. IEEE Trans. on Automatic Control, 1966, vol. AC-11, No. 2, pp. 190-197.

Nawalany, M. (1981). Observation of aquifer pollution state. In Quality of Groundwater, Proceedings of an Int. Symposium, Noordwijkerhout, the Netherlands, 23-27 March 1981, Eds. W. van Duijvenbooden et al., pp. 985-990.

Niederlinski, A. (1974). Multidimensional systems of control theory. Ed. Wydawnictwa Naukowo-Techniczne, Warszawa pp. 276-285 (in Polish)

Szmelter, J. (1980). Computer methods in mechanics. Ed. PWN, Warszawa, pp. 65-68 (in Polish).

Wiberg, D.M. (1971). State space and linear systems. Ed. McGraw-Hill Book Company, N.Y. pp. 128-136

DESIGN AND OPTIMIZATION OF WATER QUALITY MONITORING NETWORKS

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Abstract

In this paper a general approach for the design and optimization of monitoring networks is presented, which emphasizes the strong interrelations between monitoring objectives, process dynamics and the optimization techniques to be used. This approach is worked out for some specific techniques, based on Time Series Analysis, kriging and kalmanfiltering. Knowledge of the correlation structure of the processes is shown to be essential. Finally, some applications on field data are presented.

1 Introduction

Due to the increased concern for the environment, the monitoring effort for both surface water and groundwater has expanded significantly. Unfortunately, the resulting monitoring networks are for most part based on ad-hoc design criteria.
Limitation of the laboratory capacity, the restrictions of financial means and changing objectives, emphasize the necessity to optimize these networks. As a consequence, there is a growing need for design and optimization techniques, which should preferably integrate the optimization of sampling frequencies, sampling locations and the number and kind of water quality variables to be measured.

In this paper, the general principles of network optimization and of a number of optimization techniques, based on time series analysis, kriging and kalmanfiltering, are discussed. Finally, some applications on real field data are presented.

2 Principles of optimization

In general, a monitoring network should be based upon two main aspects:

- the water system to be monitored
- the monitoring objectives.

The water system (and the process dynamics incorporated) determine, together with the objectives, the dimensionality of the monitoring network. Moreover, the monitoring objectives strongly determine the scale of changes to be detected in the quality variables, and hence dictate the kind of information to be extracted from the measured data. This implies that the monitoring programme, including data collection and data analysis, should be tuned to these objectives. This can be expressed by the 'effectiveness' of a monitoring programme, being the degree to which the obtained information meets the objectives of that programme.
To enable an optimization, a quantitative, objective dependent, measure E of this effectiveness is required, which can be related analytically to the variables to be optimized. So, for a specified objective, a relation $E = E(f,L,V)$ has to be derived, with f denoting the sampling frequency, L the number and location of the monitoring stations, and V the number and kind of the measured variables. Since f, L and V determine the monitoring programme itself, also the monitoring costs C_M can be expressed as a function $C_M = C_M(f,L,V)$. By weighing the effectiveness against costs, the optimal monitoring programme can be found in principle.

Corresponding to the above, the optimization process for monitoring networks is summarized in Figure 1. This figure underlines the central role of the objectives and shows the iterative character of the optimization procedure: the optimal (future) network is based on information gained from the present one.
Five main steps can be distinguished:

- First, the monitoring objectives should be identified and quantified, which includes the definition of an effectiveness measure. If this step is ignored, one will be tempted to fit the optimization problem to the techniques available. In fact, this is perhaps the most important step in the optimization procedure, and also a very difficult one. A complicating factor within this respect are the different objectives various users of the monitoring network may have. Mostly, these objectives include the estimation of the present quality state, the detection of long term trends, the detection of standards violations, and model studies.

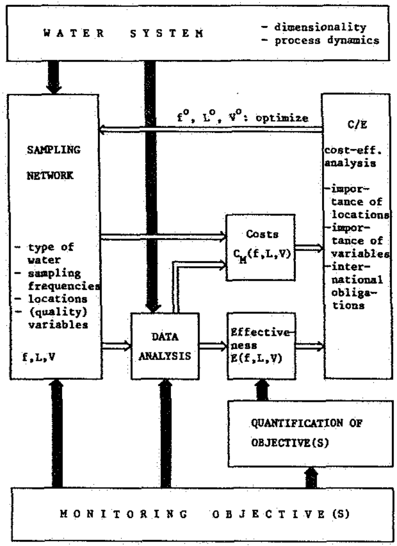

Figure 1 The optimization process

- Second, the relevant processes should be identified, because their dynamics dictate the way the data should be analysed. These dynamics depend on the water quality variables considered and the hydrodynamics of the water system.
- The third step is the determination of the effectiveness of the information, gained by analyzing the data from the monitoring network. Often, the effectiveness can be related to statistical concepts, like the variance of the samples, the probability to detect changes or violations, the interpolation error etc.
- The fourth step consists of the cost calculation of the monitoring programme. This mostly will be a purely technical matter.
- The last step includes a cost-effectiveness analysis, which is not only very difficult, but often subjective. The analysis should, amongst others, take into account the relative importance of the sampling locations and variables. In fact, such analysis is almost always avoided in practice; often a minimum level of effectiveness is specified a priori. Since in general the minimum level is based on subjective and/or political consideration, the real optimal sampling effort might not be achieved.

In the next section, some techniques will be discussed which are suitable to tackle the third step. Herein also some attention will be given to the associated problem of quantifying objectives. The other points raised above are not treated anymore because they either are of a purely technical nature (cost calculation) or belong primarily to the tasks of the management agencies (objective identification and cost-effectiveness analysis). It should be stressed that these agencies provide the designer with sufficient information on these aspects.

3 Optimization techniques

3.1 General

The effectiveness of a monitoring programme is determined by the amount of information, extracted from the data, in relation to the monitoring objectives. In general, the amount of information increases with increasing monitoring effort. The degree of increase strongly depends on the correlation structure of the physical processes being monitored. Therefore, the derivation of the function $E = E(f,L,V)$ requires knowledge about:

- the auto correlation structure of each variable in time and space
- the cross correlation structure between all variables in time and space.

In these correlations, both the process dynamics and the process dimensionality are reflected.

3.2 Time series analysis

The amount of information, which can be extracted from an ergodic one-dimensional univariate time series $x(t)$, sampled over a period of time T, is related to the so called 'effective number of independent observations' N^* (Bayley and Hammersley, 1946). This N^* is a fucntion of the sampling interval Δ, the period of observation T and the auto correlation function $\rho_x(\tau)$

$$N^*(\Delta,T) = N \{1 + 2 \sum_{i=1}^{N} (1 - \frac{i}{N}) \rho_x (i\Delta)\}^{-1} \tag{1}$$

with $N = T/\Delta$ the number of observations.
For example, the variance of an estimated mean value over a period of time T, given by

$$m_x = \frac{1}{N} \sum_{i=1}^{N} x(i\Delta) \tag{2}$$

equals

$$\sigma_m^2(\Delta,T) = \frac{\sigma_x^2}{N^*(\Delta,T)} \tag{3}$$

with σ_x^2 denoting the variance of the time series.
Moreover, the detectibility of a trend with a magnitude Tr over a period of time T is monotonically related to the statistic N_T, given by

$$N_T(\Delta,T,Tr) = \frac{Tr}{c\sigma_x} \sqrt{N^*(\Delta,T)} \tag{4}$$

with c a trendshape dependent constant (Lettenmaier, 1976).

Obviously, these relations are useful from the point of view of network optimization for one variable in <u>one</u> dimension, since they allow the derivation of the relationship of some effectiveness measure E to the sampling frequency f for fixed sampling locations and variables. This relation will be denoted by $E(f|L,V)$.
In case the objective of the network is the *estimation of mean value* (e.g. annual means), the effectiveness measure might be chosen to be the reciprocal width of the $(1-\alpha)100\%$ confidence interval for m_x, yielding

$$E(f|L,V) = \frac{\sqrt{N^*(\Delta,T)}}{\xi(\frac{\alpha}{2})\ \sigma_x} \tag{5}$$

with $f = 1/\Delta$ and $\xi(\alpha/2)$ a normal percentile point.
Likewise, for the objective *trenddetection*, the effectiveness might be defined as the trenddetectibility (or power of trenddetection), given by

$$E(f|L,V) = \Phi\{N_T(\Delta,T,Tr) - \xi(\frac{\alpha}{2})\} \tag{6}$$

where Φ denotes the standard Gaussian distribution (Lettenmaier, 1976). The expressions (5) and (6) clearly show that the quantification of these objectives corresponds to the specification of T and α in the first case and of T, α and Tr in the second case.

To evaluate the function $N^*(\Delta,T)$, the auto-correlation function $\rho_x(\tau)$ must be known rather accurately. Estimating $\rho_x(\tau)$ from historical data may cause problems, since rather long series (several times the relevant time scale) are required to reduce the statistical errors to an acceptable level. Moreover, in case of historical data with sampling interval Δ_o, the function $N^*(\Delta,T)$ can only be evaluated for discrete values of Δ, given by $\Delta_k = k.\Delta_o$, k = 1,2... Hence, only the effect of a reduction of the sampling frequency on the effectiveness can be investigated.
So, in practice, one usually assumes a (black-box) model for the correlation function, of which only the parameters have to be estimated. This imposes less demands on the record lengths, and also allows the effect of an increasing sampling frequency to be investigated. Figure 2 shows the relation between N* and N for x(t), described by a first order auto-regressive model with correlation function

$$\rho_x(k\Delta) = \rho_1^{mk} \quad (7)$$

Here ρ_1 denotes the correlation coefficient between two samples with some arbitrary time lag Δ, and $\Delta = m.\Delta_1$. For uncorrelated

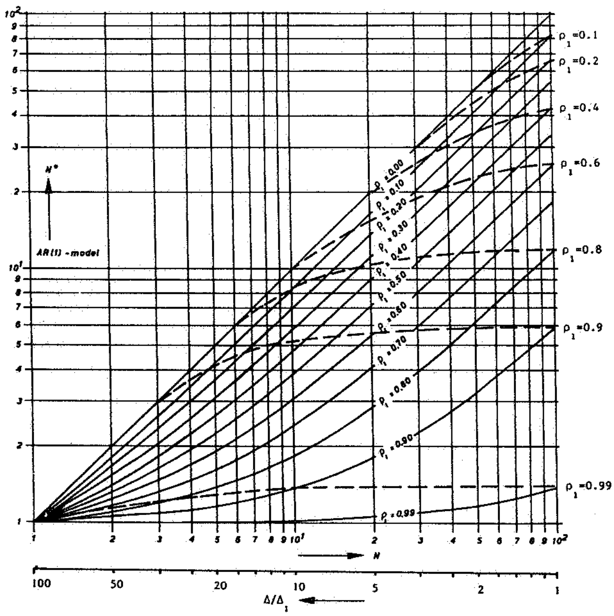

Figure 2 N^* as a function of N(-) and of Δ/Δ_1 (--) for T = 100 Δ_1 and various values of ρ_1

measurements ($\rho_1 = 0$) N* equals N. For $\rho_1 > 0$ N* becomes smaller than N. The dotted lines in Figure 2 give N* as a function of Δ/Δ_1 for $T=100\ \Delta_1$, and for various values of ρ_1. They clearly show the saturating behaviour of $N^*(\Delta)$ for decreasing Δ, indicating that the reduction of the sampling interval below some value Δ_c only results in a marginal increase in effectiveness, and hence in a waste of effort. Since this value Δ_c is large for strongly correlated processes, the sampling frequency for processes with slow dynamics often can be reduced without loss of effectiveness E.
It must be realized that N*, and hence E, has an upper bound, which decreases with increasing ρ_1. This bound might even be less than the minimal value of E as required by the objectives. In that situation, one can either try to reduce ρ_1 or to increase N. Reducing ρ_1 is possible when one can remove from the records dominant large-scale phenomena, which are due to known external sources. For instance, the correction of concentration measurements in a river for changes in discharge may improve the trend detectibility, because in this way ρ_1 may be reduces considerably. This is illustrated in section 4. The increase of N corresponds to an increase of the time to which the effectiveness measure is related. Hence, this involves a reconsideration of the objectives.

The equivalence of the auto-correlation function in time domain is the auto-spectral density function G(f) (or spectrum) in frequency domain. Since for Gaussian ergodic processes the spectral moments m_o and m_2 determine many level crossing properties, the spectral approach might be useful to optimize networks designed to *detect violation of standards*. For instance, Beckers et al. (1972) showed that the effectiveness measure , defined as

$$E(f|L,V) = \frac{\text{expected number of detected violations}}{\text{expected number of violations}}$$

can be related to the sampling interval Δ, the average non-violation duration T_o and the average violation duration T_1. Moreover, it can be shown that for Gaussian ergodic processes T_o and T_1 are uniquely related to the process level h corresponding to the standard, and to m_o and m_2 (Cramer and Leadbetter, 1967). So, knowledge about the spectrum enables the evaluation of T_o and T_1 as a function of h and hence the evaluation of the effectiveness as a function of Δ and h.

In case the objective of a monitoring network is the *reconstruction of the quality state* of the water from noisy, discrete measurements, obtained over some period of time T, an effectiveness measure might be the mean square error of the reconstructed (or interpolated) data. To relate this error to the sampling interval Δ, techniques can be used which are based on the discrete form of the Wiener-Kolmogorov filtering theory. To be more precise, suppose one wishes to estimate the state x(t) at an arbitrary time θ from noisy discrete measurements $y(k\Delta)$, given by

$$y(k\Delta) = x(k\Delta) + v(k\Delta) \qquad k = 1 \ldots N \tag{8}$$

where $N = T/\Delta$ and v(t) is a zero mean random process, describing measurement noise. Moreover, the estimate $\hat{x}(\theta)$ is restricted to be a linear function of the available data, i.e.

$$\hat{x}(\theta) = \sum_{i=1}^{N(\Delta)} \alpha_i \, y(i\Delta) \tag{9}$$

The weights α_i must be chosen such that $\hat{x}(\theta)$ is unbiased and optimal. This implies the minimization of $E\{\hat{x}(\theta) - x(\theta)\}^2$ under the restriction $E\{\hat{x}(\theta) - x(\theta)\} = 0$, which yields the following linear system for the weights α_i and the Lagrange multiplier λ.

$$\begin{aligned} &\sum_{j=1}^{N(\Delta)} \alpha_j \, \gamma_{yy}(i\Delta - j\Delta) = \gamma_{yx}(i\Delta - \theta) + \lambda \qquad i = 1 \ldots N \\ &\sum_{j=1}^{N(\Delta)} \alpha_j = 1 \end{aligned} \tag{10}$$

Here $\gamma_{yy}(.)$ and $\gamma_{yx}(.)$ denote respectively the auto-covariance function of the observations and the cross-covariance function between the state and the observations. Assuming v(t) to be a white noise process with variance σ_v^2 which is uncorrelated to the state, $\gamma_{yx}(\tau)$ equals

$$\gamma_{yx}(\tau) = \gamma_{xx}(\tau) = \gamma_{yy}(\tau) - \sigma_v^2 \; \delta(\tau) \tag{11}$$

Solving (10) yields the optimal values α_i^o and λ^o.
The corresponding optimal estimate equals

$$\hat{x}^{o}(\theta) = \sum_{i=1}^{N(\Delta)} \alpha_i^{o}\, y(i\Delta) \tag{12}$$

with minimal mean square error

$$E\{\hat{x}^{o}(\theta) - x(\theta)\}^2 = - \sum_{i=1}^{N(\Delta)} \alpha_i^{o}\, \gamma_{xx}(i\Delta - \theta) + \lambda^{o} + \sigma_y^2 - \sigma_v^2 \tag{13}$$

The inverse of this error may be used as an effectiveness measure $E(f|L,V)$. Since both (10) and (13) are independent of actual values of the measurements $y(i\Delta)$, the effectiveness, and hence the performance of the network can be evaluated as a function of Δ without use of actual measurements from that network. But again, historical data are required to estimate the correlation structure (including $\gamma_{yy}(\tau)$ and the measurement noise σ_v^2) or to fit a correlation model. Note that, since $N = T/\Delta$, one should specify within the objectives the period of time T to which the state reconstruction should be related.

The techniques described above are suitable to optimize a monitoring network for one variable in one dimension (time or one spatial direction), where the variable is assumed to be a stationary stochastic process. So, from a practical point of view, the applicability of these techniques seems to be rather limited. However, several extensions are possible.

For the simultaneous optimization of the sampling frequency for more variables, multivariate time series analysis can be used. This extension requires knowledge about all relevant auto- and cross-correlations (or auto- and cross-spectra. Sometimes nonstationary time series can be dealt with by first applying differencing operations (Box and Jenkins, 1976). Moreover, for some objectives the same techniques can be used to optimize a network in more dimensions, provided the considered variables are isotropic in those dimensions.

For instance, the extension to state reconstruction in two dimensions is rather straight forward. In fact, the equations (10) - (13) are similar to the well known Kriging equations, which originally have been derived to interpolate more dimensional geological data. The difference is that the Kriging equations are based on the (semi)variogram $\nu_x(|h|)$ in stead of covariance functions. It gives half the variance of the difference of some variable $x(\vec{r})$ for two points in a plane, separated by a distance $|h|$, i.e.

$$\nu_x(|h|) = \frac{1}{2}\,\mathrm{var}\{x(\vec{r} + \vec{h}) - x(\vec{r})\} \tag{14}$$

When $x(\vec{r})$ is isotropic, there is a direct relation between the variogram and the covariance function

$$\gamma_{xx}(|h|) = \sigma_x^2 - \nu_x(|h|) \tag{15}$$

Substituting (15) in (10) - (13) immediately yields the Kriging equations.
Kriging can be extended to variables which are not homogeneous by adopting the 'generalized intrinsic hypothesis' (Delhomme, 1978).
In essence, this is comparable with the already mentioned differencing procedure for nonstationary time series.

One can conclude that optimization techniques based on time series analysis are very suitable to solve at least parts of the optimization problem. However, some practical problems remain:

- The condition of isotropy in all dimensions seems very unlikely to be fulfilled when one considers time and space dimensions simultaneously
- Many data are required to estimate the complete correlation structure in the multivariate case
- Not every type of non-stationarity can be removed by differencing operations.

Because the multivariate correlation structure, anisotropy and non-stationarities are consequences of the underlying physical processes, these problems can (partly) be overcome by incorporating physical knowledge in the optimization process. When this knowledge can be formulated in terms of a mathematical (state space) model, techniques based on kalmanfiltering seem to offer good possibilities.

3.3 Kalmanfiltering

Information about the correlation structure is essential for the optimization of a monitoring network. This information can be obtained from observations, but can also be based on a priori knowledge of the process dynamics.

The techniques treated above only use observation based information, like covariance functions and variogrammes. A kalmanfilter can use both sources of information, since it is based on two equations

- the state equation, by which the physical knowledge is modelled
- the observation equation, which indicates the way the observations are related to the state variables.

In principle, the state equation (the model) is used to predict future values of the state vector. Each time measurements are taken, these predicted values are compared with the measurements and adjusted. The degree of adjustment depends on the uncertainties of the model ('system noise') and of the observations ('measurement noise'). In this way, the best estimate of the state vector is obtained. It is important to note that also unmeasured state variables can be estimated, because they are related to the measured ones by means of the state equations.

Apart from the estimates, also their covariance matrix is calculated. This covariance matrix, which is a measure of the reliability of the estimates, strongly depends on the measurement matrix from the observation equation. Since this matrix indicates at which moments and on which locations which variables are measured, it reflects the monitoring effort. Hence, the behaviour of the covariance matrix can be investigated for all relevant combinations of sampling frequencies, locations and variables. In the case the state equation is linear in the state variables, this even can be done without actual measurements, allowing the performance of a network to be determined a priori for different monitoring strategies. So, when the effectiveness can be related to the covariance matrix, the total optimization problem can be solved in a very elegant way. However, in practice several problems may arise.

First, most water quality models are non-linear in the state variables. Since then the covariance matrix becomes dependent on the actual state vector, the network performance cannot be evaluated a priori anymore.

Second, the dimension of the state vector, being roughly proportional to the product of the number of variables and sampling locations involved, may become too large for practical use. This situation even get worse when unknown model parameters have to be estimated simultaneously by

state augmentation.
Third, the dynamics of many water quality variables are still too poorly understood, to enable the development of a sufficiently detailed mathematical model.

In spite of these problems, the development of optimization techniques based on kalmanfiltering should be stimulated. The dimensionality problem of the state vector may be overcome when sophisticated numerical techniques are used to solve the filtering equations (Bierman, 1977). Also, much effort is put now in the development of mathematical models for water quality processes. It may be expected that due to these developments the kalmanfilter related techniques will become practical instruments to optimize complex monitoring systems.

4 Applications

4.1 Characteristics of existing monitoring networks

Processes, showing large spatial variability, generally have small spatial correlation distances. Hence, these processes require rather dense monitoring networks. Likewise, large temporal variability requires rather high sampling frequencies. These facts can be illustrated by comparing for instance the Dutch monitoring networks for ground water levels and for surface water quality.
The first one consists of about 9000 monitoring locations with a characteristic sampling interval of some months which corresponds to the large spatial and rather small temporal variability of the ground water level. The water quality network includes almost 400 sampling locations, with a typical sampling interval of one week, and also some automatic monitoring stations with a typical sampling interval of 15 minutes, corresponding to the large temporal and rather small spatial variability of most surface water quality variables.

4.2 Some applications of optimization techniques

The Delft Hydraulics Laboratory is currently involved in the optimization of the water quality network mentioned above. In first instance, attention was paid only to the optimization of the sampling frequency. The responsible management agency (the Governmental Institute of Sewage and Waste Water Treatment RIZA) formulated the objective of the network as the detection of long term trends in the quality state. The corresponding effectiveness measure was defined as trend detectibility. Quantifying this objective resulted in the requirement that trends with magnitude Tr of 20% of the mean values should be detected with a probability of 80% over a period T of 5 years. To calculate the required sampling frequencies the approach was followed as outlined in section 3.2.

First, by estimating the covariance functions and fitting of auto-regressive models of order one and two, the relation between N^* and the sampling interval Δ could be established for each of the 18 variables considered. Then, using Equation (4) and (6) with Tr, T and α specified, the effectiveness was evaluated as a function of Δ.

It turned out that for most variables the objective could not be met, due to large values of the signal variances σ_x^2 and large correlations between subsequent measurements.

For some variables this implied a reconsideration of the objective. For discharge related variables however the large auto-correlation was partly due to long term fluctuations of the discharge. Correcting the concentration measurements for discharge variations resulted in a reduction of the auto-correlation and hence in an improvement of the detectibility. This is illustrated in Figure 3 for Chloride, showing that the detectibility at Δ = 1 week increased from 25% to 95%. The required level of 80% is reached at Δ = 7 weeks.

For more detailed information on this optimization study, one is referred to the relating literature (Schilperoort (1979), Groot (1981 a,b), Schilperoort et al. (1982).

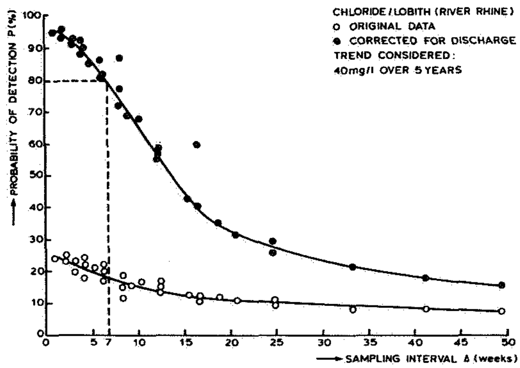

Figure 3 The detectibility as a function of Δ

In the near future, also kalmanfilter based techniques will be applied to this quality monitoring network.

A similar optimization study is executed for the two-dimensional (space) water quality monitoring network of the coastal water of the Netherlands.

5 Summary and conclusions

A general approach for monitoring network optimization is presented, which involves five steps:

- identification and quantification of objectives, including the definition of an effectiveness measure;
- determination of the relevant process dynamics;
- evaluation of the effectiveness as a function of the monitoring effort, yielding the relation $E = E(f,L,V)$;
- evaluation of the costs as a function of the monitoring effort, yielding the relation $C = C(f,L,V)$;
- execution of a costs-effectiveness analysis.

The techniques to be used in the third step strongly depend on the monitoring objectives, indicating that a careful definition of these objectives is a prerequisite for any optimization. A close co-operation between management agency and network designer therefore is essential.
For the application of these techniques, knowledge of the correlation structure of the processes is required. When this knowledge is based on observations only, black-box techniques like time series analysis and Kriging are appropriate. When, apart from the observations, also mathematical models of the processes are available, kalmanfilter based techniques can be used. With these techniques sampling frequencies, locations and variables can be optimized simultaneously, whereas the black-box techniques are suitable to solve only parts of the optimization problem.
The development of kalmanfilter based techniques should therefore be stimulated.

References

Bayley, G.V., Hammersley, J.M. 1946. The 'effective' number of independent observations in an auto-correlated time series. J. Royal Stat. Soc. vol. 8, no. 1-B, pp. 184-197.

Beckers, C.V., Chamberlain, S.G., Grimsrud, G.P. 1972. Quantitative methods for preliminary design of water quality surveillance systems. Report EPA-R5-72-001. U.S. Environmental Protection Agency, Washington.

Bierman, G.J. 1977. Factorization methods for discrete sequential estimation. Ac. Press, New York.

Box, G.F.P., Jenkins, G.M. 1976. Time series analysis: forecasting and control. Holden-Day, San Francisco.

Cramer, H., Leadbetter, M.R. 1967. Stationary and related stochastic processes. Wiley, New York.

Delhomme, J.P. 1978. Kriging in the Hydrosciences. Adv. Water Res., vol. 1, no. 5, pp. 251-266.

Groot, S. 1981. The optimization and structuring of the water quality monitoring network in the national surface waters. Part II: Application of the method due to Lettenmaier Report R1391-2 (in Dutch). Delft Hydraulics Laboratory, Delft, The Netherlands.

Groot, S.1981. The optimization and structuring of the water quality monitoring network in the national surface waters. Part III: Preprocessing of data. Report R1391-3 (in Dutch). Delft Hydraulics Laboratory, Delft, The Netherlands.

Lettenmaier, D.P. 1976. Detection of trends in water quality data from records with dependent observations. Water Resources Research, vol. 12, no. 5, pp. 1037-1045.

Schilperoort, T. 1979. The optimization and structuring of the water quality monitoring network in the national surface waters. Part I: A literature survey of optimization techniques for the sampling frequency. Report R1391-I (in Dutch). Delft Hydraulics Laboratory, Delft, The Netherlands.

Schilperoort, T., Groot, S., Van de Wetering, B.G.M., Dijkman, F. 1982. Optimization of the sampling frequency of water quality monitoring networks. Publication 261. Delft Hydraulics Laboratory, Delft, The Netherlands.

REDUCTION OF A GROUND-WATERLEVEL NETWORK

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Abstract

A practical prediction algorithm has been developed to discriminate between independent or primary locations and dependent or secondary locations of a groundwaterlevel observation network. The selected criterium is an objective measure: a value of 15 cm for the root mean squared error of the prediction error in the model validation period. The algorithm is based on the Kalman filtering technique. After an introduction of the theoretical model the link will be made between the theoretical model and its practical use for network reduction. Four applications directed to network density and observation frequency will be described. This paper is solely concerned with reduction of a network. Network reinforcement at unmeasured sites will be discussed in the paper: Spatial uncertainty in piezometric head.

1 Network reconsideration strategy

Since the foundation of the Archive of Groundwater levels in the Netherlands in 1948 a total number of 7 million groundwater levels have been collected. The groundwater level network consists of 8000 shallow wells and 8000 deep wells. The strategy for this network development has always been in the category "we need more data than we have". This is justified as long as basic data is lacking for verification of some kind of criterium for the development of the network. As pointed out by

Rodriguez-Iturbe (1972) network optimization for a base-level network can be hardly fulfilled due to lack of information with regard to economic development. Nevertheless, criteria should be formulated in order to approach some kind of homogeneity in the network. A statistical approach should be favoured. Logically and in accordance with the WMO criterium (1970) for hydrological networks the standard deviation of the interpolation error can be used as the criterium.
Two types of interpolation errors for a groundwater level network should be considered.

a) interpolation error in space
b) interpolation error in time and space

An estimate of the interpolation error in space can be obtained with the kriging technique (Delfiner, 1976). This technique will be relevant when questions should be answered with regard to the most favourable location for a new observation site.
The interpolation error in time and space is relevant when answers are needed with regard to observation frequency at existing observation locations. Indirectly this is also related to network density: when observation interval approaches infinity and the standard deviation of the interpolation error is still less then a predefined maximum value, this observation site can be skipped from the base level network.
With regard to type b) interpolation it will be shown that in some situations the Kalman interpolation technique can be used.

2 Theory of Kalman interpolation

The basic idea of Kalman interpolation will be introduced first for the one-dimensional case, subsequently the higher order dimensionality will be discussed.

One dimensional case.
When a time series of groundwater levels is given the following relations are fundamental to the Kalman filtering technique. The groundwaterlevel at time k+1 is supposed to be linear dependent on the groundwater level at time k:

$$x_{k+1} = h\, x_k + u_k \tag{1}$$

where h is a transition factor and u_k a stochastic variable with expectation 0 and variance $\sigma_u{}^2$. Relation (1) will be referred to as the state transition equation.

The second basic equation will be called the measurement equation:

$$y_k = s_k \;\; x_k + v_k \tag{2}$$

where y_k is the measured groundwater level at time k, being dependent on the true groundwater level x_k at time k.

s_k is called the measurement factor which is usually taken to be 1. The measurement error v with expectation 0 and variance σ_v^2 is assumed to be uncorrelated with u.

The Kalman filter is designed to provide an optimal estimate of x_{k+1}, based on data observed up to time k+1 in a recursive way.

As an estimate for x_{k+1}, before a measurement at time k+1 has been made we take $h\hat{x}_k$, where $\hat{x}_k$ is an estimate for x_k from the previous step. After a measurement at time k+1 has become available we estimate the groundwater level as a weighted mean:

$$\hat{x}_{k+1} = (1-w_{k+1})h\; \hat{x}_k + w_{k+1}y_{k+1} \tag{3}$$

An optimal estimate of the groundwater level at time k+1 is found if the weighting factor is chosen such that the variance of $\hat{x}_{k+1}$ is minimized.

We could write $x^*_{k+1} = h\hat{x}_k$

From equation A6 in the appendix it follows that:

$$w_{k+1} = \sigma^2_{x^*_{k+1}} \;/(\sigma^2_{x^*_{k+1}} + \sigma_v^2\,) \tag{4}$$

and (see equation 3 and A7):

$$\hat{x}_{k+1} = x^{*}_{k+1} - (h\,\hat{x}_k - y_{k+1})\,(\sigma^2_{x^{*}_{k+1}})\,/\,(\sigma^2_{x^{*}_{k+1}} + \sigma^2_v) \tag{5}$$

$$\hat{\sigma}^2_{\hat{x}_{k+1}} = \sigma^2_{x^{*}_{k+1}}\,\sigma^2_v\,/(\sigma^2_{x^{*}_{k+1}} + \sigma^2_v) \tag{6}$$

The impact of equation 5 and equation 6 can be illustrated as follows. Suppose the groundwater level at time k+1 is measured accurately so the variance of the measurement error σ^2_v is relatively small. The factor w_{k+1} then approaches the value 1. According to equation (3) the estimate is largely determined by the measurement y_{k+1} and for a small part by the prediction of the groundwater level according to equation 1.

Multidimensional case.

The basic theory can be extended to n time series and is commonly called the n-dimensional case. Equation 1 and equation 2 can be written in matrix notation:

$$X_{k+1} = HX_k + U_k \text{ and}$$

$$Y_k = S_k\,X_k + V_k \text{ whereas:}$$

$$\hat{X}_{k+1} = H\hat{X}_k - W_{k+1}\,(H\,\hat{X}_k - X_{k+1}) \text{ or with}$$

$$H\hat{X}_k = X^{*}_{k+1} \text{ and } W_{k+1} = K_{k+1}\,S_{k+1}$$

$$\hat{X}_{k+1} = X^{*}_{k+1} - K_{k+1}(S_{k+1}X^{*}_{k+1} - Y_{k+1}) \tag{7}$$

$$\text{and } P_{k+1} = P^{*}_{k+1} - K_{k+1}S_{k+1}P^{*}_{k+1} \tag{8}$$

The derivation of K_{k+1} such that $\hat{X}_{k+1}$ has minimal variance is given in the appendix. Analogous to the one-dimensional case H is called the transition matrix (nxn); X the groundwaterlevel vector (nx1), K the Kalman gain matrix (nxm), S the measurement matrix (mxn), Y the measured groundwater level vector (mx1), P_{k+1} and P^{*}_{k+1} are the variance-covariance matrices of $\hat{X}_{k+1}$ and X^{*}_{k+1} respectively.

We now can recursivily estimate the groundwater level in the following way. Q and R are the VC-matrices of respectively the system noise U and the measurement error V.

1. $X^*_{k+1} = H\hat{X}_k$

2. $P^*_{k+1} = H\hat{P}_k H^T + Q$

3. $K_{k+1} = P^*_{k+1}\ S^T_{k+1}(\ S_{k+1} P^*_{k+1} S^T_{k+1}\ +\ R_{k+1})^{-1}$

4. $\hat{X}_{k+1} = X^*_{k+1} - K_{k+1}\ (S_{k+1}\ X^*_{k+1}\ -\ Y_{k+1})$

5. $\hat{P}_{k+1} = P^*_{k+1} - K_{k+1}\ S_{k+1}\ P^*_{k+1}$ (9)

The recursive equations (9) assume that the matrices H,S,Q and R are known. In our applications R will be a diagonal matrix with the variances of the measurement error as its elements. The matrix S is an mxn matrix with zero's and one's, m being the number of measured groundwater levels. The matrices H and Q are unknown and will be estimated from the available records in the calibration period, that is the period (2 or 3 years) preceding the period where the Kalman technique is actually applied. For the estimation procedure, we follow Van der Made (1979). For the calibration period S=I, because all the relevant groundwater levels should be measured during that period. By assuming an autoregressive process for Y_k (no correlation between U_k and U_{k-1}) the transition matrix H can be estimated by means of a regression of Y_k on Y_{k-1}. The variance-covariance matrix of the (multivariate) residuals is then an estimate of Q.
Van der Made showed that, if all the series are "prewhitened" such that the mean equals zero, these two estimates can be expressed as a function of the sample correlation coefficients and standard deviations, for example with 2 series $y_1(k)$ and $y_2(k)$ the estimate of H is a function

of $\rho_{y_1(k-1), y_1(k)}$, $\rho_{y_2(k-1), y_2(k)}$, $\rho_{y_1(k), y_2(k)}$, $\rho_{y_1(k), y_2(k-1)}$

and $\rho_{y_2(k-1), y_2(k)}$:

$$\hat{H} = \left[\begin{bmatrix} 1 & \rho_{y_1(k),y_2(k)} \\ \rho_{y_1(k),y_2(k)} & 1 \end{bmatrix}^{-1} \begin{bmatrix} \rho_{y_1(k-1),y_1(k)} & \rho_{y_1(k-1),y_2(k)} \\ \rho_{y_1(k),y_2(k-1)} & \rho_{y_2(k-1),y_2(k)} \end{bmatrix} \right]^T$$

A similar expression can be given for Q.

As mentioned earlier, all the groundwater levels are measured during the calibration period in order to estimate the unknown parameters. For the application-period, it is not necessary that all the measurements are available.

3 Practical network considerations

In the following some examples of the Kalman methodology for actual data and practical network considerations will be given. For all case studies a measurement error of 1 cm is assumed.

3.1 Observation frequency

For three time series the correlation structure in a calibration period (2 years, 48 observations) was estimated in three different areas in the Netherlands. A scheme of the extrapolation is shown in Figure 1.

Only for the second time series in each area the relation between the standard deviation and lead time is visualized. The first and third time series for estimating the extrapolation error in the second time series, are recorded in each area within 8 km distance. Extrapolating the original series results in a small reduction of the standard deviation compared with the standard deviation in the original series, even if the groundwater level is predicted only two weeks in advance (Figure 1, left),

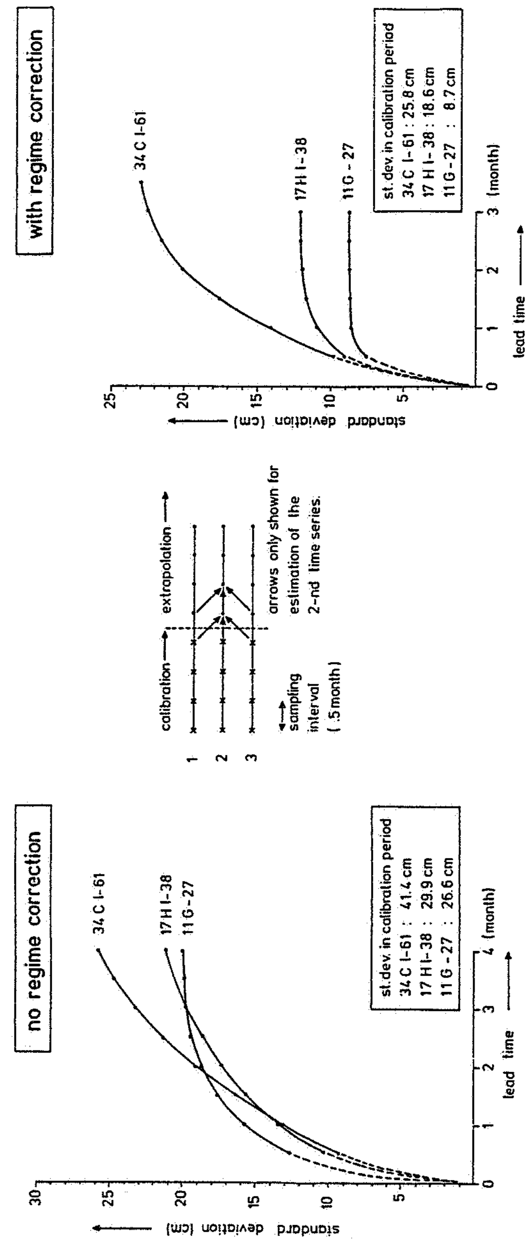

Figure 1 Extrapolation errors for the indicated prediction scheme related to lead time for three aquifers in the Netherlands and the effect of regime correction

The observations are also corrected with the means of the observations measured at the same date during the calibration period -for each series seperately- : called regime correction. With regime correction lower extrapolation errors are found, notably for 11 G 27 and 17 H-1-38 (Figure 1, right). Which method works best can only be investigated in an objective way by comparing observed values and predicted values.

3.2 Reducing existing network density

With three time series the second time series could also be predicted according to the scheme in Figure 2.

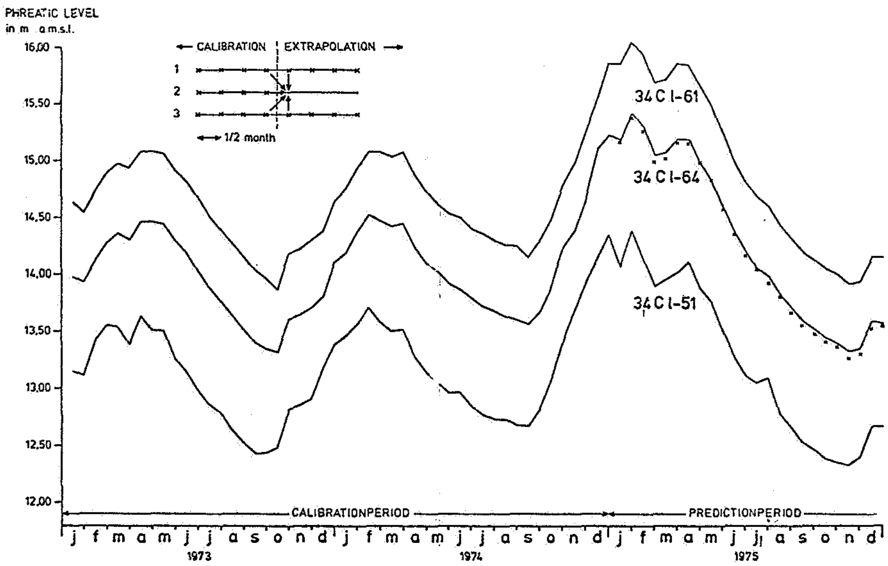

Figure 2 The observed groundwater levels of three observation locations and the predicted groundwater levels according to the extrapolation scheme

In this figure also the measured groundwater levels in the calibration period and prediction period are shown. After the regime correction the Kalman algorithm was applied. As was expected for the quite similar time series a rather good predictor could be derived from the data. The theoretical standard deviation for the predicted groundwater level in 1975 amounts to 1.2 cm, slightly more than the assumed standard deviation of the measurement error of 1 cm. The root mean squared error (rmse) of the observed minus the predicted values is 4.1 cm; This is due to a systematic underestimation of 3.4 cm in the relatively wet year 1975.

3.3 Network density versus observation frequency

For a theoretical comparison of two interpolation schemes (Figure 3)

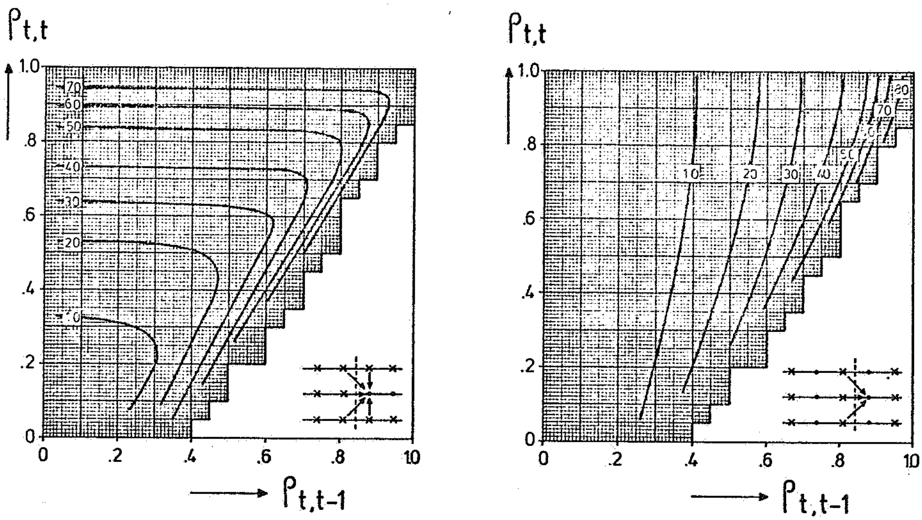

Figure 3 The theoretical percentage reduction in the standard deviation of the original time series as a function of the cross correlation $\rho_{t,t}$ and autocorrelation $\rho_{t,t-1}$

the following assumptions are made:

- correlations for three time series with a time shift are all equal ($\rho_{t,t-1}$)

- correlations without time shift are also equal ($\rho_{t,t}$), but not necessarily $\rho_{t,t}=\rho_{t,t-1}$

The percentage reduction in standard deviation due to Kalman filtering of the original series is given in Figure 3. In the right hand part of both figures the area is shaded for physically impossible combinations of $\rho_{t,t}$ and $\rho_{t,t-1}$. From these figures a more rapid reduction in the standard deviation for higher correlations in time and space is remarkable. Comparing the two sampling schemes a more flexible approach is obtained when the left hand sampling scheme is used.

Depending on the actual correlations found in groundwater level time series and for a predefined reduction these kind of figures may be of help for choosing an adequate sampling scheme.

3.4 Network reduction in the Achterhoek

For actual groundwater level data measured during a period of five years with an observation interval of two weeks the interpolation scheme according to Figure 3 , left was applied. The phreatic aquifer considered is the main aquifer in this area. The total thickness varies from less than 5 m in the eastern part to 40 m in the western part of the study area (Ernst, 1970).

The calibration period 1974-1976 was choosen because different climatoligical years are represented in this period. The years 1977, 1978 were taken for the application period.

The original time series y_1, y_2, y_3 are prewhitened as follows:

$$x_{i,k} = y_{i,k} - 1/72 \sum_{k=1}^{72} y_{i,k} \qquad k=1,\ldots,120;\ i=1,2,3$$

$$\text{let } x_l^* = \frac{1}{9} \sum_{i=1}^{3} (x_{i,l} + x_{i,24+l} + x_{i,48+l}) \qquad l=1,\ldots,24$$

$$\text{then } x_{i,k}^* = x_{i,k} - x^*_{(k-1)\bmod(24)+1} \qquad k=1,\ldots,120;\ i=1,2,3$$

With the aid of Kalman filtering the observation locations will be devided into two groups so called primary and secondary locations.

The groundwater level at secondary locations should be predicted from measured groundwater levels at primary locations within a predefined criterium for the interpolation error. As there are a large number of locations in the region and there is a freedom to choose for each secondary locations the two predicting locations, an efficient algorithm is needed. The following considerations have led to the search algorithm presented in Figure 4.

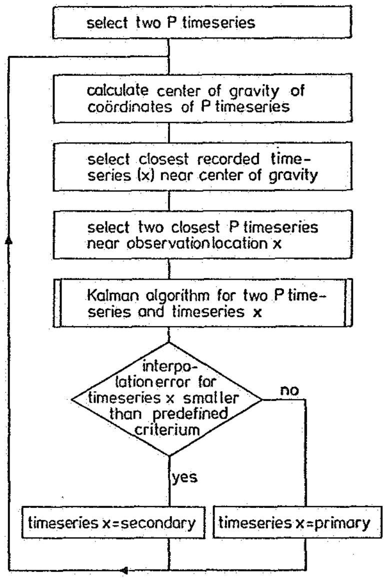

Figure 4 The selection algorithm for primary (P) and secondary time series

- A long search for "as many possible" secondary locations can not be recommended. Even with random observations such a search will result in high correlations; and thus many secondary locations.
- A secondary location and its two predicting primary locations should be not far away

For those observation locations with more than one observed time series corresponding to screens at different depths each time series was analysed seperately. As there may be a change in mean level, correlation structure and variation, a model validation is warranted.

The theoretical standard deviation therefore was compared with the rmse of the observed minus the calculated groundwater levels in the application period of two years (Figure 5a).

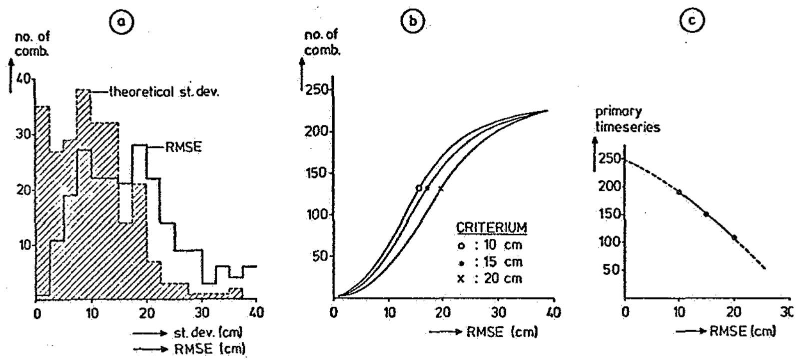

Figure 5a The distribution of the theoretical standard deviation and the rmse for a criterium of 15 cm for the rmse

5b The cumulative distribution of the rmse for three values of the rmse

5c The relation between the number of primary time series and the rmse

The rmse was found to be substantially higher than the theoretical standard deviation. This is merely due to a non-zero systematic error per combination. However the mean systematic error for all 249 combinations is only 1 cm. This low value supports the choice of the application period. But more over as a realistic criterium a specific value for the rmse should be preferred. The results for three rmse values are shown in Figure 5b.

For a criterium of 10 cm and thus more close observations locations better predictive qualities are obvious as the rmse is generally lower. Comparing the rmse and the number of primary time series a linear relation seems to be indicated in the Achterhoek (Figure 5c).

The resulting distribution of primary and secondary time series with a criterium of 15 cm is only partly related to the existing network-density (Figure 6).

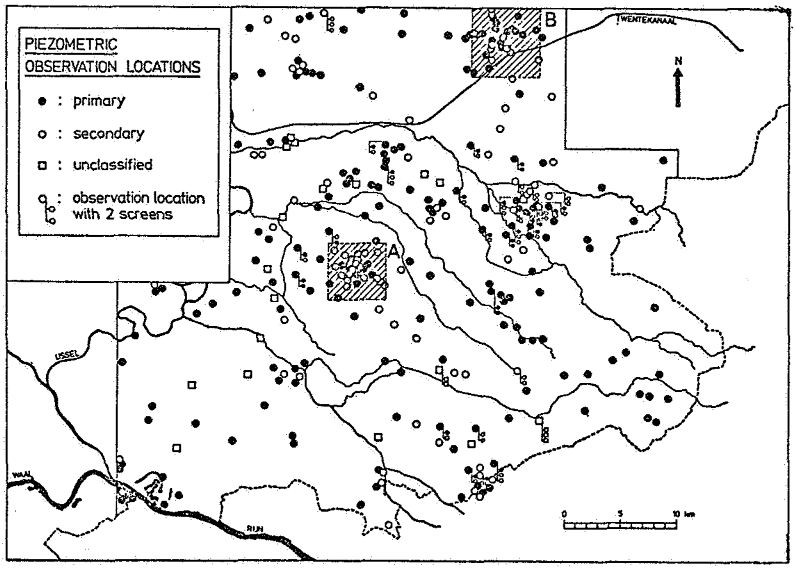

Figure 6 The distribution of primary and secondary observation locations in The Achterhoek with a criterium of 15 cm for the rmse

This distribution reflects also the complexity of the studied aquifer. This is in particular obvious for the clusters A and B in Figure 6. Near A the aquifer is rather homogeneous whereas near B an ice pushed ridge is present.

4 Conclusions

The Kalman interpolation technique can be considered as a useful method for groundwater level network reduction based on an objective criterium.

Possible instabilities in the statistical properties of groundwater level time series lead to a criterium based on the actual difference between predicted and measured groundwater levels in the model validation period: the root mean squared error (rmse)

The number of primary locations in a reduced network in the Achterhoek is linear dependent on the rmse within a range of 10 and 20 cm for the rmse.

Monthly or longer observation intervals result in large prediction errors. Consequently the standard observation interval of two weeks in the Achterhoek should be regarded as a maximum.

References

Barham, P.M. and D.E. Humphries (1970). Deriviation of the Kalman filtering equations from elementary statistical principles. In: Theory and Applications of Kalman Filtering. Advisory Group for Aerospace Research and Development, Agardograph 139 Nato. Ed. C.T. Leondes. pp. 45-49

Delfiner, P. (1976), Linear estimation of non-stationary spatial phenomena. In Advanced Geostatistics in the mining industry. Ed. M. Guarascio D. Reidel, Dordrecht, Holland pp. 49-68

Ernst, L.F., N.A. de Ridder, J.J. de Vries (1970), A geohydrologic study of East Gelderland. Geologie en Mijnbouw Vol. 48 no. 6 pp. 457-488

Made, J.W. van der (1979), Gebruik van Kalman filters voor controle van waterstanden Nota WW-WH 79.02 Rijkswaterstaat, Directie Waterhuishouding en Waterbeweging, The Hague, the Netherlands

Rodriguez-Iturbe I. (1972), General report. International Symposium on uncertainties in hydrologic and water resource systems. Tucson.

WMO (1970, Guide to hydrometeorological practices WMO-no. 168, TP. 82

APPENDIX
WEIGHTING OF INDEPENDENT ESTIMATES

The derivation of the basic equations of the Kalman filter model follows closely Barham and Humphries (1970)

For the one-dimensional case:

Suppose x_1 and x_2 are two independent estimates of a quantity x. The variances are respectively σ_1^2 and σ_2^2 .

We now want a linear combination of x_1 and x_2 so that:

$$E(\hat{x}) = x \quad \text{if} \quad E(x_1) = E(x_2) = x \text{ and}$$

$$E\left[\{\hat{x} - E(\hat{x})\}^2\right] \text{ is minimal}$$

with:

$$\hat{x} = (1-w)\, x_1 + w\, x_2 \tag{A1}$$

it follows that:

$$E(\hat{x}) = (1-w)\, E(x_1) + w\, E(x_2) \tag{A2}$$

The variance σ^2 of $\hat{x}$ is then:

$$\begin{aligned}\sigma^2 &= E\left[\{\hat{x}-E(\hat{x})\}^2\right] = \\ &E\left[\{(1-w)x_1+wx_2-(1-w)\, E(x_1)-w\, E(x_2)\}^2\right] = \\ &(1-w)^2\{x_1-E(x_1)\}^2+w^2\{x_2-E(x_2)\}^2+2w(1-w)\{x_1-E(x_1)\}\{x_2-E(x_2)\} = \\ &(1-w)^2\, \sigma_1^2+w^2\, \sigma_2^2\end{aligned} \tag{A3}$$

because:

$$E\left[\{x_1 - E(x_1)\}\ \{x_2 - E(x_2)\}\right] = 0 \tag{A4}$$

The value of w for minimum variance σ^2 is found by differentiating:

$$\frac{\delta\sigma^2}{\delta w} = -2(1-w)\ \sigma_1^2 + 2w\ \sigma_2^2 = 0 \tag{A5}$$

'and then:

$$\hat{w} = \frac{\sigma_1^2}{\sigma_1^2 + \sigma_2^2} \quad \text{and} \tag{A6}$$

$$\sigma^2 = \frac{\sigma_1^2\ \sigma_2^2}{\sigma_1^2 + \sigma_2^2} \tag{A7}$$

Multidimensional case:
The quantity to be estimated is now an n-dimensional vector. X_1 and X_2 are also n-dimensional vectors.
The elements of vector X_1 can be thought of as estimates of the groundwater levels at n locations at time t and the vector X_2 of elements consisting of measured groundwater levels at the same n locations at time t.
According to equation (A1):

$$\hat{X} = (I-W)X_1 + WX_2$$

or

$$\hat{X} = X_1 - W\ (X_1 - X_2) \tag{A8}$$

W is nxn weighting matrix and I is the identity matrix.
When the number of measurements is smaller than n we could write $Y = SX_2$. S will be now a mxn weighting matrix. A matrix K is now introduced, so that: W = K.S and

$$\begin{aligned}\hat{X} &= X_1 - K\ (SX_1 - Y)\\ &= (I-KS)\ X_1 + KY\end{aligned} \tag{A9}$$

The variance-covariance (VC) matrices of X_1 and Y will be denoted by P^* and V.

By definition the VC-matrix of $\hat{X}$ is:

$$\hat{P} = E\left[\{\hat{X} - E(\hat{X})\}\ \{\hat{X} - E(\hat{X})\}^T\right] \tag{A10}$$

From equation (A9) and (A10) we write:

$$\begin{aligned}\hat{P} = {} & E\left[\{(I-KS)X_1 + KY - (I-KS)\ E\ (X_1) - KE(Y)\}\ .\right.\\ & \left.\{(I-KS)X_1 + KY - (I-KS)\ E\ (X_1) - KE(Y)\}^T\right] = \\ & (I-KS)\ E\left[\ \{X_1 - E(X_1)\}\ \{X_1 - E(X_1)\}^T\right]\ (I-KS)^T + \\ & KE\left[\ \{Y - E(Y)\}\ \{Y - E(Y)\}^T\ \right]K^T\end{aligned} \tag{A11}$$

because X_1 and Y are uncorrelated.

In short-hand notation:

$$\begin{aligned}\hat{P} &= (I-KS)P^*\ (I-KS)^T + KVK^T \\ &= P^* + (KS)\ P^*\ (KS)^T - (KS)P^* - P^*(KS)^T + KVK^T \\ &= P^* + K(S\ P^*\ S^T + V)\ K^T - K(S\ P^*) - (S\ P^*)^T K^T\end{aligned} \tag{A12}$$

In analogy to the one dimensional case we choose K so that the sum of diagonal elements of P is minimal:

$$\begin{aligned}&\frac{\partial}{\partial K}\ \text{trace}\ \hat{P} = 0 \ \text{or} \\ &2K\ (S\ P^* S^T + V) = 2\ P^*\ S^T\ \text{or} \\ &K = P^*\ S^T\ \left[S\ P^*\ S^T + V\right]^{-1}\end{aligned} \tag{A13}$$

So X could be calculated according to (A9). Equation (A12) and equation (A13) give :

$$\hat{P} = P^* - K\ S\ P^* \tag{A14}$$

SPATIAL UNCERTAINTY IN PIEZOMETRIC HEAD

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Abstract

Uncertainties in spatial interpolation of piezometric levels depend on the variability of the piezometric surface and geohydrological boundary values and parameters. According to recently published opinions optimal interpolation of piezometric levels by the kriging method is considered to be a potential tool for network reinforcement.
In our opinion too a practical model for reinforcement of a country wide network has to be applied. In this study a complex geohydrological system and piezometric levels will be modelled.
The parameters of the kriging model were found to be time-independent for the piezometric levels of an aquifer below a loam-sandy toplayer in the south-eastern part of Friesland.
Therefore the kriging method was applied to the mean piezometric levels in a year. The geohydrological system was modelled by a quasi three dimensional flow equation. The steady state differential equation was solved numerically by the finite element method. Again kriging was applied, but now on the differences between the calculated and observed piezometric levels. The accuracy of piezometric levels for both methods was estimated at 9 independent observation sites. An equal performance for both methods was obtained. The reliability of the standard deviation of the interpolation error seems to be better when modelling the geohydrological system according to the deterministic-stochastic approach.

1 Introduction

Since the foundation of the Archive of Groundwater levels TNO in the Netherlands in 1948 a total number of 7 million groundwater levels have been collected. The groundwater level network consists of 8000 shallow wells and 8000 deep wells. The strategy for this network development has always been in the category "we need more data than we have". This is justified as long as basic data is lacking for verification of some kind of criterium for the development of the network.

As pointed out by Rodriguez-Iturbe (1972) network optimization for a base-level network can be hardly fulfilled due to lack of information with regard to economic development. Nevertheless, criteria should be formulated in order to approach some kind of homogeneity in the network. A statistical approach should be favoured. Logically and in accordance with the WMO criterium (1970) for hydrological networks the standard deviation of the interpolation error can be used as the criterium The predefined value for this standard deviation can be made region dependent and could be based on financial budgets.

Two types of interpolation errors for groundwater levels can be distinguished: interpolation error in time and interpolation error in space. In this article the interpolation error in space will be dealt with. Interpolation error depends on the interpolation technique used. Considering the huge amount of observation locations in the Netherlands we should search for a method which requires a reasonable amount of data preparation and computer time. Furthermore the methodology should be accepted and applied by a wide range of potential users of the groundwater level network. An optimal interpolation technique in space is the kriging technique. Input requirements for the method are restricted to only the coordinates and the observations. It is believed that the kriging technique should be investigated for practical adjustments in groundwater level networks. An important question is how this stochastical model performs with respect to a physical based model. Both models were tested in the south-eastern part of the province Friesland in the Netherlands.

2 The kriging technique

The kriging method is based on the theory of intrinsic random functions (Matheron, 1973). The basic methodology and practical applications have been described by many authors (e.g. Delhomme, 1978, Gambolati and Volpi, 1979a, 1979b, Delfiner, 1976, Hughes and Lettenmaier, 1981).

The general problem can be stated as follows: given a number of locations x_1 to x_n with measured groundwater levels $z(x_1)$ to $z(x_n)$. We want an estimate of the groundwaterlevel $z(x_o)$ at location x_o, such that this estimate is:

a) unbiased

b) with minimum variance.

We will restrict ourselves to estimates that are a linear combination of the data:

$$\hat{z}(x_o) = \sum_{i=1}^{n} \lambda_i z(x_i)$$

The problem is to find the weights λ_i.

The statistical model assumes that the piezometric surface is one realization of a stochastic proces $Z(x)$. Depending on the statistical properties of this proces, the optimal weights can be obtained. Without going into detail, it can be shown that these weights depend on:

a) the form of the drift (= the expection of $Z(x)$)

b) The shape and form of a function $K(h)$, where h is a vector.

In practice it is usually assumed that the drift is a polynomial of order k with $k \leqslant 2$. Appropriate models for the generalized covariance $K(h)$ are also polynomials of order $\leqslant 2k + 1$. If we assume isotropy ($K(h)=K(|h|)$) the following models are suggested by Matheron:

Drift	k	Function
Constant	o	$K(\|h\|) = c\ \delta(\|h\|) + a_1 \|h\|$
Linear	1	$K(\|h\|) = c\ \delta(\|h\|) + a_1 \|h\| + a_3 \|h\|^3$
Quadratic	2	$K(\|h\|) = c\ \delta(\|h\|) + a_1 \|h\| + a_3 \|h\|^3 + a_5 \|h\|^5$

with $\delta(|h|) = 1$ for $|h| = 0$ and $\delta(|h|) = 0$ otherwise, $c \geqslant 0$, $a_1 \leqslant 0$, $a_5 \leqslant 0$, $a_3 \geqslant -\,{}^{10}\!/_3\ (a_1 a_5)^{\frac{1}{2}}$

Given k and $K(|h|)$ it is a matter of simple mathematics to calculate the weights λ_i and also the variance of the interpolation error. In practice however k and $K(|h|)$ are unknown and we have to estimate them from the data. The estimation procedure suggested by Delfiner (1976) has been

generally accepted as reasonable. The algoritm consists of three steps. In the first step the degree of the drift is determined. Delfiner suggests to krige some known points using $K(|h|) = -|h|$ assuming in turn that k=0,1,2 but using the same neighbouring points for a typical point with all three orders.
For each point the interpolation errors are ranked. The order with the smallest mean rank is chosen as the best. One reason to justify such an approach is that this generalized covariance is a valid function for each k and also because the weights λ_i are independent of the coefficient a_1, so we can choose an arbitrary value, in this case -1.
The second step is the estimation of the unknown coefficients of $K(|h|)$. By assuming some of the coefficients zero a priori there are a number of appropriate generalized covariance models. For instance if k=1:

$$K(|h|) = c\delta(|h|)$$
$$K(|h|) = a_1|h|$$
$$K(|h|) = a_3|h|^3$$
$$K(|h|) = c\delta(|h|) + a_1|h|$$
$$K(|h|) = c\delta(|h|) + a_3|h|^3$$
$$K(|h|) = a_1|h| + a_3|h|^3$$
$$K(|h|) = c\delta(|h|) + a_1|h| + a_3|h|^3$$

The coefficients of $K(|h|)$ can be estimated by constructing so-called generalized increments with respect to the data. These increments are linear combinations of the data that are stationary under the assumed statistical model. Let the linear combination be $\sum_{i=1}^{p} \mu_i Z_i$ then the theoretical variance is equal to $\sum_{i=1}^{p} \sum_{j=1}^{p} \mu_i \mu_j K(i,j)$ which is still a linear function of the unknown coefficients of $K(|h|)$. By forming many generalized increments and performing a regression of the observed variances on the theoretical variances, estimates for the unknown coefficients can be obtained.
In the third step, the best fitting generalized covariance function is selected. Given the order k, all the appropriate generalized covariance functions can be compared in the following way. Each of the data points is left out in turn and kriged with the neighbouring points. It is then possible to obtain an estimate of the ratio V_o/V_k, that is the observed variance (= the square of the interpolation error) versus the kriging

variance. Because the estimator Σ((interpolator error)^2 / kriging variance)/n is biased, Delfiner suggests to split up the data into two groups and then to use a jackknife estimator. The generalized covariance with the jackknife estimate closest to unity is selected as the best. For a more detailed description of Delfiner's procedure the reader is referred to Delfiner (1976), Hughes and Lettenmaier (1981) and Kafritsas and Bras (1981). The last reference also contains a Fortran listing of a computerprogram, called AKRIP. This program enables the user to perform all the above mentioned calculations. We have used AKRIP in the follwing case-study.

3 Practical results

The kriging method was applied for an aquifer below a loamy-sandy toplayer (Figure 1). A total number of 43 wells was selected whereas 9 wells were used only for controlling model results. For each well the mean yearly piezometric head in 1970 was calculated from 24 observations. AKRIP could not discriminate between k=0 and k=2 (average rank for k=0,1,2 is 1.98,2.05 and 1.98).We decided to krige with k=0 and 8 neighbouring points. This choice implies three possible generalized covariance (GC) functions. The second best was found to be $K(|h|) = -0.4487|h|$. The jackknife estimate of the ratio V_δ/V_k for this fit equals 0.94. It also turned out that the GC: $K(|h|) = c\delta(|h|)$ performed even better (c= 5825, jackknife = 1.00). This GC gives equal weight to each location independent of the distance to the location to be kriged. We decided not to use this function because we assume, considering the geohydrology of the area, that information from neighbouring points is more valuable than information from locations further away.
The kriged values, with $K(|h|) = -0.4487|h|$ at nodes 2500 m apart are contoured (Figure 1). The estimate standard deviation of the interpolation error has also been visualized.
The maximum values for the standard deviation occur, as can be expected with the selected GC, at locations where the distance to surrounding observations wells is relativily large.
In order to check the validity of the model the kriged value and its standard deviation were calculated at the 9 control points. For these points a mean non-significant difference between measured and kriged

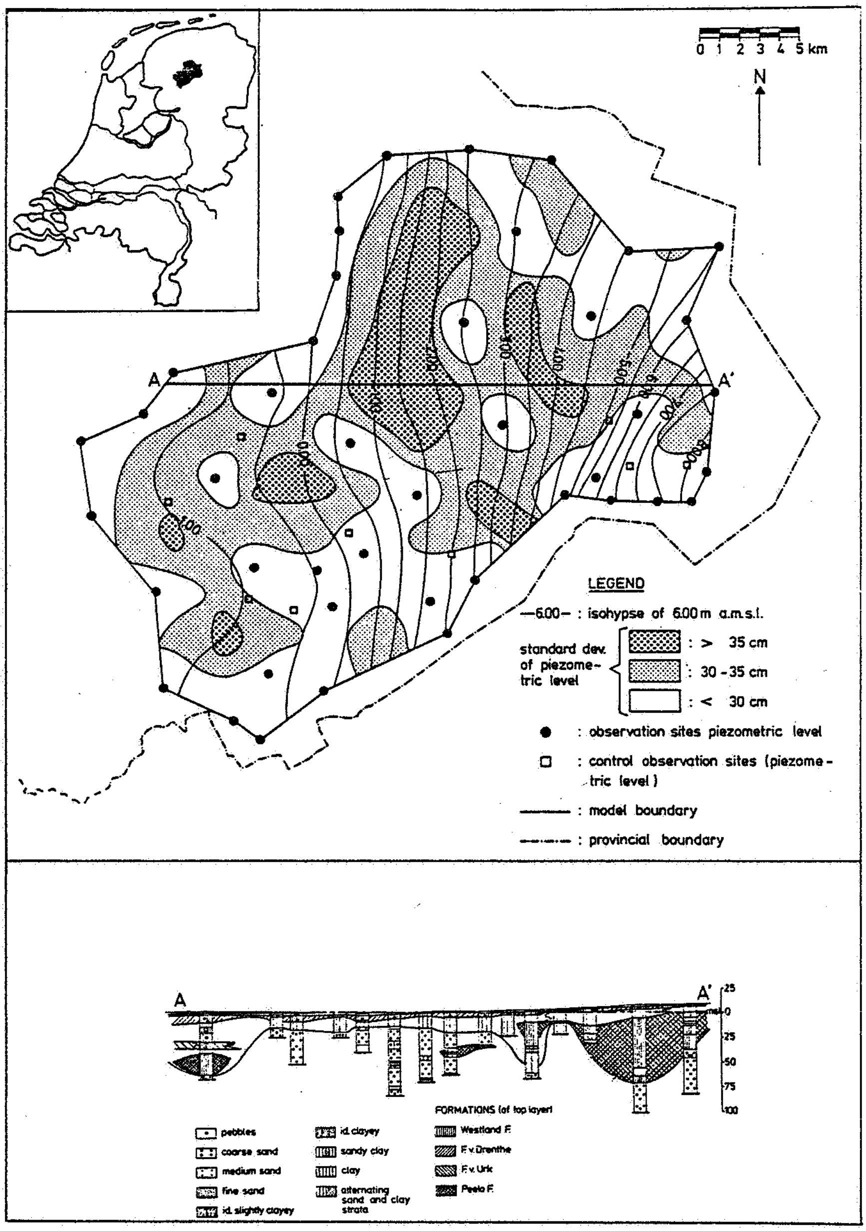

Figure 1 Contoured piezometric level and standard deviation estimated with the kriging interpolation technique in the study area S.E. Friesland. The geohydrological cross section (Jousma,1979) depicts the variability of the toplayer

value of 2.3 cm was found, whereas the root mean squared error (rmse) was equal to 16.5 cm. Actually the absolute difference between observed and calculated value was less than the kriging standard deviation at each of the 9 control locations. This result is not consistent with the simulation results of Hughes and Lettenmaier (1981), who showed for a specific situation that Delfiner's model identification and parameter estimation techniques give rise to an underestimation of the true variance for small sample size ($n \leqslant 30$, say). Our results suggest an overestimation.

4 The deterministic model

In order to compare the results with a model based on geohydrological information not only concerning the piezometric levels a guess field methodology as suggested by Delhomme (1978), has been used.
The well known 2-dimensional staedy state equation for flow through porous media is (Bear, 1979):

$$T\left(\frac{\delta^2 z}{\delta x^2} + \frac{\delta^2 z}{\delta y^2}\right) - W - \frac{1}{c}\,(z - z_c) = 0$$

where

T : transmissivity (m^2/d)

W : flux of infiltration or discharge (m/d)

c : hydraulic resistance of top layer (d)

z : piezometric head in aquifer (m)

z_c: piezometric head in top layer (m)

For known transmissivity, hydraulic resistance, well discharge and boundary conditions this equation can be solved. The mean transmissivity value resulting from several pumping test amounts to 4000 m^2/d. Hydraulic resistance of the toplayer is thought to be very variable. A mean value of 400 days is suggested by pumping-tests. From a map indicating the thickness of the toplayer, we estimated weighting values according to thickness of the space elements defined for the applied finite element solution of the above equation. These weighting values were choosen in such a way that the mean value of c(400 days) over the region remained valid.

For the finite element solution the program Sofia-103 was used (Verruijt, 1980). This program estimates linear basis functions for triangular elements, so linear interpolation was applied for locations within the element bounded by three nodes. A rather difficult decision is how well we should model the phreatic surface. As there are about 250 shallow piezometers in the study area, preparation of input would be enormous. As this is against a search for a practical network reinforcement method, we decided to model the phreatic level at only 75 locations. The deterministic model results in an overestimation of the piezometric level at sixteen observation locations inside the study area. This is merely due to an overestimation of the groundwater level in the phreatic aquifer.

In order to correct for this systematic error and making a transformation to a stochastic framework, we kriged on the differences between the observed and estimated piezometric levels. So kriging was performed with the sixteen inside observation locations and twenty seven boundary observation locations. Of course at this boundary this difference is zero following from the deterministic approach.

For the determination of the trend mean ranks of 1.91, 1.81 and 2.28 were obtained for k=0,1,2 respectively. Note that the differences between these mean ranks are very small. For that reason and also to facilitate a comparison with the first approach, we again kriged with k=0 and 8 neighbouring points. The generalized covariance function $K(|h|)=a_1|h|$ performed well (a_1=-0.0839, jackknife = 0.99) Again $K(|h|)=c\delta(|h|)$ performed better (c=385.8, jackknife = 1.00) but we did not use this function because of the earlier mentioned reason.

The maximum kriging standard deviation of the interpolation error was found to be 18.0 cm. Because the selected order of the drift and also the form of the generalized covariance function are the same as with "direct" kriging, the same spatial distribution of the standard deviation is obtained. In magnitude there is a decrease with a factor $(0.4487/0.0839)^{1/2} = 2.31$. A non-significant systematic error of 2.0 cm at the 9 control locations was found. The rmse of the difference between calculated and observed levels is 14.7 cm.

5 Discussion

The results of this case-study are somewhat curious. Before going into detail we summarize the main results.

1. The accuracy of the prediction of piezometric heads at unknown locations with a linear interpolation model could not substantially be improved by modelling additional information like boundary conditions and geohydrological parameters.

2. The "direct" kriging approach overestimated the interpolation error. The combined deterministic-stochastic approach gives a more realistic view of the uncertainty in the estimated piezometric head. Notice that we have discarded the unknown uncertainty introduced by the estimation of the piezometric surface with the deterministic model.

However one should keep in mind that these conclusions are based on only a few data and will not necessarily hold in general.
The spatial distribution of the interpolation errors which are obtained in the third step of the estimation procedure for both approaches are shown in Figures 2 and 3. Each location is kriged with its neighbouring points. The most striking in these figures are the three large errors in the north east part of the experimental area in Figure 2. If we look at the location of the control points, which are also plotted in these figures, we see that in this part there are no control points. This explains the overestimation of the interpolation error at the control points. Obviously, the combined deterministic-stochastic approach should be preferred for a realistic quantitative mapping of the uncertainty in piezometric head, especially in the north eastern part of the study area. The direct kriging approach is less costly and gives at least a relative measure of the uncertainty in piezometric head. New observation sites should be planned where uncertainty is highest.

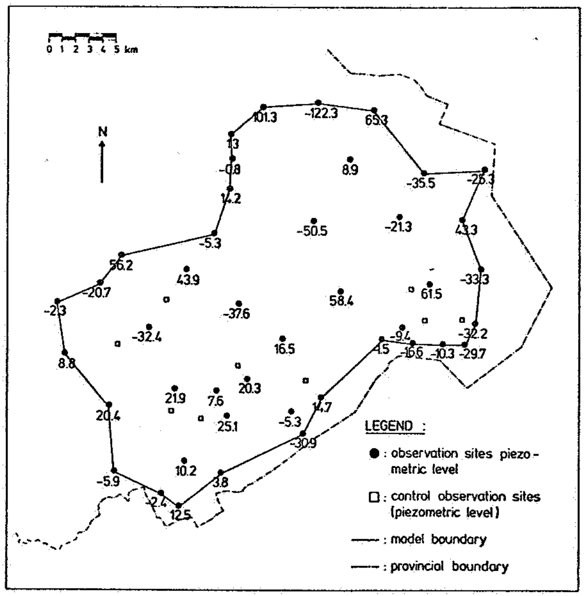

Figure 2 Interpolation errors according to the third step in the direct kriging approach

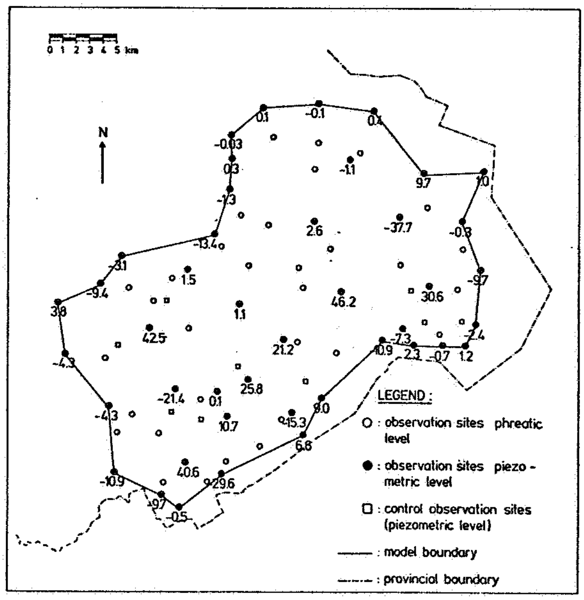

Figure 3 Interpolation errors according to the third step in the combined deterministic-stochastic approach

References

Bear J. (1979). Hydraulics of groundwater. MC Graw-Hill series in Water Resources and Environmental Engineering, New York

Delfiner P. (1976). Linear estimation of non-stationary spatial phenomena, in Advanced Geostatistics in the Mining Industry , edited by M. Guarascio, pp. 49-68, D. Reidel, Dordrecht, Holland

Delhomme J.P. (1978). Kriging in the Hydrosciences. Advances in Water Resourses, Vol. 1 No. 5 pp. 251-261

Gambolati, G. and G. Volpi (1979a). Groundwater contour mapping in Venice by stochastic interpolators. 1. Theory. Water Resources Research Vol. 15, No. 2, pp. 281-290

Gambolati, G. and G. Volpi (1979b). A conceptual deterministic analysis of the kriging technique in hydrology. Water Resource Research, Vol. 15 No. 3, pp. 625-629

Hughes, J.P. and D.P. Lettenmaier (1981). Data requirements for kriging estimation and network design. Water Resources Research Vol. 17, No. 6, pp. 1641-1650

Jousma, G. (1979). Optimalisatie landelijk waarnemingsnet voor grondwatergegevens. Deel 2. Herziening van het net in de provincie Friesland. Dienst Grondwaterverkenning TNO, Delft

Kafritsas, J. and R.L. Bras (1981). The practice of kriging. Report No. 263. Massachusetts Institute of Technology

Matheron, G. (1973). The intrinsic random functions and their applications. Advances in applied probability , Vol. 5. pp. 438-468

Rodriguez-Iturbe, I. (1972). General report. International symposium on uncertainties in hydrologic and water resource systems, Tucson

Verruijt A. (1980). Applications of the finite element method in geomechanics. Delft University of Technology, Delft

WMO (1970). Guide to hydrometeorological practices. WMO-No. 168, TP 82

LE MODELE CONCEPTUEL HYDROGEOLOGIQUE IDENTIFICATION NUMERIQUE DU SYSTEME AQUIFERE.

ACQUISITION ET SYNTHESE DES DONNEES

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Résumé

Le développement du traitement informatique pour l'identification du système aquifère et l'emploi de modèles numériques distribués de simulation, obligent l'hydrogéologue à recueillir des données quantitatives, précises et nombreuses. La synthèse des informations est exprimée par un modèle conceptuel hydrogéologique, base de la modélisation.

Abstract

The development of approaches to the identification of aquifer system based on computing science and the application of distributes numerical simulation models compels the hydrogeologist to collect data which are both accurate and numerous. The synthesis of the information is expressed by a conceptual hydrogeological model which constitutes the basis of the model approach.

An aquifer is identified by four groups of numerical data to be obtained through field investigations, i.e. the hydrological, hydrodynamical, hydrochemical and historical data.

Groundwater reservoirs perform three functions: the storage function, the conveyance function and the exchange function. The hydrodynamical, hydrochemical and hydrobiological processes are related to these functions.

1 Introduction. Le modèle conceptuel hydrogéologique base des modèles numériques

L'informatique, appliquée à l'hydrogéologie au cours de la dernière décennie et son développement futur, imposent des méthodes et des techniques d'acquisition et de traitement de données quantitatives précises et nombreuses. L'emploi, nécessaire, des modèles numériques distribués de simulation permet:

- par calage, en régime permanent, l'identification plus précise du système aquifère et l'évaluation de la ressource en eau souterraine.
- par utilisation, en régime transitoire, la planification de l'exploitation de la ressource en eau souterraine.

L'établissement et la mise en oeuvre de ces modèles reposent sur la présentation, par l'hydrogéologue, d'un modèle conceptual hydrogéologique du système aquifère. Ce modèle est une synthèse, élaborée à partir de données concrètes, recueillies sur le terrain, et de l'expérience acquise. C'est l'expression graphique des données numériques concernant la structure et les fonctions du réservoir ainsi que les comportements du système considéré.

2 Données numériques à recueillir sur le terrain

Un système aquifère, système dynamique, séquence d'espace et de temps fraction du cycle de l'eau, est identifié par quatre ensembles de données numériques, acquises sur le terrain:

- hydrogéologiques: conditions aux limites géologiques et paramètres physique du réservoir;
- hydrodynamiques: conditions aux limites et paramètres hydrodynamiques;
- hydrochimiques: géochimie du réservoir et hydrochimie de l'eau souterraine. Interactions eaux/roches;
- historiques des variations temporelles des paramètres hydrodynamiques et hydrochimiques.

La synthèse de ces données paramètriques définit les fonctions du réservoir et les comportements du système aquifère. Elles doivent être mesurées avec le maximum de précision et au moindre coût (optimisation des mesures).

2.1 Données hydrogéologique du réservoir

Les données hydrogéologiques identifient le réservoir, domaine d'espace fini et continu, déterminé par une formation hydrogéologique ou leur combinaison (système aquifère multicouche). Trois ensembles de données fixes sont à acquérir:

- configuration du réservoir définie par les conditions aux limites géologiques. Ce sont les surfaces limites: toit, substratum et les limites latérales: affleurements, passages latéraux de faciès, failles, etc. Elle porte également sur la puissance et la superficie;
- localisation dans le sous-sol: altitude et profondeur des surfaces limites géologiques;
- structure du réservoir, identifiée par les caractéristiques des matériaux qui le constituent: physiques (pétrologie sémentaire et cristalline, granulomètrie, faciès, etc.) et structurales (tectonique et fissuration). Une importance particulière est portée à la granulométrie des milieux poreux et à la fissuration des milieux fissurés.

2.2 Données hydrodynamiques du système aquifère

Les données hydrodynamiques du système aquifère portent sur deux ensembles:

- conditions aux limites hydrodynamiques: potentiels et flux imposés, surface piëzomètrique;
- paramètres hydrodynamiques: coefficient d'emmagasinement, S; coefficient de perméabilité, K, transmissivité, T et diffusivité, T/S; niveau piëzomètrique, H, charge hydraulique, h et potentiel hydraulique, h_p; débit de la nappe, Q, débit unitaire, q et débit spécifique relatif des ouvrages, q_s; vitesse effective, V_e et vitesse de déplacement, V_d; dispersivité, D.

Les deux paramètres hydrodynamiques principaux sont le coefficient d'emmagasinement et la transmissivité.
Les données sont obtenues par des mesures sur le terrain: dispositifs ponctuels, stations d'essais, périmètres expérimentaux et bassins représentatifs.

2.3 Données hydrochimiques

Elles concernent la géochimie du réservoir (sels solubles et argiles échangeuses d'ions) et l'hydrochimie de l'eau souterraine. Elles caractérisent les échanges eaux/roches. Une importance particulière est donnée à la géochimie des isotopes du milieu.

2.4 Historiques des variations temporelles des paramètres hydrodynamiques et hydrochimiques

Les données recueillies identifient le système aquifère à un instant donné (état initial), exprimé par le modèle conceptuel hydrogéologique. Les variations temporelles des paramètres hydrodynamiques sont exprimées par des historiques. Le plus important porte sur les fluctuations de la surface piézométrique. Une période optimale de dix années hydrologiques est conseillé. Leur recueil est obtenu par des enregistrements continus. Leur connaissance est nécessaire au calage des modèles numériques de simulation en régime transitoire.

2.5 Documents graphiques

La synthèse des données est présentée par des documents graphiques: cartes ou coupes hydrogéologiques et histogrammes. La distribution spatiale des données est exprimée par des cartes, coupes et diagrammes hydrogéologiques. Ces documents graphiques servent au calcul du volume des réservoir, base de l'évaluation de la réserve en eau souterraine et de canevas pour l'interpolation des données numériques ponctuelles sur les paramètres hydrodynamiques, moins nombreux et d'acquisition plus onéreuse. Les historiques se traduisent par des histogrammes.

3 Fonctions du réservoir et comportements du système aquifère

Le système aquifère étant un système dynamique les fonctions du réservoir vis-à-vis de l'eau souterraine et les comportements du système aquifère en réaction avec les impacts de son environnement doivent être identifiés.

3.1 Fonctions du réservoir

Le réservoir remplit trois fonctions, vis-à-vis de l'eau souterraine qui le traverse. Elles sont la conséquence de mécanismes internes, imposés par les paramètres hydrodynamiques et hydrochimiques de sa structure. Ce sont:

- la fonction capacitive: emmagasinement de l'eau souterraine (stockage et libération de l'eau); la différence de la réserve. Cette fonction est associée au concept de réserve;
- la fonction conductrice: transfert de masse et d'énergie, associée au concept d'écoulement de l'eau souterraine;
- la fonction d'échanges ou d'interactions physico-chimiques permanentes entre le réservoir et l'eau souterraine. Dans certaines conditions, zone non saturée en particulier, le réservoir remplit également une fonction hydrobiologique par le pouvoir autoépurateur des sols. Ces fonctions sont associées au concept de qualité de l'eau souterraine.

La mise en oeuvre de ces trois fonctions du réservoir aboutit à une régulation des caractéristiques hydrodynamiques et hydrochimiques des écoulements. Elle s'exprime par des équations de transfert.

3.2 Comportements du système aquifère

Le système aquifère, soumis aux impulsions provoquées par les impacts de son environnement sur les limites hydrodynamiques, émet des réponses modulées par les paramètres de la structure du réservoir. Impulsion, transfert et réponse constituent les comportements du système aquifère. Le système aquifère répond à trois types d'impulsions: hydrodynamiques, hydrochimiques et hydrobiologiques. Celles-ci impliquent trois comportements:

- comportement hydrodynamique assurant la régulation des flux à la sortie. Les facteurs de ce comportement sont: les conditions aux limites hydrodynamiques, les variations de stock d'eau souterraine, le régime de flux (exprimé par les systèmes de flux) et l'état initial et les variations dans le temps des trois facteurs. Il s'exprime par un modèle conceptuel hydrogéologique et par l'expression d'équilibre hydrologique du bilan. Il est régit par les lois de l'hydrodynamique souterraine;

- comportement hydrochimique. L'eau souterraine, au cours de son long séjour et de son écoulement lent, dans le réservoir, subit des échanges géochimiques avec le réservoir. Ces interactions eaux/roches imposent les caractéristiques de la qualité chimique de l'eau souterraine;
- comportement hydrobiologique localisé surtout dans la zone non saturée.

4 Modèle conceptuel hydrogéologique du système aquifère

Le modèle conceptuel hydrogéologique du système aquifère représente, à un instant donné, par des données numériques, interprétées par des coupes

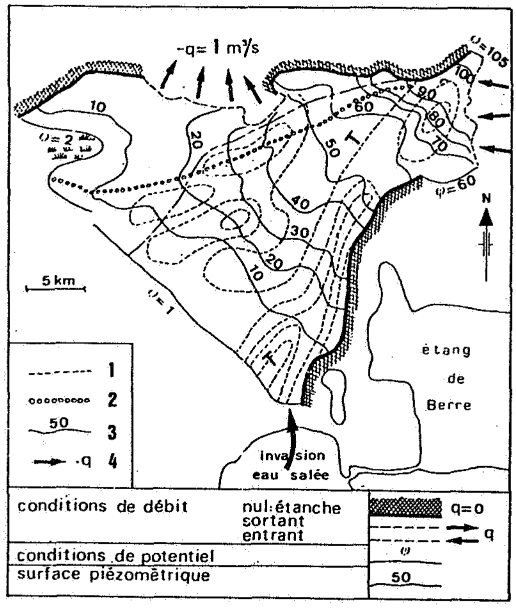

Figure 1
Modèle conceptuel hydrogéologique du système aquifère libre des alluvions de la Crau (Sud de la France), obtenu par synthèse des données et affiné par calage d'un modèle numérique
1, courbe d'isotransmissivité;
2, limite de zone hydrogéologique;
3, courbe hydroisohypse;
4, flux entrant et sortant;
5, conditions aux limites.

ou des cartes, la synthèse de quatre ensembles d'informations:

- caracteristiques de la (ou des) formations hydrogéologiques: surfaces limites, épaisseur, lithologie, granulométrie ou fissuration, etc. ;
- conditions aux limites géologiques et hydrodynamiques: potentiels et flux imposés des apports et des écoulements ;
- distribution spaciale des deux paramètres hydrodynamiques: coefficient d'emmagasinement et transmissivité ;
- distribution spatiale des potentiels et des flux dans le système: surface pièzomètrique et systèmes de flux.

Le modèle conceptuel hydrogéologique est établi, conlointement par le traitement des données acquises sur le terrain et par le calage du modèle numérique distribué, en régime permanent. Deux ces concrets peuvent être cités comme exemples: le système aquifère libre des alluvions de la Crau, dans le Sud de la France (Figure 1) et le système aquifère captif du continental intercalaire du Sahara septentrional (Figure 2). Des documents annexes présentent les historiques des conditions aux limites hydrodynamiques (potentiels et flux) et la synthèse des paramètres hydrochimiques (graphiques, diagrammes et cartes hydrochimiques de l'eau souterraine.

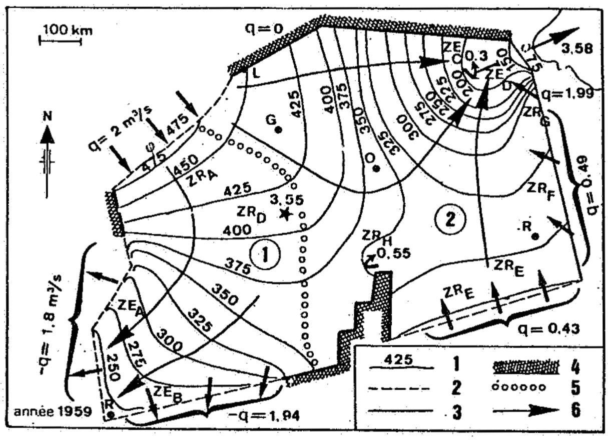

Figure 2 Modèle conceptuel hydrogéologique du système aquifère captif du continental intercalaire du Sahara septentrional. D'après UNESCO (1972). 1, courbe hydroisohypse; 2, limite à potetiel imposé; 3, limite à flux imposé, entrant ou sortant; 4, limite à flux nul; 5. limite de zone hydrogéologique; 6, grands axes de flux.

5 Conclusions

L'identification quantitative du système aquifère, base de l'établissement des modèles numériques distribués de simulation, est exprimée par un modèle conceptuel hydrogéologique. Ce modèle est obtenu par le traitement des données, recueillies sur le terrain. Il est affiné par le calage du modèle numérique en régime permanent. Ainsi une coopération étroite entre l'hydrogéologue et l'hydrodynamicien aboutit à une conception plus réaliste du système aquifère.

Références

Castany, G. 1982. Principes et méthodes de l'hydrogéologie. Dunod, Paris.

Castany, G. 1982. Bases de l'étude scientifique des eaux souterraines. Méthodes et techniques avancées. Colloque international sur le calcul du bilan des eaux souterraines, UNESCO, Varna, 27.9-2.10.1982.

Marsily, G. de. 1982. Hydrogéologie mathématique. Masson, Paris.

Hubbert, K. 1969. The theory of ground water motion and related papers. Hafner Publishing Company, New York and London.

UNESCO. 1972. Etude des ressources en eau du Sahara septentrional. Algérie-Tunisie.

FUNDAMENTAL PRINCIPLES OF ORGANIZING AND CARRYING OUT FIELD HYDROGEOLOGICAL STUDIES IN GROUND-WATER EXPLORATION

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Abstract

The fundamental principles of organizing and carrying out the ground-water exploration and their application under various hydrogeological conditions are discussed in the paper.

In recent years, in many countries of the world there is observed a great increase in the fresh ground-water use mainly for domestic-drinking water supply, and in some regions for land irrigation. Increasing water demand causes a necessity of creating large concentrated water intakes, the productive capacity of which amounts to hundreds and sometimes thousands of litres per second. These circumstances have accounted for large-scale hydrogeological studies on ground-water exploration for estimating the possibilities of the ground-water use. The fresh ground-water exploration is a complex of geological-prospecting works carried out with the aim of determining the ground-water development area [1] and assessing the safe ground-water yield [2]. This complex includes a great number of various studies: geological-hydrogeological or hydrogeological surveys on different scales, ground geophysical studies (electrical prospecting, seismic prospecting, magnetic prospecting, etc.), drilling works, geophysical well studies, pumping-test and test-migration works, stationary observations of the natural and disturbed regime of ground-water; balance-hydrometric works, geobotanic, isotopic, tracer studies; laboratory works for studying physical properties; chemical and bacteriological composition of ground and surface waters; determination of aqueous-physical properties of rocks, topogeodesic works.

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[1]) and [2]): see at the end of the paper

Due to a variety of geological and hydrogeological conditions under which the ground-water exploration and production are carried out, as well as due to a great number of the methods applied for exploration, of great significance are the problems of a rational performance of researches - determination of a necessary type of studies, rational complexing of various work types, establishment of a sequence of their conducting and their volumes etc. In this connection, it is important, above all, to ascertain the factors determining a type and a procedure of the studies. These factors involve[3]:

a) purpose of the problem under solution - water consumption type (municipal water supply, agricultural water supply, industrial water supply, land irrigation) and water demand;

b) complexity of hydrogeological conditions;

c) necessary and actually attainable degree of realibility of the hydrogeological problem under solution;

d) degree of a hydrogeological study of the area under investigation.

Taking into account these factors it is possible to distinguish the following fundamental principles of carrying out hydrogeological studies in the course of the ground-water exploration.

1 Principle of successive approximations

This principle provides for carrying out the exploration by stages (and also in the course of the water-intake operation) with the refinement of the types, volumes and a procedure of works at each of the subsequent stages based on the results of the previous one. In the U.S.S.R. these works are generally conducted by four stages. The first stage is prospecting. It is aimed at the determination of the ground-water development areas, and the promising aquifers within these areas in order to organize further exploration. The second stage is a preliminary exploration of the ground-water development area established, in the course of which the main peculiarities of the geological and hydrogeological conditions of the ground-water development area are studied, a preliminary assessment of the main sources of the formation of the safe ground-water yield is made, and its total value is determined. At the third stage (a stage of detailed exploration) the hydrogeological studies are carried out to

[3]) see at the end of the paper

refine the formation conditions of the ground-water resources and the aquifer parameters to a degree which allows a more rational (in technical and economic respects) scheme of wells to be selected, and the evaluation of the safe ground-water yield to be made in reference to the scheme selected. Finally, the fourth stage involves a productive exploration carrying out in the course of a water-intake operation with the aim of establishing a correspondence of the productive regime to the predicted calculations, reestimating the ground-water resources according to the production data, substantiating a rational productive regime. No doubt, depending upon the above factors, particular stages can be omitted from or combined into a general complex. Thus, the stage of prospecting can be dropped and the stages of a preliminary and detailed exploration can be combined into a single one under the following conditions: small-scale water demand, high degree of the hydrogeological study, simple hydrogeological conditions. Allowance for the principle of successive approximations enables the economic efficiency of field studies to be significantly increased, as based on the results obtained at various stages non-promising can be excluded from the further works, and the types and methods for the studies in promising areas can be refined.

2 Principle of feedback

This principle involves a consideration of the character of a water-intake operation and a degree of the impact of the hydrogeological situation on the selection of a rational scheme and regime of the water-intake operation when substantiating the types and methods of investigations. Allowance for the principle of feedback requires carrying out the works at a stage of a detailed exploration in reference to the type of a water intake and its scheme. Thus, if a future water intake will represent a linear line of wells, the basic volumes of drilling and pumping-test works must be carried out along the line of wells under design; if the ground-water production is planned under the conditions of artificial ground-water recharge, special works must be conducted to determine the number, an operating regime and a location system of infiltrational structures. Another aspect of the principle in view is associated with the determination of a degree of studying various natural factors, depending upon their affect on the formation of the safe ground-water yield and the selection of a more rational scheme of a water intake. Thus, in case of the ground-water exploration of limited structures, where the depletion of natural (storage) ground-water resources plays an important role in the formation of the safe ground-water yield, attention must be given mainly to an assessment of a water yield of water-bearing rocks; in case of the ground-water exploration of multilayered systems a study of the leakage parameters etc. should be taken into consideration. Application of an exploratory (imitation) simulation is more effective for determining the role of different factors in using the principle of feedback.

3 Principle of methodological substantiation

This principle involves the selection and substantiation of the types and methods for the works in the programme of studies depending on the complexity of hydrogeological conditions and the prediction method adopted. The safe ground-water yield is estimated by using various methods, such as: hydrodynamic method incorporating analytical calculations and analog and digital simulation; hydraulic, balance methods and a method of hydrogeological analogy. The procedure for conducting different types of studies should be developed in accordance with the method selected. Thus, under highly complicated hydrogeological conditions, for example, when the ground-water is confined to the zones of tectonic faults, and when it is practically impossible to determine the main sources of the ground-water resources formation using the exploration data, the safe ground-water yield is usually estimated by a hydraulic method, which involves the data processing of long-term test-production pumpings. Under such conditions it is inexpedient to carry out special experimental works in order to study the hydrogeological parameters or to estimate the role of the water-bearing stratum boundaries. When applying the method of hydrogeological analogy for the safe yield estimation, the prospecting works must be organized, so that to obtain the data which substantiate the analogy of the processes of the ground-water resources formation at a standard and assessed sites. If a hydrodynamic method is applied for evaluating the safe ground-water yield, the works at a stage of a detailed exploration must be conducted in accordance with the computational model selected. For example, if separate aquifers are arranged into a single aquifer complex, while assessing the safe yield of multilayer systems, it is necessary to determine the permeability parameters of these aquifers; if each of the aquifers is considered separately in the computational scheme, it should be tested individually.

4 Principle of attaining the required degree of reliability

This principle consists in the determination of the types and procedure of the works, providing the prescribed reliability of the information obtained depending on the complexity of hydrogeological conditions, the character of the problems under solution, requirements to the reliability of a water-intake operation. The necessity of keeping this principle is accounted for the fact that the safe ground-water yield determined on the basis of the exploration data is mainly an estimated value, and the reliability of its assessment is largely dependent upon the quality of information obtained over the period of field investigations. At the same time, it is evident that the information, obtained under various hydrogeological conditions by different methods of investigation, is characterized by an unadequate degree of the reliability. Thus, the values of permeability parameters determined on the data of the group pumping tests are more exactly than those estimated by the results of the single pumpings. Defining the location of any aquifer boundary on the drilling data can be more correct as compared to the one based on the results of the ground geophysical works. On the other hand, the requirements to the reliability of hydrogeological predictions, made on the results of field studies, can be considerably changed depending on the stage of prospecting works as well as on the class, purpose and cost of a water intake. Therefore, the requirements to the reliability of the safe yield assessment under various hydrogeological and water-management conditions must be determined for a rational performance of prospecting works. When developing these requirements, it should be taken into account that the following situations do occur under the complicated hydrogeological conditions: it is impossible to provide a sufficiently high degree of the reliability of hydrogeological predictions on the exploration data or its attainment will require carrying out hydrogeological studies of such a volume that their cost may exceed the cost of a water-intake construction and production.

In the U.S.S.R. the requirements to the degree of exploration of the ground-water development areas, which determine the reliability of assessing their safe yield, are regulated by the "Classification of the safe ground-water yield" representing an official standard. This classification provides for the safe yield division into four categories (A, B, C_1 and C_2) according to the degree of reliability of its estimation, and establishes that the construction of water intakes (with their cost exceeding 0.5 mln rubles) is allocated only with the safe yield being explored to the certain categories. All the ground-water development areas are divided into three groups according to the complexity of hydrogeological conditions, due to the fact that the degree of reliability of the safe yield evaluation is different under various hydrogeological conditions. Various relationships of the safe yield of different categories defining a possibility for allocating the construction of water intakes, is determined for each of the groups.

In this case, these groups with more complicated conditions are allowed to have the safe yield of much lower categories established, since bringing the study of ground-water development areas to much higher categories may prove to be impossible or inexpedient from the economic point of view, as it has been mentioned above. In order to divide the explored resources according to the categories, the qualitative criteria were established on the basis of assessing a degree of substantiation of the initial data used in estimating the resources. These criteria also determine the types of the necessary studies and volumes of various types of works. Undoubtedly, a degree of error of hydrogeological calculations made in assessing the resources could be a more objective criterion of reliability of the estimated ground-water resources. Developing the quantitative criteria of calculating errors, however, is highly complicated due to the impossibility of estimating the errors related to the schematization of natural conditions on the basis of the field data.

5 Principle of purposefulness

This principle is a correspondence of the volumes and types of studies with the character of problem under solution, stage of studies, complexity of hydrogeological conditions and requirements to the degree of reliability of hydrogeological predictions. As mentioned above, different problems are solved and also various requirements to the substantiation of initial information are formulated at various stages of hydrogeological studies. Thus, pumping-test works should be carried out by single pumpage at the stage of prospecting, where the major problem is to ascertain promising areas and aquifers, which must be comparatively estimated. At the same time, at the stage of a preliminary exploration a definite number of group pumpings must be carried out, the group pumpings being the basic type of pumping-test works at the stage of a detailed exploration. The principle of purposefulness also involves allowance for a resolving power of different methods when selecting the types of investigations. Thus, it is inexpedient to study the leakage processes in poorly permeable deposits by intensive pumpings, if under the given conditions this effect can be seen in a very long period of time, considerably exceeding a reasonable duration of pumping-test works.

6 Principle of optimal character (economic efficiency)

This principle requires performance of the studies, which provide for obtaining such a complex of information that determines an effective solution of the final engineering problem. Following this principle, it is necessary to coordinate the volumes and the location of prospecting works in the study area with the prescribed reliability of solving the task set up, and also with a risk of economic losses in the course of the water-intake operation. In this connection, the requirement to obtaining a maximum information with minimum expenses is incorrect. The information should be obtained that can be used for assessing water resources and calculating a water intake; an efficiency of expenses for studies should be determined by the possibility of obtaining the most effective engineering solutions on the data of prospecting works. For example, the cost of group pumping tests considerably exceeds the cost of the single ones. However, the parameters determined by group pumpings enable the more reliable selection of the most optimal (in technical and economic respects) scheme of a water intake; it results in an economic efficiency when constructing and operating the water intake.

The above principles of organization and performance of hydrogeological studies in the ground-water exploration are being widely used in the Soviet Union. It provides for an increase in a rational performance of the works as well as in the reliability of hydrogeological predictions made on the data obtained from the studies.

Notes

1 The ground-water development area in the U.S.S.R. is understood to be a part of the aquifer or aquifer complex occurrence, within which natural or artificial factors favour the ground-water withdrawal sufficient for its reasonable use.

2 Yazvin, L.S. and B. Paczyhski. Methods for groundwater prospection. In Knige "Metody otsenki resursov podzemhykh vod." Trudy Mezhdunarodnoi assotsiatsii gidrogeologov. Tom XV, chastj 2, Moscow: Nauka, 1980.

3 Gavich, I.K., V.A. Mironenko and L.S. Yazvin. Optimizatsia gidrogeologicheskikh issledovanii v svyazi s zadachami prognozov. V knige "Formirovanie podzemnykh vod kak osnova gidrogeologicheskikh prognozov", Materialy I-oi Vsesoyuznoi gidrogeologicheskoi konferentsii. Tom I, Moscow: Nauka, 1982. str. 76-88.

References

Gavich, I.K., V.A. Mironenko and L.S. Yazvin. 1982. Optimizatsia gidrogeologicheskikh issledovanii v svyazi s zadachami prognozov. V knige "Formirovanie podzemnykh vod kak osnova gidrogeologicheskikh prognozov", Materialy I-oi Vsesoyuznoi gidrogeologicheskoi konferentsii. Tom I, Moscow: Nauka, str. 76-88.

Yasvin, L.S. and B. Paczyhski. 1980. Methods for groundwater prospecting. In knige "Metody otsenki resursov podzemnykh vod". Trudy Mezhdunarodnoi assotsiatsii gidrogeologov. Tom XV, chatj 2, Moscow: Nauka.

A FIELD MONITORING PROGRAMME FOR THE ASSESSMENT OF THE QUANTITY AND QUALITY OF THE GROUNDWATER RESOURCES IN THE BOHEMIAN CRETACEOUS BASIN

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Abstract

The Bohemian Cretaceous Basin is nearly 16 000 km^2 in extent and at present more than 6 m^3s^{-1} of groundwater is exploited there. A further increase in exploitation by more than 100 % is expected in the next decade. Such an increase, however, requires thorough investigation.

The geological structure of the basin is very complicated. Tectonics and combination of hydrological/hydrogeological boundaries divide the basin into nine subbasins. Two main aquifers are recognized: unconfined in the Middle Turon and confined in the Cenoman. The considerable variations of climatic, hydrological, tectonical, geological and soil conditions and manifold vegetational cover indicate that the ground-water resources are not uniformly distributed in the basin and owing to the fluctuation of the rainfall and runoff, both also highly variable year by year.

With regard to the extent of the investigated area and the large quantity of monitored data (1000 observation points), a conceptual model approach has been found to be most suitable for the assessment of groundwater resources and of their quality under existing conditions. As a model input, a databank from the daily and weekly records of standard climatological, hydrological, hydrochemical and groundwater networks was set up. However, the model parameters characterizing the

structure and formulas describing various components of the hydrological cycle in the basin,need to be verified by a more frequent and complex monitoring system in a small representative area selected in the basin.Such an approach has resulted in the establishment of the monitoring programme in the model watershed at Nedamov.

1 Introduction

Headwaters of the river Liběchovka, a tributary of the Labe (Elbe) river below the city of Mělník have been selected as a model watershed given the name of Nedamov.The upper reaches of the river originate from numerous springs and the baseflow originating in the Middle Turonian aquifer.A substantial part of the area is formed by sandstone sedimentary formations,typical for the basin.Pinus Silvestrus L. and several types of deciduous species form the afforested part of the watershed while the rest is covered by fields and meadows.The urbanisation rate is low and thus the local impact of man on the hydrological cycle is rather insignificant.Actually, a dominant part of the phenomena affecting the water quality is of exogeneous origin, which appears to be typical for the whole basin.The groundwater resources in the model area are used only for domestic purposes.

2 Field monitoring programme

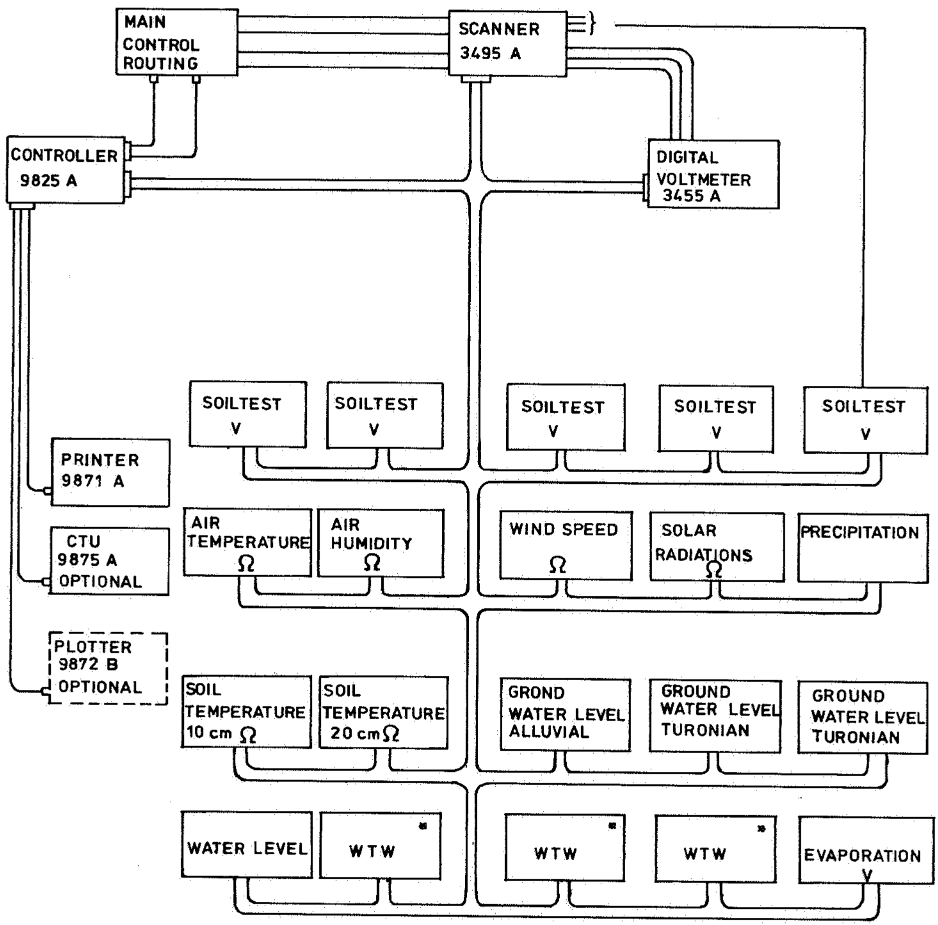
Because the principal conceptual model SG provides the groundwater assessment as a result of complete and continuous water balance calculation,it has been found essential to observe in the model watershed all phases of the hydrological cycle.For the development and verification of the formulas it is inherent to collect data at short time intervals.This leads to a great number of collected data,so an advanced system facilitating rapid and direct access to the collected data and their further processing is needed for this purpose.

The Hewlett-Packard 3052 A Data Acquisition System,consisting of a scanner 3495 S, desk top calculator 9825 A, digital voltmeter 3455 A, printer 9871 A, casette unit 9875 A and real time watch 98035 A with interface HPIB 98034 A, were selected as the most convenient configuration.A newly added plotter / digitalizer 9872 A facilitates the visualisation of the monitored records as well as the corrected records after the gaps resulting from the missing records have been filled (Balek,Kalábová 1982).

The system provides the scanning,digitalising and monitoring of data at pre-selected time intervals in real time. The data either in Volts or Ohms are stored on magnetic tape which is considered to be a basic record.Further the data are evaluated by using calibration curves.The records are evaluated by using calibration curves.The records of both types are graphically and numerically processed in an attempt to check up on unreliable and/or missing records. Providing the calibration curves have been revalidated the basic tape can be used for re-evaluation.The results as obtained from standard observation in graphical and numerical form can be corrected,digitalised and stored by a reverse process on the plotter/digitaliser. Missing records can be filled on and stored in a similar way. An hourly interval has been preselected for the standard monitoring programme,however,any other interval can be used,particularly for special measurements.For instance,water quality monitoring intervals for chemical or microbilogical analyses were set up only exceptionally at hourly intervals,usually daily or weekly monitoring intervals are more frequent.

3 Quantitative measurements

At present the system is capable of monitoring 40 measuring stands (Figure 1).From the great variety of the sensors available on the market such types have to be selected which first, are able to measure with higher accuracy than standard instruments,second, which are compatible with the data acquisition system and third,which can be be operated for a prolonged period under strenuous field conditions.



* CHANNEL RESERVED FOR CHEMICAL SENSORS

Figure 1 Symbolic flowchart of wiring sensors

The meteorological phenomena affecting evaporation and evapotranspiration are measured by Schenk's sensors. The evaporimeter FO 103 with automatic needle adjustment is also of Schenk type. As has been ascertained during measurements, this instrument can also monitor the impact of horizontal precipitation and dew. Rainfall is monitored by an instrument, added to a standard recorder of the Lamrecht type. This also has been fitted with a heating system so that it can measure solid precipitation.

For soil moisture measurement Soiltest cells have been utilized. Some adaptation was necessary to replace direct current measurement by alternating current, otherwise the cells could be damaged. At present new dielectric cells, developed at Prague Technical University are being tested.

Bell and Howell transducers have been used for groundwater level and discharge measurement. At present an alternative system developed at SG and based on the photo-electric cell is being tested.

Direct volumetric monitoring of the transpirational loss is provided by an instrument developed at the Institute of Forest Ecology, Brno. This consists of a power input generator, a series of thermocouples, electrodes and recording system (Balek et al., 1983). The system operates as a separate unit, an attachment to the monitoring system is in preparation.

Last but not least the operation of an independent standard network has been found to be essential, having regard to blackouts during and after storms and the time required for the maintenance of minor repairs to the data acquisition system.

4 Qualitative measurements

Qualitative changes of the hydrological system in time and space are under study. Therefore the qualitative properties of all the components of the hydrological cycle is the object of investigation.Due to the cost of the laboratory analytical work the optimal frequency of water quality sampling is also studied.

Water quality monitoring of the main chemical macrocomponents is provided by the WTW sensors directly in field.Results are stored in the said data acquisition system.Accuracy requirements,however,call for the regular transportation of samples to the central laboratory in Prague.

With regard to the energetic situation of the Czech Socialist Republic with thermal power plants using brown coal prevalently,additionally to the current macrocomponents all microcomponents of heavy metals - Zn, Pb, Cu, Cr, Cd, Be, V, Ni, As, Se, Sc are analysed and the contents of hydrocarbons,detergents,phenols and of microbial components are also monitored.It has been concluded in the model watershed that the direct activity of man is without significance there and thus only the impact of large scale industrial exhalations with the subsequent acidification of the rainfall is under study.This type of pollution originates far from the model area,but significantly affects the ecosystem and hydrological regime of the model area.

4.1 Precipitation

A special device for precipitation sampling,developed in the Central Institute of Geology in Prague,was installed in the model watershed. This device functions automatically immediately rain begins and is closed the moment it stops.From each precipitation a separate sample is available. Precipitation analyses are realised exclusively in laboratories by nuclear absorption spectrometry methods and in a complex of equipment which consists of a gas chromatograph and a mass spectrometre Hewlett-Packard 5985 A. The physical properties of aerosols are studied by elctronic microscopy method.

4.2 Surface water

Special attention is paid to qualitative changes in water in the course of a day,in dependence on biological activity.Sampling at hourly intervals occurs only in the case of the observation of conductivity. A small Liběchovka tributary drains out of zones of agricultural and urban pollution.Another one drains out of zones covered by forest and meadows not affected by human activity.Both torrents are monitored and the results compared. Analyses of microcomponents of organic and microbiological matters are carried out in laboratories. The chemical composition of the fluvial deposits is also under study.

4.3 Water quality changes in the unsaturated zone

The quality of water which flows from the lysimetre surface and from vertical collectors placed at depths of 80, 120, 160, 200 and 240 cm is also monitored.Sampling is provided when precipitation exceeds 1 mm.Monitoring is concerned with the washout of trace elements and of nitrates from the soil layer into groundwater.The most significant quality changes have been found between the surface and a depth of 1 metre.

4.4 Groundwater quality

Hitherto groundwater quality under natural conditions has been under study. In principle,from the boreholes a double quantity of accumulated groundwater is discharged before sampling.Quality changes due to long lasting permanent pumping from the boreholes will be studied from 1983 - 1986.The results of all analysis are deposited in the main data bank system which also contains statistical,graphical and cartographic collections.

5 Supplementary field work programme

In addition to the monitoring programme special field measurements have been provided,which can be utilized for the further generalization of the results achieved in the model watershed.Besides expeditional discharge measurements identifying the baseflow development in each subregion,special attention is paid to the determination of the groundwater age.

By using ^{14}C, ^{13}C, ^{3}H, ^{226}Ra, U_{nat} and ^{222}Rn the origin of the baseflow and the composition of it from the groundwater resources of a different age is assessed in the boreholes where no ^{3}H has been traced and then the result can be used for the determination of the ratio of old and young water in the samples which also contain ^{3}H and ^{14}C (Anonym. 1982).

Such a type of field work is aimed at determining the interactions between the hydrogeological subbasins.An analysis of the spatial and time distribution of the other isotopes as given above serves for the identification of vertical interaction between the aquifers.

One of the typical outputs of such a combination of various programmes is an extended water balance equation.For example in the year 1981 was found (in mm) :

$$743.9\ P + 61.6\ G_i = 5.3\ R + 32.4\ G_o^1 + 8.7\ G_o^2 + 48.4\ O_o^1 + 13.2\ O_o^2 + 64.0\ V + 6.2\ M + 45.9\ E_T + 581.4\ E_i$$

where

P = the annual precipitation

G_i= the groundwater inflow from another subbasins

R = surface runoff

G_o^2= the recent groundwater outflow below the observation profile

G_o^1= the recent groundwater outflow through the profile

O_o^1= the old groundwater outflow through the observation profile

O_o^2= the old groundwater outflow below the profile

V = the volume of pumped water

M = the soil moisture increase

E_T= the forest/meadow transpiration

E_i= transpiration loss from the intercepted water

In an attempt to extend the validity of the field programme in the model watershed,a method based on the spectral analysis of infrared photography has been introduced.A camera installed in a small model aeroplane operated from the ground by a radio transmitter takes infrared pictures of the measured species and of the surrounding area.A comparison of the ratio of blue,red and green for measured points and for the region provides information on the behaviour of various types of vegetational cover,bare soil etc.The evaluation of the infrared pictures is fully computerized and thus objective.

6 Databanks

Basically there are two types of databanks set up for groundwater resources assessment.The databank as obtained from the data acquisition system serves for the solution of the general conception of the main model and for the verification of the input parameters.As preliminarily indicated,other encouraging results are being utilised by the University for educational and research projects.Actually,such a type of work is encouraged by Stavební Geologie.

For the main model a databank based on standard records of the climatic,hydrological,hydrogeological and hydrochemical observation networks has been set up.A period of hydrological years 1974-1976,which appears to be representative when compared with long-term means,has been selected for this purpose.The records are stored at daily intervals.In the case of chemical and microbiological analyses,weekly and monthly intervals are also used.Thus for various types of strategies and conceptual approaches solved by the main model,arbitrarily chosen combinations of the observation points can be selected as an input. At present records from some 1000 observation points are available for this purpose.

7 Manpower

The field monitoring programme as described is the product of continuous team work.The basic team consists of a hydrologist,hydrogeologist,system analyst,an electronic engineer and hydrochemist.Supplementary field work is provided in cooperation with various specialists working on part time and/or contract terms.An ecologist,soil physicist nuclear physicist,photographer and one or two programmers are cooperating on the project.

References

Anonymous 1982.Survey of the exploitability and protection of groundwater in extensive sedimentary structures.Interim Report SG 210 600 (In Czech), 420 p., Prague.

Balek,J. and Kalábová L. 1982.Automatic data acquisition system for groundwater resources studies.Int.Symp.on Hyd.Res. Basins Bern,12 p.

Balek,J., J.Čermák,J.Kučera and K.Prax 1983.A direct method for forest transpiration measurement.Journal of Hydrology,in print.

COST-EFFECTIVE GROUNDWATER
QUALITY SURVEILLANCE

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Abstract

Given that the objectives of monitoring are clearly understood by all parties and that the hydrogeology is adequately understood, the cost-effectiveness of any system must be seen in the perspective of the economic value of the groundwater resource. Drilling is costly and a first objective must be to keep the number of holes to the minimum compatible with adequate coverage in terms of area <u>and</u> depth; multiple depth-interval monitoring within individual boreholes can offer an important saving, but requires a greater initial emphasis on an understanding of the basic hydrogeology. Even where the sample interval/ location is properly defined, errors in sampling are unavoidable though they can be assessed. Indeed, an understanding of these is critical if the unnecessary cost of specifying spurious analytical accuracy is to be avoided.

1 Introduction

Groundwater is still regarded as a prime source of drinking water, protected as it is from the direct effects of pollution at the surface. However groundwater,unlike surface water, can suffer very long-term damage from the effects of pollution. In recent years there has been an increased awareness of this especially in countries where groundwater is intensively used. In some European countries in particular, where population densities are high, the cumulative effects of a range of apparently acceptable activities is now measurable. Even if some of the causes could be removed it is probable that the damage has been done, so that all we can do is observe the development of contamination and attempt to reduce its effects as far as possible. In developing countries there are also cases of serious groundwater pollution, but in general terms there is still the opportunity to establish a suitable monitoring system which will allow them to avoid the worst problems of the Western world. Whenever groundwater forms an important element of public supply, irrigation or industrial supply, it is increasingly understood that deliberate and carefully considered management of quality as well as of quantity must be included in the planning.

2 Defining objectives

One of the most important objectives in setting up a monitoring system must be to protect public health and ensure that statutory requirements are being met. It is also important to establish ambient groundwater quality on an aquifer-wide scale so that contamination can be readily recognised. Where particular pollution hazards are known to exist, a specially high density of monitoring boreholes may be used for "offensive"-type monitoring around the potential hazard. For specially susceptible groundwater supplies, a similar "defensive" system may be appropriate. In all cases it is imperative that the basic hydrogeological situation is fully understood; without it, the significance of any results may be unnecessarily difficult to assess. This is only one of the many unacknowledged deficiencies and problems in the acquisition and use of groundwater quality; some others are listed below.

- Incorrect design of observation boreholes, inadequate knowledge of local geology, and failure to understand the chemical changes that inevitably occur between borehole and laboratory bench.
- Failure to define objectives, in discussion with the engineers, chemists, biologists, administrators, mathematical modellers etc., who must use the results.
- Data are not regularly reviewed with the aim of either simplifying the monitoring system, or improving it. The collection, handling

and presentation of data is often not well thought out.

Even a properly and economically designed monitoring scheme may be expensive, compared with a scheme for surface water for example. This is inevitable because drilling is usually necessary, but these costs should be viewed in a wider and fairer context (Wilkinson & Edworthy, 1981).
There are many new techniques for sampling groundwater more economically, and for linking sampling and measurement systems (Robin et al, 1982). In this paper some applications of two of these systems are described, and the way in which their use helps to deal with some of the problems listed above, in an economical and effective way. Other methods of improving effectiveness while reducing cost are also discussed.

3 In-situ sampling

An 'in-situ' sampling device, based partly upon the "pressure-suction" lysimeter of Parizek & Lane (1970), was developed during research at the U.K. Water Resources Board (Satchell & Edworthy, 1972) and the Water Research Centre (Edworthy and Brown, 1976 HMSO, 1978). This has since been modified for different applications including 'open-hole' depth-sampling and low-rate pumping. It comprises essentially two chambers, connected by a non-return valve. One chamber allows groundwater to flow in through fine mesh-covered ports, and to be collected in the second chamber. A pressure-line and outlet tube are connected to the second chamber, and it is possible to eject a sample for collection at the surface or for direct on-site measurement (Edworthy & Baxter 1980). For "in-situ" use the sampler is placed in the middle of a 2 m silica-sand pack against the aquifer interval of interest,(or maybe several in one hole),and sealed in using clay; bentonite may not always be suitable however because of its chemical reactivity. To illustrate the ways in which the "in-situ" sampler has been used to improve data collection at minimum cost, some examples are described below.

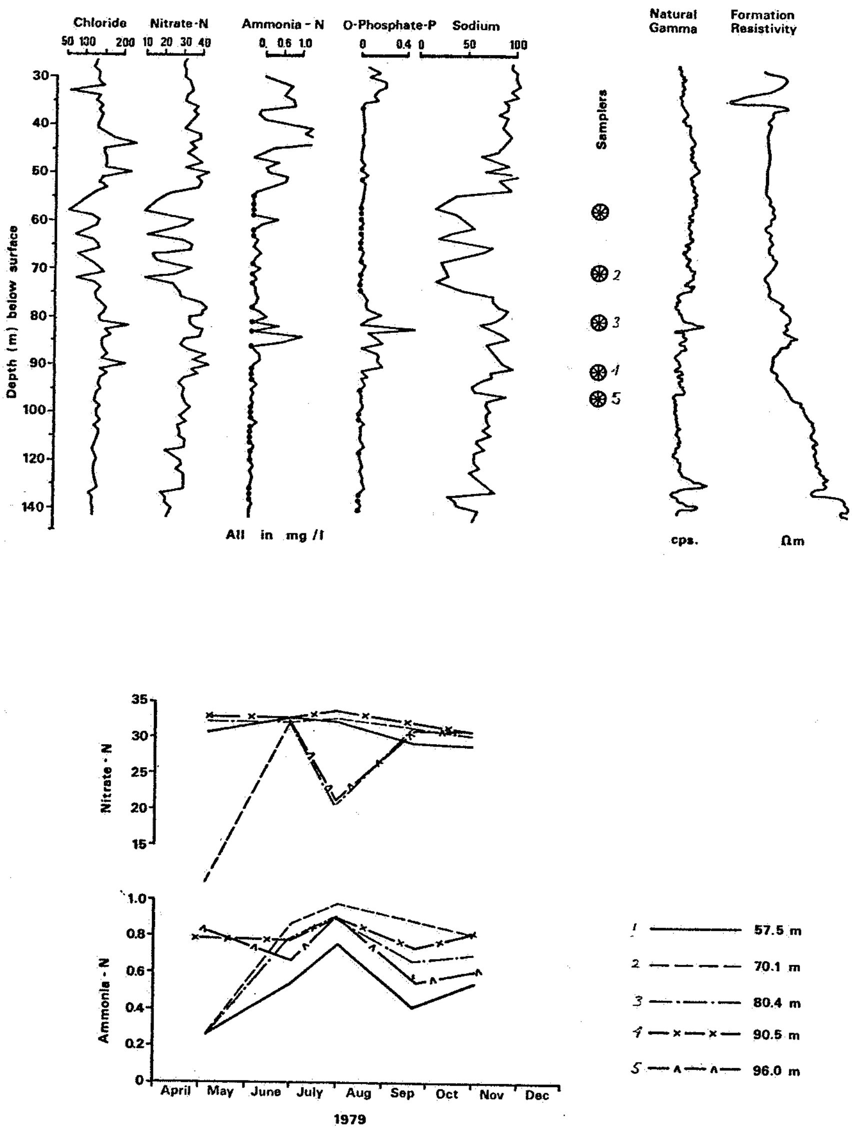

Figure 1 Profiles from a borehole in the Bunter Sandstone illustrating selection of sampler intervals and the results of long-term sampling at these depths; West Midlands, U.K.

3.1 Examples of Practical Applications

3.1.1 Whittington, West Midlands; Bunter Sandstone

Detailed studies at a sewage effluent recharge site have included aquifer core sampling for pore-water analysis down to 122 m depth. Samplers were installed on the basis of detailed stratigraphic and groundwater quality information, to allow long-term monitoring within five narrow intervals within the contaminated zone. A summary of the data used in this example, the location of the samplers and the results of a sequence of analyses, are shown in Fig. 1. Groundwater is shown to be contaminated to the total depth drilled. It is also possible to identify several chemical zones in the pore-water profile which can be directly and satisfactorily correlated with lithological zones identifiable from cuttings and geophysical logs. In this case particularly good hydrogelogical data have been available to guide the planning of longer-term specific-depth sampling. The initial depth samples show some odd variation as drilling water within and in the vicinity of the borehole is replaced by natural groundwater. After three months, stability and a consistent differential appears to have become established.

3.1.2 Stourbridge, West Midlands; Bunter Sandstone

Exploratory drilling was carried out in this area to define the extent of groundwater contamination resulting from agricultural activities and sewage sludge spreading. The profile of concentration of nitrate in pore-water showed clearly that a well-defined plume of contamination occurred between 35 and 95 m depth with uncontaminated water beneath (Fig 2). The concentration of nitrate in the borehole indicated relatively minor contamination. In this type of situation where large resources of fresh groundwater are available beneath the contamination, scavenging of the nitrate-rich water could allow continued use of the source, which may otherwise have to be closed down. Knowledge of the details of the distribution of contamination obviously allow this in the first instance. Longer term monitoring at this site was possible using "in-situ" samplers and the remarkable consistency of groundwater quality is also well shown in Figure 2.

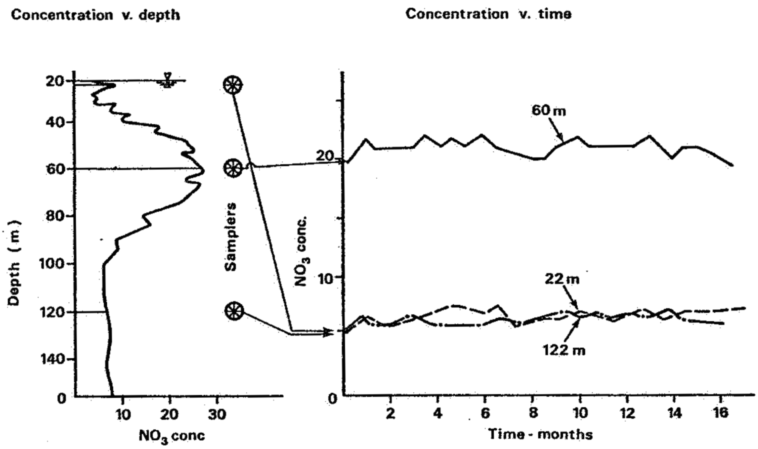

Figure 2 Nitrate concentration in Bunter Sandstone porewater selected sampler depths and monitoring results, West Midlands, U.K.

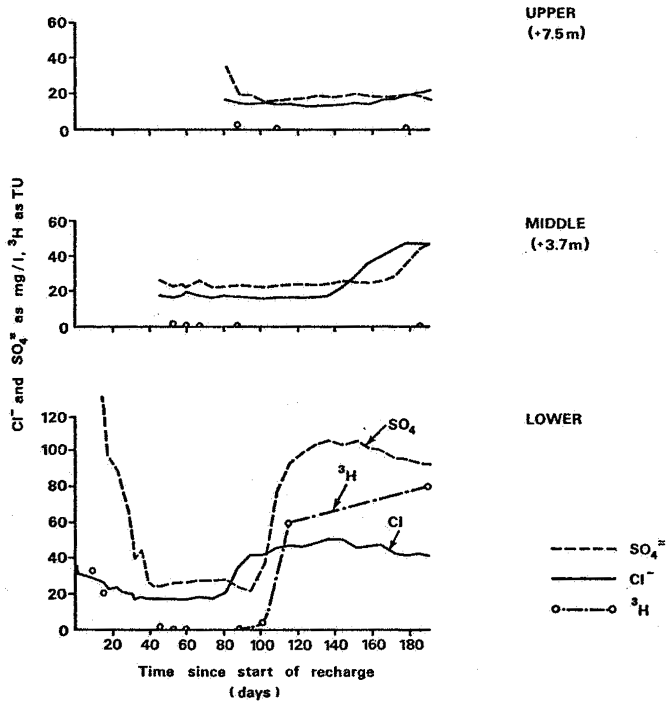

Figure 3 Detailed water quality changes at three levels in the Tertiary Sands following the rise in groundwater level caused by artificial recharge - Lee Valley, London.

3.1.3 *Lee Valley, London; unconsolidated Tertiary sands*

The London Clay upon which the city of London stands, also forms a good protective cover to the Tertiary sand and underlying Chalk aquifers. In the last 150 years these aquifers have been heavily developed and partly dewatered. Artificial recharge of the storage space created by dewatering was seen as a valuable opportunity to increase the overall yield of the local surface water sources (Water Resources Board, 1972), and experiments culminated in a prototype recharge scheme in 1977. Artificial recharge had been found to be a cause of some groundwater quality changes and special monitoring was needed. On the basis of experience obtained in earlier work, "in-situ" sampling was found to be desirable and three narrow depth intervals within the Tertiary Sands were monitored near a recharge borehole (Figure 3). Measurements taken during operational use, both recharge and pumping, enabled the movement of groundwater to be identified and related to piezometric changes. It also allowed an estimation of the uptake of various dissolved substances from the sand aquifer, and proved that there was hydraulic continuity through that part of the aquifer system. The use of "in-situ" samplers for longer-term monitoring of the effects of recharge is particularly important in this setting where future operational use for public supply is envisaged.

3.1.4 *Winchester, Hants; Chalk*

One other important example of the use of "in-situ" methods has been at Winchester, where the effects of sewage effluent infiltration have been studied (Water Research Centre, 1980). Four samplers were installed at different levels in boreholes in the Chalk aquifer. The depth-intervals were selected, as in the Whittington example described above, from downhole geophysical and pore-water quality information. Results so far appear to indicate that pore-water may differ substantially from the 'mobile' groundwater in a fractured aquifer like the Chalk, which is pumped by in-situ samplers. This fact in itself may be of some importance when longer-term records of water quality are available.

3.2 Estimated Cost-Benefit

In each of the cases discussed, essential data has been obtained without additional costly drilling, and considerable savings achieved. Of greater importance, the samples obtained have been better samples, certainly better than could have been obtained from an open borehole. The estimated cost savings for the above examples can be estimated by comparing the cost of drilling a series of holes to specific depths, with the chosen option.

Table 1 Comparison of the costs of in-situ and conventional borehole monitoring installations

Example	A In-situ sampling £000's	B Drilling £000's	A/B
Whittington	1.6	4.1	0.39
Stourbridge	2.6	11.8	0.22
Lee Valley	3.6	10.6	0.34
Winchester	4.4	18.0	0.24

(The cost of in-situ samplers has been assumed to be £150 each and the gas supply and instrumentation set-up about £750. Drilling has been costed at £40/metre)

It must be accepted, however, that the most effective use of specific depth-samplers usually requires good, relatively detailed information as discussed above. Acquisition of these data entails significant cost and the comparison in Table 1 is not therefore entirely accurate in this respect. Even allowing for this however, it is still believed that much better information can be obtained by the "in-situ" type of sampling for considerably less than one-half of the cost of conventional methods.

4 Unsaturated Zone Sampling

The unsaturated zone is the site of many important chemical reactions which affect the composition of groundwater. Contaminants may be substantially removed during infiltration as a result of such reactions. There are many direct ways of sampling for groundwater from the unsaturated zone; a good summary of these has been given by Wilson (1980). Sufficient water for detailed analysis however is often difficult to obtain without great expenditure of time or effort. The process of sampling generally involves the application of suction or pressure, which can cause substantial changes in the concentration of some contaminants.

An alternative indirect method of monitoring conditions in the unsaturated zone is to sample the pore-gas which is at equilibrium with the water-phase. This method has been used qualitatively by McMichael and McKee (1966), Satchell and Edworthy (1972) and Rightmire (1978). In all three cases the application was to wastewater recharge monitoring and results showed that the gas composition, particularly the oxygen content, could be used as measure of the "treatment" capacity of the unsaturated zone. An example of the compositional changes encountered during and between recharge periods is given in Fig.4, which is taken from sampling points beneath a wastewater infiltration lagoon, on the Bunter Sandstone, near Mansfield.

This method could potentially be used to monitor the unsaturated zone wherever phreatic groundwater was susceptible to a localised source of pollution from the surface. This could be sewage disposal areas, sewerage lines, beneath landfills or where slightly contaminated industrial effluent was being discharged to groundwater. Simple ceramic tensiometer cups have been used to obtain the gas samples by suction, although some care is needed to avoid disturbing the gas-liquid equilibrium (Rightmire, 1978). Accepting this small constraint, sampling and analysis is simple, rapid and inexpensive.

5 Appreciation of Error & Variability

Aquifer systems are nearly always more complex than our best exploratory work and monitoring systems would lead us to believe. An understanding of this actual variability (which is usually not achieved because of the way we look at groundwater) is essential for proper interpretation of any set of data. The basic problem in sampling - how representative is the sample of the whole? - is a considerable

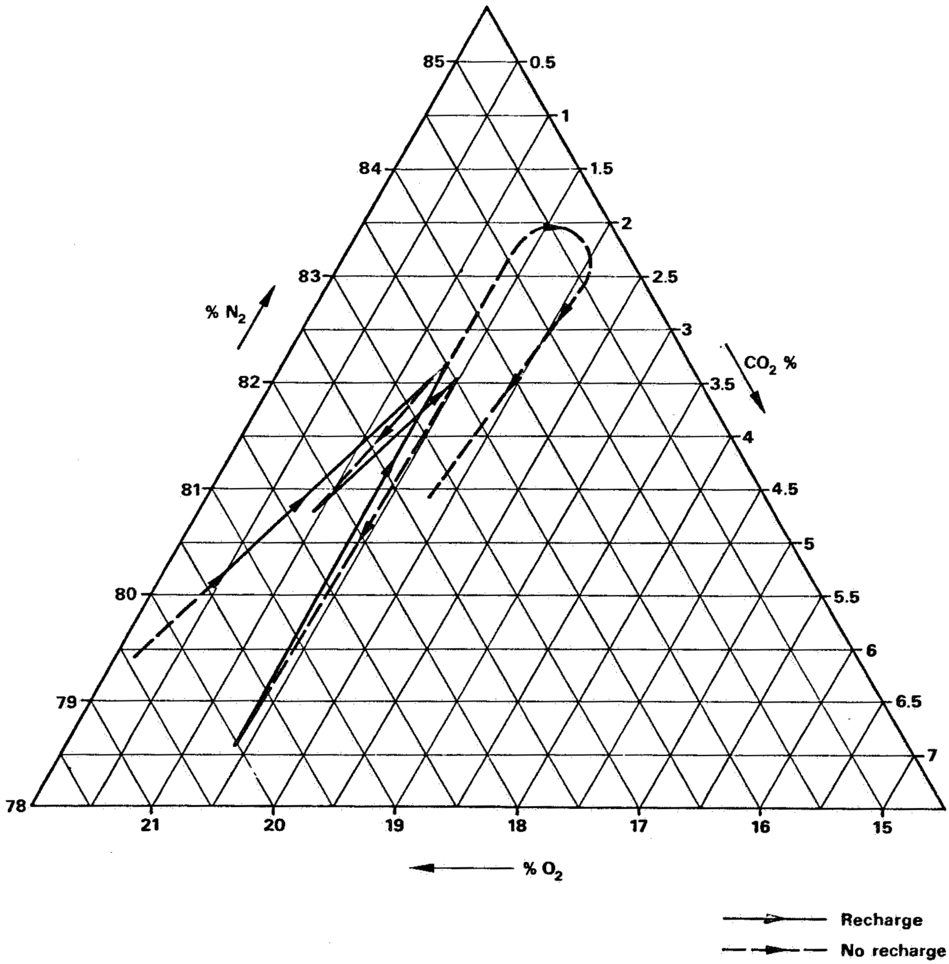

Figure 4 Variation in unsaturated-zone pore gas composition due to wastewater infiltration - Bunter Sandstone, Mansfield, U.K.

stumbling block in hydrogeological work, hence the stress earlier in this paper, of obtaining the most comprehensive information possible in the exploratory stage.

5.1 Sampling

At the time of sampling groundwater, whatever method is chosen, there are potential errors due to seasonal/recharge effects (Sarma et al 1979), to the period between sampling and the amount of pre-sample pumping (Colchin et al 1978, Slawson et al 1982). The effects of high nutrient concentration (N, P, dissolved organics) may cause biological changes which can affect chemical composition, and groundwater may react with screen or casing (Marsh & Lloyd, 1980). Many of these effects are now well understood, and pre-sample pumping of boreholes is now routine for most competent monitoring agencies. Pumping of 'in-situ' samplers is relatively simple and inexpensive, in that only a small volume of groundwater needs to be circulated, and the time taken is correspondingly shorter.

5.2 Sample storage

Storage and preservation of water samples before analysis is a further source of "error" and there is valuable advice in some recent publications on this subject (Wood 1976). It is almost inevitable that there will be some "error" incurred at this stage but it should, of course, be minimised. It should also be identified and assessed as either 'random' or 'systematic', in the same way as analytical data are tested (Wilson, 1982), where necessary. At this stage in the sampling/ analysis scheme there is already, potentially, considerable cumulative error and little justification for high accuracy in the analysis of the sample.

5.3 Analysis

Apart from interpretation, analysis is the final phase of the groundwater monitoring chain. There are three major areas, in which the excellent and very sensitive methods available to us today, are misused. Firstly, we fail to understand the concept of analytical accuracy, a subject well dealt with by Summers (1972). He illustrates the variation in analytical results both when the same sample is sent to several laboratories, and between several analyses of the same sample. Secondly, we may specify an analytical accuracy which is not realistic in the perspective of the monitoring system as a whole. For example, we may be uncertain of the precise origin of the sample, the extent to which the sample reflects conditions in the aquifer, or the degree of change which has occurred between sampling and analysis. In this context, high accuracy (at high cost) is usually spurious. Finally, the number of samples and determinands should be minimised once the period and amplitude of the important changes, normally after at least one hydrological year, have been identified. Ramon (pers. comm 1980), in describing groundwater monitoring in the Agence Financiere de Bassin Rhin-Meuse, has emphasised this aspect and found that failure to review monitoring needs could lead to analytical costs being six times higher than necessary, a significant proportion of the overall monitoring cost.

6 Conclusions

- Monitoring is an essential element of groundwater quality and resource management. As such, its objectives should be clear when all the interested agencies have made their requirements known.
- Insufficient effort is expended on defining the hydrogeological setting. Schemes planned without an adequate understanding will almost inevitably be unnecessarily costly and also provide misleading data.
- Monitoring methods used today are not usually technically adequate, either because the sampling method is unsuitable, or there is no appreciation by the users of the factors effecting the reproducibility of analytical results.
- New sampling techniques promise great improvement on the old traditional techniques. Methods such as the 'in-situ' gas-driven ejection sampler offers better and less costly groundwater sampling.

References

Anon, 1980. Effects of Effluent Recharge of the Major Aquifers; Final Report to the Dept. of the Environment. LR 1179, Water Research Centre, Medmenham, United Kingdom. 16 pp. + App.

Colchin, M.P., Turk, L.J., & Humenick, M.J. 1978. Sampling of groundwater and monitoring data for in-situ processes. University of Texas Centre for Research in Water Resources, Tech. Rept. EHE 78/01 CRWR-157, 121 pp.

Edworthy, K.J. & Baxter, K.M. 1981. Virology of Wastewater Recharge: Part I - Hydrogeology and Sampling. In "Viruses and Wastewater Treatment", Eds. M. Goddard & M. Butler, Pergamon, Oxford, pp 53-58.

Edworthy, K.J. & Brown, S.L. 1976. "An in-situ groundwater sampler". Water Research Centre, Medmenham, United Kingdom, ILR 611, 10 pp.

H.M.S.O. 1978. British Patent No. 1527751 "Groundwater Sampler". The Patent Office, Dept. of Trade and Industry, London.

Marsh, J.M. & Lloyd, J.W. 1980. Details of hydrochemical variations in flowing wells. Ground Water 18:4, 366-373.

McMichael, F.C. & McKee, J.E., 1965. Report of Research on Wastewater Recharge at Whittier Narrows. W.M. Keck Lab. of Env. Health Eng., Caltech, Pasadena.

Parizek, R.R. & Lane, B.E. 1970. Soil water sampling using pan and deep pressure-vacuum lysimeters. J. Hydrol., 11, 1-21.

Rightmire, C.T. 1978. Changes in formation gas composition and isotope content as indicators of unsaturated-zone chemical reactions related to recharge events. Int. Atomic Energy Auth. Symposium, Germany, June 1978, Publication IAEA-SM-228/35.

Robin, M.J.L., Dytynyshyn, D.J., & Sweeney, S.J., 1982. Two gas-drive sampling devices. Ground Water Monitoring Review, (Winter) 1982, 63-66.

Sarma, V.V.J., Prasad, N.V.B.S.S., & Prasad, P.R. 1979. The effect of hydrogeology on variations in the electrical conductivity of groundwater flüctuations. J. Hydrol. 44, 81-87.

Satchell, R.L.H. & Edworthy, K.J. 1972. Trent Research Programme. "Artificial Recharge: Bunter Sandstone Vol. 7, Water Resources Board (HMSO) 1972.

Wilson, A.L. 1982 "Design of Sampling Programmes", in Examination of Water for Pollution Control, Vol. 1, Editor M. Suess, WHO Reg. Office for Europe, Pergamon, Oxford; 360 pp.

Slawson, G.C. Kelly, K.E., Everett, L.G. 1982. Evaluation of ground-water pumping and bailing methods - applications in the oil-shale industry. Ground Water Monitoring Review 2:3, 27-32.

Wood, W.W. 1975. Guidelines for collection and field analysis of groundwater samples for selected unstable constituents. U.S. Geol. Surv., Techniques of Water Resources Investigations Book 1, Chap.D2,24 pp.

Summers, W.K, 1972. Factors affecting the validity of Chemical Analyses of Natural Water. Ground Water 10, 2; 12-17.

Water Resources Board 1974. Artificial Recharge of the London Basin: Pilot Works in the Lee Valley, Water Resources Board, Reading.

Wilkinson, W.B. & Edworthy, K.J. 1981. Groundwater Quality Monitoring - Money Wasted? Science of the Total Environment 21, 233-246.

Wilson, L.G. 1982. Monitoring in the Vadose Zone: Part II. Ground Water Monitoring Review, 2,1; 31-42.

CONTROLE DE LA RECHARGE ARTIFICIELLE DANS UN AQUIFERE SUREXPLOITE

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Résumé

Le long de la côte orientale de la Sicile, dans le département de Syracuse, l'exploitation des nappes d'eau souterraine , à la suite de la construction d'importantes industries pétrochimiques, a atteint des pointes très élévées, nettement supérieures à la remarquable capacité de recharge des aquifères, ce qui a entrainé la destruction des réserves, l'abaissement de plus de 100 m du niveau piézométrique original, en portant ce niveau à plus de 70 m au-dessous du niveau de la mer. Dans ces conditions on a commencé à enregistrer une intrusion d'eau de mer. Puisqu'il était nécessaire de préserver les caractéristiques des eaux souterraines qui, à part l'usage industriel, dans la zone sont aussi employées comme eau potable et pour des usages agricoles, on a réalisé une station expérimentale de réalimentation artificielle capable d'injecter dans les aquifères une portée de 350 l/s d'eaux provenant d'un fleuve qui coule plus au nord, ces eaux étant opportunément traitées au préalable.
Pour observer les effects provoqués par l'exécution de la réalimentation artificielle, on a mis sous observation plus de 60 puits, sur beaucoup desquels on a installé des appareils de contrôle permanent tels que piézographes, salinographes, thermographes. La relation illustre les conditions locales, les exigeances d'acquisition des données pour la reconstruction tridimensionnelle des aquifères et pour l'exécution d'un modèle mathématique et les problèmes pratiques qu'on a dû affronter pour le choix, la mise en œuvre et la gestion des instruments de contrôle.

Abstract

Along the east coast of Sicily, in the Syracuse district, following the settlement of huge oil-refineries exploitation of aquifer has reached very high peaks, clearly superior to the consistent recharge capacity of the aquifer. This has provoked the destruction of reserves, the lowering of more than 100 m of the original piezometric level, this level being brought below 70 m under the sea level. Under these circumstances we have begun to record an intrusion of sea water. As it was necessary to preserve the subterranean waters characteristics - since these waters apart from the industrial use, are employed for domestic and agricultural ends, we have carried out an experimental plant for artificial recharge, capable of injecting into the aquifer a flow of 350 l/s of conveniently treated waters drawn for a river which flows in a neighbouring northern district. Both to check the effects of the artificial recharge, we have kept under control more than sixty wells, and on many of them we have installed permanent checking devices (piezographs, salinographs, termographs). This report explains the local conditions, the way we obtained the data we needed to bring about a tridimensional reconstruction of the aquifer and the drawing of a mathematical model, as well as the practical problems we had to tackle to choose, to install and to manage the control devices.

1 Introduction

La récharge artificielle d'une couche aquifère, peut être réalisée par des moyens naturels aussi bien qu'artificiels. Dans le premier cas il s'agit de trouver des zones particulièrement perméables, aussi bien par porosité que par fissuration, sur lesquelles on concentre, moyennant des barrages ou en creusant des fossés ou des canaux, les eaux superficielles qui ensuite, lentement, seront absorbées par le terrain et iront grossir les nappes sous-jacentes; dans le deuxième cas il s'agit d'introduire de l'eau en profondeur à travers des puits d'injection. A l'apparence l'edée est simple, mais en réalité les problèmes à affronter et à résoudre sont nombreux et complexes: quelques uns dépendent des caractéristiques de l'eau qu'on utilise pour effectuer la recharge, d'autres sont liés aux caractéristiques de l'aquifère dans laquelle il faut injecter l'eau. Et c'est surtout ce dernier problème qui présente la majeure partie des inconnues. Une fois la nappe aquifère atteinte, comment se comportera l'eau injectée, elle restera stockée dans la nappe même ou de celle-ci elle s'écoulera plus ou moins rapidement vers d'autres zones? le temps de séjour dans la nappe sera-t-il tel qu'il en consentira l'exploitation pendant un certain temps? Quelles interactions vont-elles se vérifier entre l'eau injectée et l'eau déjà présente dans la nappe, surtout si celle-ci est en contact direct avec la mer? Ces points d'interrogation permettent, une fois résolus, d'atribuer une rentabilité effective à l'opération de recharge et d'en justifier la réalisation.

Pour résoudre ces problèmes, pourtant, il ne suffit pas de quelques considérations hypothétiques même si mathématiques ni de quelques observations casuelles: il faut des contrôles précis et continus, il faut des appareillages capables d'exécuter des mesurations: du niveau statique et dynamique des nappes, de la température de l'eau, de la teneur en chlorures, et ces mensurations doivent être effectuées dans une telle quantité et selon une distribution géographique telle qu'elles consentent le contrôle du comportement de la nappe aquifère dans toutes ses parties.

Les mensurations et les contrôles sont plus faciles à interpréter, bien sûr, quand on est en présence d'une couche perméable par porosité tandis qu'il est beaucoup plus difficile de les interpréter quand la nappe est du type à perméabilité par fissure avec ou sans phénomènes karstiques.

Pendant la rédaction du projet et la construction d'une grande installation expérimentale de réalimentation artificielle réalisée en Sicile, dans la zone comprise entre la ville d'Augusta et la ville de Syracuse, on a considéré ces problèmes.

2 Puisage d'eaux souterraines et mode d'exploitation

Le long de la côte orientale de la Sicile, dans la zone comprenant les territoires d'Augusta, Priolo et Syracuse, on a bâti, pendant les vingt dernières années, un ensamble d'industries pétrochimiques de grandes dimensions. Faute d'installations publiques d'alimentation hydrique les industries pourvoient à leur approvisionnement hydrique par des puits. Sur une surface d'environ trente km^2, on a creusé 162 puits, dont 121

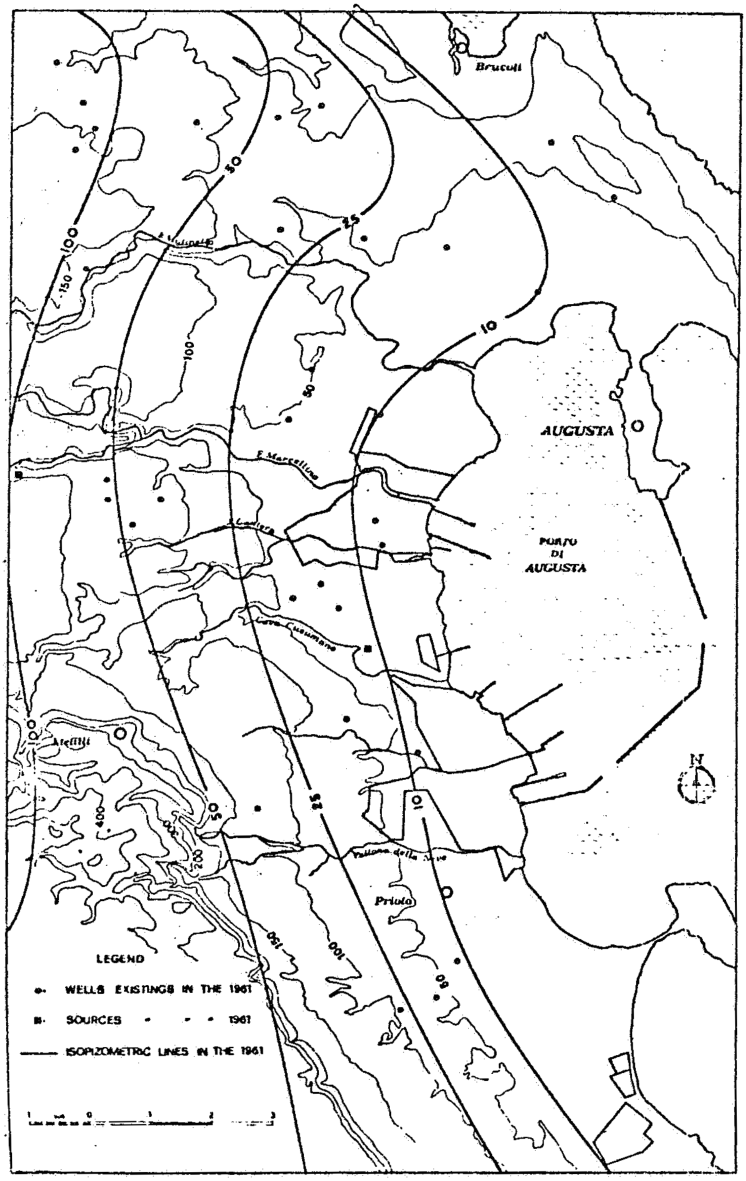

Figure 1 Niveaux piézométriques en 1961

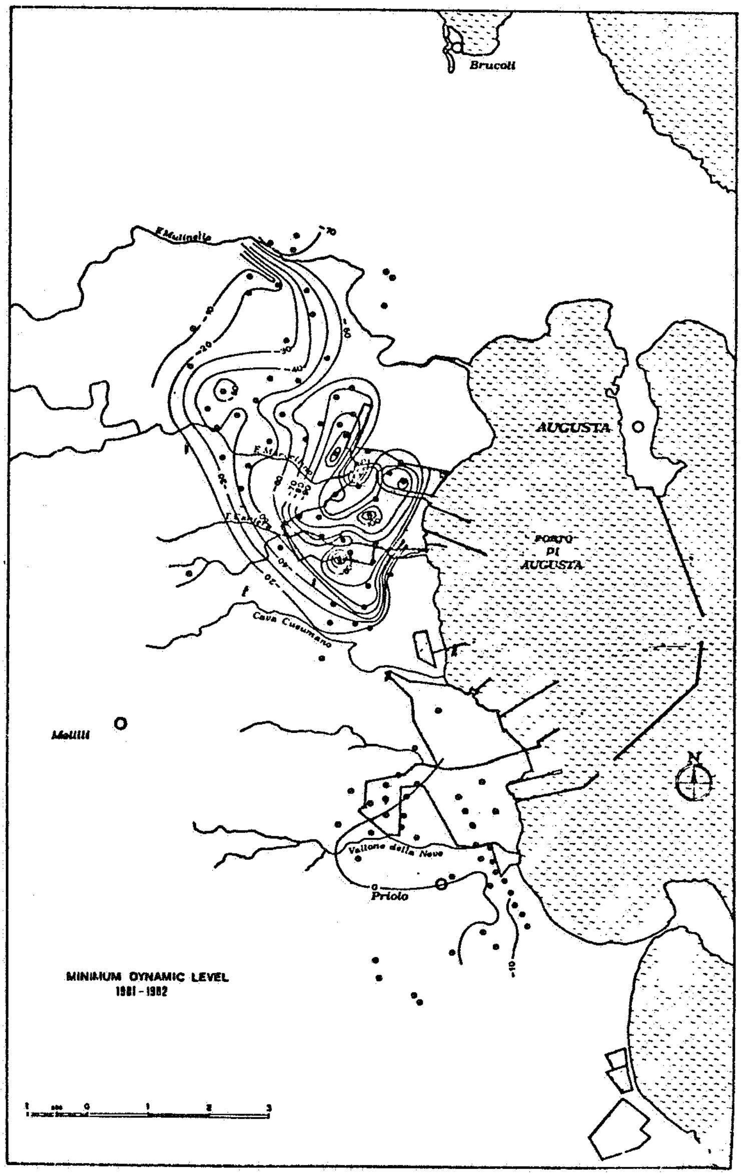

Figure 2 Niveaux dynamiques minimales 1981 - 1982

sont employés de façon permanente et 41 seulement de temps en temps. Dans l'ensemble on estime à 51 $m^3 \times 10^6$ par an la quantité d'eau tirée des nappes pour des usages industriels et potables, tandis que 13 $m^3 \times 10^6$ par an sont utilisés pour l'irrigation. Sur la base de recherches géologiques, géophysiques et hydrologiques, et en utilisant aussi un modèle mathématique construit exprès, on est arrivé à estimer à 53,790 $m^3 \times 10^6$ le volume moyen annuel à attribuer à l'alimentation naturelle. Il en résulte donc un déficit, c'est à dire un volume soustrait aux réserve, de 10,210 $m^3 \times 10^6$ par an. Dans l'ensemble on estime à 120 $m^3 \times 10^6$ le volume déjà extrait des réserves.

La preuve de cette exploitation irrationnelle a été donnée par l'abaissement extraordinaire des niveaux piézométriques, qui dans la région sont passés, pendant la période 1961-1982, (Figures 1&2) de valeurs toujours positives à des valeurs négatives qui dans quelques endroits, ont dépassé moins 80 ÷ 90 m au-dessous du niveau de la mer. Il s'est ainsi créé une sorte d'entonnoir qui constitue un appel pour toutes les eaux contenues dans la nappe aquifère, et en particulier pour les eaux de la mer le long de la côte, Figure 3.

3 Constitution hydrogeologique

La géologie de la zone est caractérisée par la présence des termes lithologiques suivants, de bas en haut:

- Calcaires à Anphistegine de l'Oligo-Miocène
- Roches volcaniques miocènes ou supra-miocènes
- Alternance calcaire-grès calcaires calcaires marneux du Miocène supérieur
- Roches volcaniques plio-pléistocènes
- Grès calcaires plio-pléistocènes
- Argiles Pléistocènes
- Sables et grès calcaires pléistocènes
- Alluvions terrassée
- Alluvions récentes.

Les nappes aquifères identifiées sont trois.

La première est constituée des alluvions, des sables et des grès calcaires quaternaires. Elle présente un intérêt assez modeste parce qu'elle

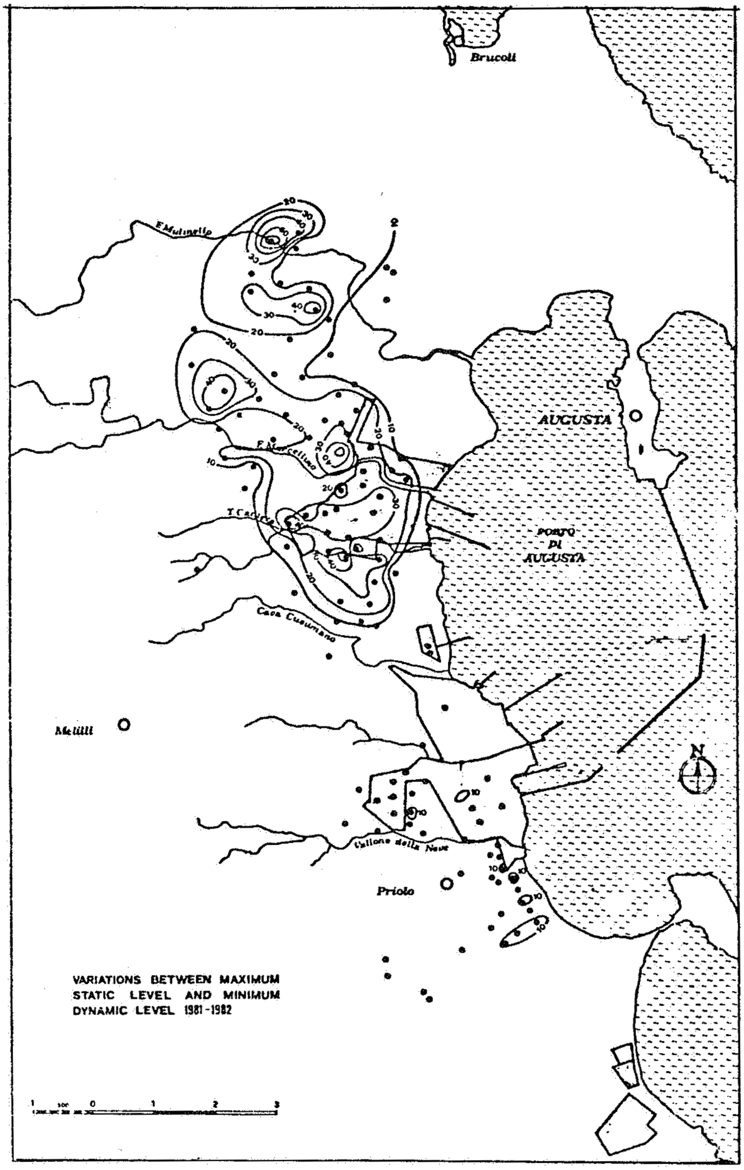

Figure 3 Variations entre niveaux statiques maxinales et niveaux dynamiques minimales 1981 - 1982

est maigrement alimentée et parce qu'elle est drainée par les cours d'eau qui la traversent. Son lit est constitué d'argiles gris-bleues pléistocénes.

La deuxième est constituée de grès calcaires plio-pléistocènes et des roches volcaniques du même âge. Son lit est constitué de tufs très altérés et de l'alternance calcaire-marneuse du Miocène supérieur.

La troisième nappe acquifères, la plus importante, est constituée de calcaires oligo-miocènes. Presque tous les puits de la zone sont alimentés par la nappe contenue dans cette nappe aquifère.

La présence de nombreuses failles met souvent en contact la deuxième nappe aquifère avec la troisième et rend impossible de parvenir, sur la base des connaissances actuelles, à une définition précise des caractéristiques de chacune d'elles.

Les argiles pléistocènes sont distribuées le long toute la ligne de la côte qui entoure la baie d'Augusta, et avec leur conformation en coin qui se prolonge au-dessous de la surface de la mer ainsi constituent un septum imperméable (aquiclude) qui sépare les terrains perméables quaternaires superficiels des terrains perméables pliocènes et miocènes sous-jacents. Cette favorable condition de gisement a empêché ou du moins remarquablement diminué l'intrusion d'eau de mer dans les deux couches aquifères inférieures, et en particulier a empêché que l'appel déterminé par la dépression piézométrique qui s'est vérifiée juste en correspondance avec le septum argileux se manifestât par un afflux d'eau salée normale à la ligne de la côte. L'ampleur limitée du septum argileux fait pourtant en sorte que l'appel déterminé par la dépression piézométrique se manifeste latéralement, si bien qu'à présent on observe un enveloppement à nord et à sud du septum argileux et les eaux salée sont déjà arrivées à polluer de nombreux puits placés à la périphérie de la région en question, là où les couches aquifères inférieures sont en contact direct avec les terrains superficiels perméables.

La présence de plusieurs couches aquifères superposées, caractérisées par la présence d'hétéropies de facies et par une pérméabilité essentiellement par fracture, entrainerait une interprétation de type multicouche, mais comme les couches aquifères sont fréquemment mises en contact par failles il faut considérer la couche aquifère, dans son ensemble, comme une mono-couche.

L'existance de conduits karstiques, qu'on n'a pas observés mais dont on a induit l'existence moyennant des traceurs, rend encore plus complexe la reconstruction tridimensionnelle de la structure géologique locale.
La couche aquifère, dans son ensamble , a des caractéristiques de grande hétérogénéité, aussi bien verticalement qu'horizontalement, et les quelques données qu'on possède à l'égard de la transmissivité, et qu'on a obtenues par de rares essais de portée, ne peuvent pas facilement être mises en relation entre elles.
Les valeurs de T obtenues varient de 0,69 à 11,5 x 10^{-3} m^2/s. les valeurs du coefficient de stockage sont pratiquement inconnues.

4 Ré-alimentation par injections dans les nappes

Dans les conditions qu'on vient d'exposer on a dû prévoir des interventions capables de modifier la situation qui s'était produite, et qui était extrêmement grave.
Les interventions déjà réalisées prévoient:

a) l'utilisation des eaux du Simeto, un fleuve important qui coule au nord de la zone en question, pendant les quatre mois d'hiver, et cela dans les quantités disponibles et avec la limitation déterminée par le débit de la canalisation, c'est à dire 3,5 m^3/s;
b) la clarification de ces eaux moyennant des installations spécifiques;
c) la distribution des quantités d'eau disponibles, dérivées du Simeto, pendant la période de l'hiver, de l'usine de clarification aux industries;
d) l'injection dans la nappe, à travers des puits opportunément choisis parmi ceux qui existent, de la quantité d'eau maximum que ces puits peuvent absorber, en tenant compte de la disponibilité des eaux du Simeto;
e) le contrôle constant - à travers des instruments et à travers un modèle mathématique - des variations subies par la nappe.

La recherche préliminaire est partie de l'étude des caractéristiques des puits existant dans la zone, et le choix est basé sur deux éléments; l'emplacement de ces puits et la quantité de renseignements qu'on avait sur chaque puits.

L'emplacement a été considéré utile quand le puits se trouvait très près de l'endroit où l'on avait observé l'abaissement maximum du niveau piézométrique.

Les renseignements sur chaque puits ont été considérés suffisants quand la quantité de données connues permettait d'estimer le comportement présumé du puits si on l'utilisait pour exécuter l'alimentation artificielle.

Les puits qu'on a pris en considération ont successivement été divisés en deux groupes:

1) Puits aptes pour des opérations de recharge artificielle;
2) Puits aptes à fonctionner comme piézomètres et pour y installer des salinomètres;

A la fin de toutes les vérifications on en est arrivé à reconnaitre 13 puits comme appartenant au premier groupe et 39 puits comme appartenant au deuxième groupe.

Les treize puits du premier groupe ont été reliés par des tuyaux en acier à l'usine de clarification qui est capable de fournir plus de 350 l/s rien que pour l'usage de la réalimentation. Sur chaque puits on a installé un mesureur de débit et un mesureur de pression, reliés à un appareil enregistreur, et on a aussi prévu la possibilité d'augmenter la pression à la bouche du puits par l'introduction d'une pompe capable de fournir une charge additionnelle de quinze atmosphères.

Sur les 39 puits appartenant au deuxième groupe, 20 étaient déjà fournis d'une installation manométrique capable de donner des indications sur les niveaux piézométriques; sur treize on a installé un piézographe du type à pression, sur six puits on avait prévu d'exécuter les mensurations par une petite sonde électronique.

Les salinographes installés, ont été trois. Les sondes thermiques achetées ont été deux, Figure 4.

5 Installation pilote

Puisque pourtant les expériences ainsi réalisées se limitaient à utiliser des puits déjà existant, et sur lesquels on avait seulement des connaissances insuffisantes, on a prévu de réaliser aussi une installation pilote dans le cadre des expériences générales qu'on a déjà décrites.

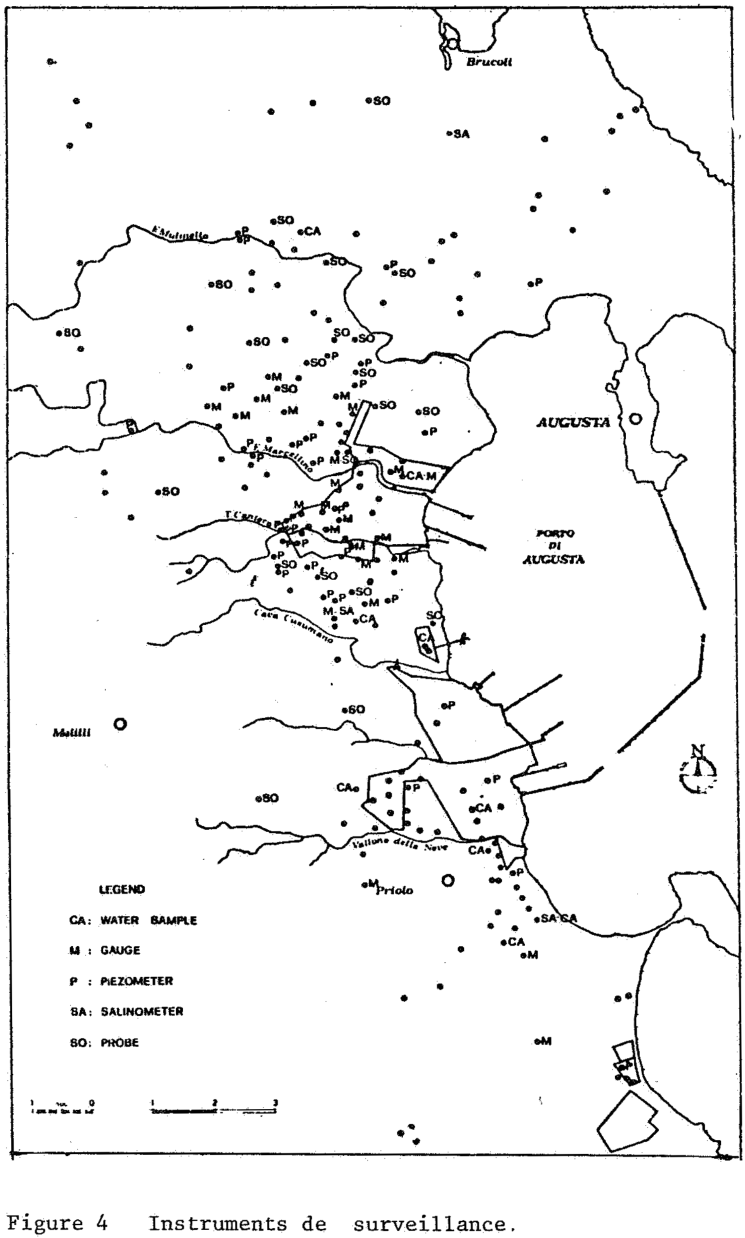

Figure 4 Instruments de surveillance.

Cette installation pilote consiste dans le creusement d'un puits profond 300 m et avec un diamètre de 300 mm finaux, entouré de douze puits piézométriques (Ø 100 mm) placés selon le schéma le plus classique, c'est à dire par groupes de trois sur les bras d'une croix. Sur chacun des douze puits piézométriques on est en train d'installer un piézographe. Dans le puits principal on exécutera aussi bien des soigneux essais de débit que des essais d'injection d'eau pour la recharge des nappes.

Les difficultés à vaincre ont été nombreuses, aussi bien au moment du projet qu'au moment de l'exécution.

La première difficulté qu'on a rencontrée a été l'acquisition des données sur les puits existants. Beaucoup de ces puits, étant dépourvus d'une autorisation officielle, étaient tenus secrets par les propriétaires; pour ces puits, de toute manière, on avait très peu de renseignements aussi bien sur la stratigraphie rencontrée au moment du creusement que sur les crépines qu'on avait installées.

On a eu beaucoup de peine à convaincre et à intéresser les titulaires, spécialement les industries, pour arriver à achever le recensement. Une autre grande difficulté était donnée par l'emplacement des puits, qui souvent se trouvaient à l'intérieur de zones intéressées par des appareillages industriaux (raffineries), ou à côté de grands réservoirs pour le stockage des hydrocarbures. Il n'a pas toujours été possible de relier ces puits avec l'usine de clarification à travers des canalisations en acier, et en tout cas il aurait manqué l'espace nécessaire pour effectuer les contrôles et mener l'expérience.

Une fois qu'on a procédé au choix de treize puits d'injection et de 39 puits piézométriques, dans la phase exécutive on a dû affronter d'autres problèmes.

Dans trois des puits d'injection on s'aperçut que le revêtement des tuyaux était fissuré, même à côté de la surface, si bien qu'au moment de la recharge on vit soudain l'eau sortir du terrain autour de la bouche, et il s'avéra impossible de mettre ces puits sous pression. Il fut nécessaire de procéder à cimenter le contour du tuyaux de revêtement jusqu'à atteindre les argiles du substrat. Dans d'autres puits, sur lequels on voulait installer un petit tuyau manométrique, il a été nécessaire d'extraire les pompes qui, maintenues à l'arrêt, auraient empêché les mensurations du niveau, et cela parce que les tuyaux attachés à la pompe, au lieu d'être soudés, étaient reliés par des collets qui obstruaient toute la place existant entre les tuyaux de revêtement et les tuyaux d'exploitation. Un problème d'un type différent a été déterminé par la nécessité de mesurer des excursions, entre les niveaux statiques et les niveaux dynamiques, dépassant même 50 m Fig. n. 3. Avec les instruments qui existent, pour obtenir une bonne précision, le rapport d'echelle devrait être contenu au maximum à l'interieur de 1/10. Avec un rapport de 1/50, sur une excursion de 10 m si rapporté à une ampleur d'enregistrement de 20 cm, un millimètre résulte en effet correspondre à une excursion de 5 cm. Sur un manomètre étalonné à fond d'echelle une atmosphère, il résulte impossible de lire des variations dépassant 1/100, qui de toute façon correspondent à 10 cm. Dans les conditions où l'on s'est trouvé à opérer, des mensurations avec la précision du centimètre sont pratiquement impossible avec des instruments manométriques ou avec des instruments à flotteur, ou du type "strain-gauge", à part la difficulté de fonctionnement des instruments quand le niveau piézométrique à mesurer se trouve 90 - 100 mètres plus bas que l'instrument enregistreur lui-même. On a dû décider de limiter la précision des mensurations de niveau à 10 cm, en ayant soin de reporter le stylo enregistreur au fond de l'échelle chaque fois que la variation piézométrique dépassait les 10 m. En ce qui concerne les mensurations manométriques, comme il était impossible de faire varier chaque fois la profondeur d'immersion du petit tuyau dans les puits, on a du recours à l'installation de cinq petits tuyaux dont les bouts étaient placés chacun 10 m plus bas que le précédent. Une batterie de manomètres reliés aux petits tuyaux par des robinets spéciaux consent d'exécuter des mensurations de niveau en passant d'un manomètre à l'autre suivant la variation du niveau piézométrique.

Un autre problème qu'on est encore en train de résoudre est celui posé par la nécessité d'effectuer des contrôles sur les phénomènes de subsidence des terrains liés à la surexploitation des nappes. Des contrôles de la compacité des terrains de la zone, par mensurations extensométriques, sont prevus dans les puits contrôlés.

Les expériences, qu'on a déjà entreprise et qu'on vient de décrire, coûteront environ un milion de dollars, distribués ainsi:

Appareillage	130.000 $
Travaux (creusement, puits et tuyauteries)	470.000 $
Modèles mathématiques	60.000 $
Personnel pour le contrôle et la gestion des instruments (pendant trois ans)	250.000 $
Frais Généraux	90.000 $

Le coût de la clarification de l'eau, évalué en raison de 230 lires italiennes par mètre cube ajoutera, en considérant une recharge annuelle de 4.000.000 m^3, encore 600.000 U.S. Dollars en moyenne par an.

THEME 3:

METHODS AND TECHNIQUES FOR THE DETERMINATION OF GEOHYDROLOGICAL PARAMETERS AND VARIABLES

METHODES ET TECHNIQUES PERMETTANT DE DETERMINER DES PARAMETRES ET VARIABLES GEOHYDROLOGIQUES

PRACTICAL APPLICATION OF CONTINUUM APPROACH TO CHARACTERIZE THE POROSITY OF CARBONATE ROCKS

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Abstract

The statistical distribution of the size of fractures is needed for the determination of the hydraulic behaviour of solid rocks. The application of the continuum approach requires also some knowledge about the variability of the fractures. A detailed investigation was carried out, therefore, to describe the linear porosity and fracture-size distribution in four different carbonate formations (two dolomites and two limestones). The paper summarizes the numerical results of the field work, which is the only one practical application of the continuum approach known from the literature.

1 The basic concept of continuum approach

The idea concerning the application of continuum approach to characterize porous media was raised in 1965 (Bachmet) and since that time many important theoretical conclusions were derived on the basis of this theory (Bear, 1972).

The extremely wide fluctuation of porosity calculated for a very small sample can be mentioned as an example proving that there is a limit of the size below which the determined physical parameters can not be accepted as the average characteristics of the porous medium (i.e. representative elementary unit). This limitation can be observed especially in hard rock formations, like dolomites and limestones. Therefore, it can be stated that the precondition of the application of the continuum approach (hence the use of the various seepage laws giving relationship between the gradient of potential and seepage velocity) is the determination of the size of the representative elementary unit.

When porosity is determined for a point, its value is 1 or 0, depending on the position of the point. The variance decreases with the increase of the investigated sample and tends to a relatively small constant variance around the expected value characterizing the random character of the internal structure of porous media. The investigation repeated several times gives a set of the expected values having only insignificant variance. Finally, the porosity can be characterized by the probability distribution of the expected values and by the random variation determined in this way.
The purpose of the research the result of which is summarized in this paper was the determination of the representative elementary unit for several carbonate rocks giving some guidance for the practice in this way to determine the applicability of continuum approach.

2 The execution of the research

The linear porosity was chosen as basic parameter to characterize the porous medium, which was fractured carbonate rock in the research. Linear porosity is the sum of the widths of fractures measured along a given line related to the total length of the line.
The linear porosity was determined in quarries on the fresh surfaces of four different carbonate rocks of Triassic age (L. Balashazy and J. Kovacs, 1975). At every place three directions (one vertical and two horizontal, approximately perpendicular to each other) were fixed on the walls of the quarry, to show the anisotropy of the rocks in the different directions. These directions were influenced by the conditions of the quarry and, therefore, the angles between the directions of the horizontal measuring lines and the north direction were different at the various places. The investigated lines were marked on the surfaces of the rocks. Where it was possible, the measurements were made along one con-

tinuous line. Where the size of the wall excluded the use of one line two or more smaller lines parallel to each other were used. In the vertical direction this second solution had to be applied in all cases. The size of the openings was measured by magnifying device. In this way the minimum size directly measured was 0.35 mm, and we were able to estimate the half of this length (0.18 mm). In the case of smaller openings only the number of the openings was counted.

The research was made on more or less smooth surfaces to obtain the exact place and width of the openings. Bedding- and fault-planes were the smoothest surfaces giving the possibility to choose the measured lines in the tectonically characteristic directions. The absolute width of fissures was measured in the cross-section of the line, even in cases when the size of the gap between the walls was varying along the fractures.

The measurements were executed in four different carbonate formations:

a) Obarokk - dolomite with fine structures of joints of Upper Triassic age.
b) Csolnok - dolomite bedded limestone of Triassic age.
c) Szar - dolomite of Upper Triassic age in fault-zone.
d) Dorog - karstic limestone of Upper Triassic age (Dachstein formation).

3 The result of the research

The data determined as the results of the investigation (i.e. n_L linear porosity calculated for measuring lines having different length) were plotted in a coordinate system, where the investigated length was used on the horizontal axis and the determined linear porosity on the vertical one. The enveloping curves enclosing the range of the scattering of the points tend to a horizontal line, indicating the average linear porosity.

The investigated length, above which the scattering of n_L is smaller than 10 percent of the average linear porosity and where the enveloping curves become almost horizontal, was accepted as representative elementary length characterizing the rock in the investigated directions. This representative elementary length was determined in three orthogonal di-

Table 1 The determined data at various locations

Location	Formation	Measured directions	Representative elementary length	Porosity (%)		
				linear n_L	areal n_A (perpendicular to the direction)	volumetric n
Obarokk	dolomite with fine structures of joints	x - 5^o - 185^o y - 96^o - 275^o z - vertical	0.24 ~ 0.32 0.24 ~ 0.60 0.24 ~ 0.32	1.46 2.51 1.88	(yz) 4.34 (xz) 3.27 (xy) 3.86	5.69
Csolnok	dolomite bedded limestone	x - 69^o - 249^o y - 159^o - 339^o z - vertical	1.2 ~ 3.2 1.6 ~ 2.4 0.8 ~ 2.4	1.55 0.80 1.05	(yz) 1.79 (xz) 2.48 (xy) 2.29	3.26
Szar	dolomite, situated in a fault-zone	x - 120^o - 300^o y - 10^o - 190^o z - vertical	30.0 ~ 34.0 5.0 ~ 30.0 ~ 5.0	8.02 5.63 1.89	(yz) 7.39 (xz) 9.75 (xy) 13.15	14.80
Dorog	karstic limestone	x - 24^o - 104^o y - non measurable z - vertical	~ 30.0 - ~ 8.0	1.95 non measurable 0.66	(yz) - (xz) 2.50 (xy) -	-

rections for the four different rock types (Table 1).

Ten pairs of enveloping curves determined from the data of ten independent vertical measuring lines at Obarokk (fine structured dolomite) are represented in Figure 1. The comparison of these curves proves that they are very similar to each other and both the average linear porosity and the representative elementary length have only random variation. Hence, these parameters can be regarded as average values characterizing the rock at the given place and in the investigated direction.

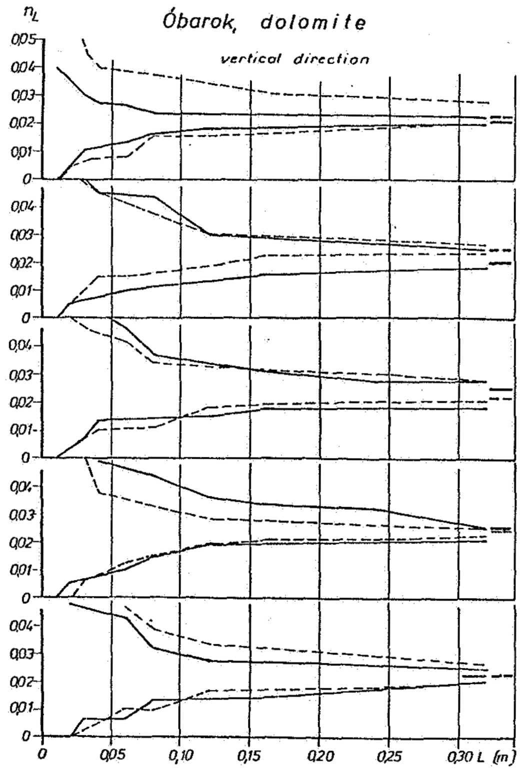

Figure 1 Comparison of curves enveloping the linear porosity values calculated along independent lines having the same direction

On the basis of the previous figure all the data determined at Obarokk are summarized in one figure and the enveloping curves belonging to the three different directions were constructed (Figure 2). Since there were several mean values calculated for different stretches, the expected value and the variance of these parameters were also calculated and the figure was supplemented by the characterization of the probability distribution of average linear porosity.

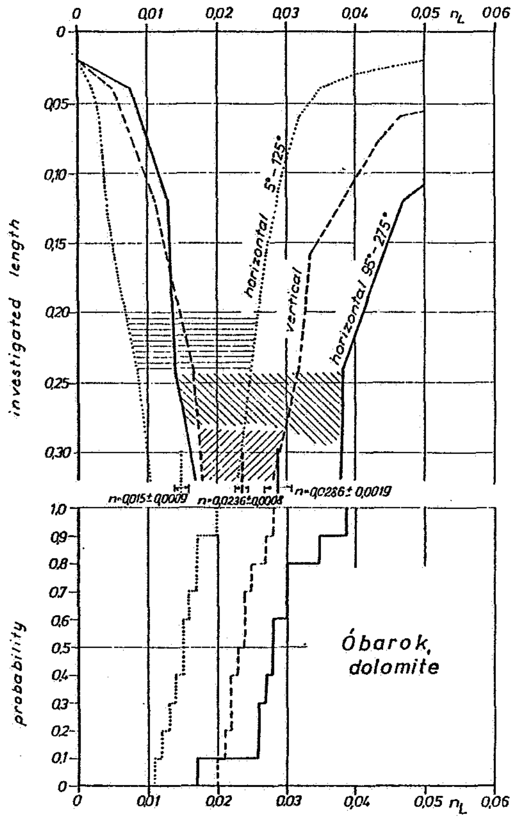

Figure 2 Determination of average linear porosity and representative elementary length (dolomite, Obarokk, Hungary)

The same analysis was carried out with the data collected at a quarry near Csolnok (dolomite bedded limestone). Here the linear porosity values determined in the vertical direction and in one horizontal direction respectively have shown great similarity while in the other horizontal direction the parameter was two times larger than the previous ones (Figure 3). This fact indicates that the openings are closed under the effect of gravity and due to tectonical pressure, but perpendicular to the main direction of the pressure the fractures remained open. It is necessary to note, that the statistical evaluation was executed combining the vertical and one horizontal direction, but the same analysis was not carried out in the third direction, because of the small number of data.

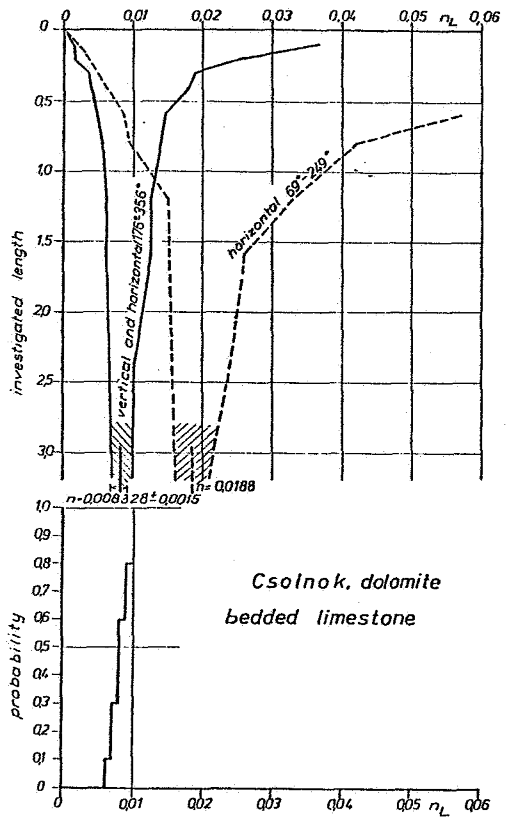

Figure 3. Determination of average linear porosity and representative elementary length (dolomite bedded limestone, Csolnok, Hungary)

At the third place of the investigation (Szar) the same dolomite formation was studied as in the first example (Obarokk). The quarry was, however, in a fault-zone, and therefore a secondary, very intensive fractured system was observable. The joints were open in both horizontal directions and closed along the vertical measuring line (Figure 4). Unfortunately the walls of the quarry were not large enough to reach the representative elementary length. It was shown, however, by the trend of the enveloping curves, that the representative elementary length is at least ten times longer in vertical direction and about hundred times longer in horizontal direction, than the same parameter at Obarokk.

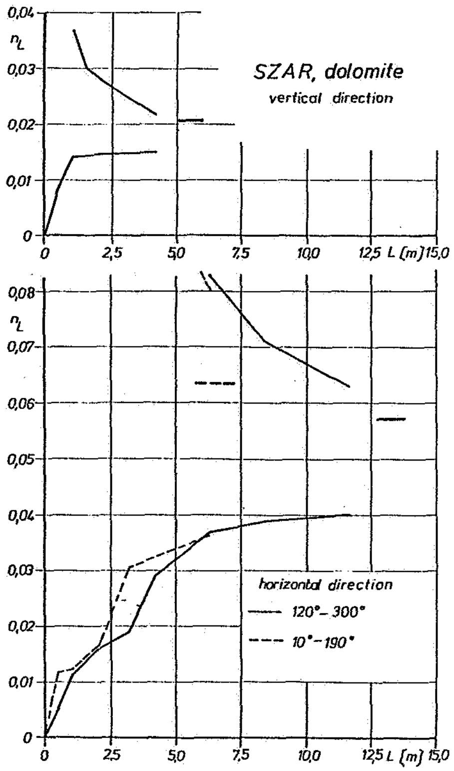

Figure 4 Determination of average linear porosity and representative elementary length (dolomite, Szar, Hungary)

Finally, Figure 5 represents the results of the same analysis in a karstic limestone (Dorog). The properties of this formation is very similar to that of the faulted dolomite. Considering the relationship existing between linear, areal and volumetric porosity, all the parameters can be calculated if the linear porosity values are known in three orthogonal directions (n_{Lx}; n_{Ly}; n_{Lz}).

Areal porosities in planes normal to the x-axis (n_{Ax}), the y-axis (n_{Ay}) and the z-axis (n_{Az}) are:

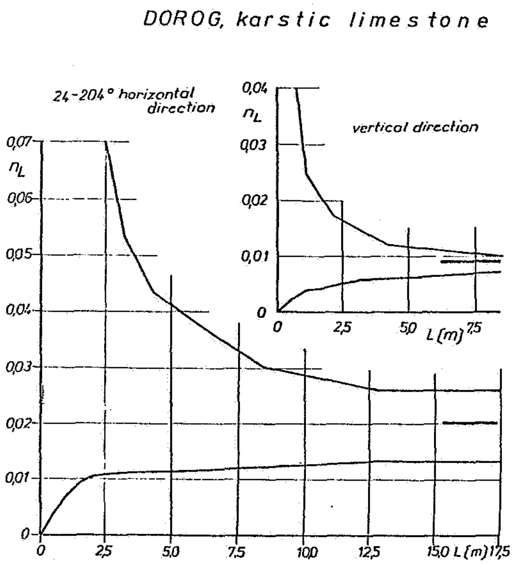

Figure 5 Determination of average linear porosity and representative elementary lenght (karstic limestone, Dorog, Hungary)

$$n_{Ax} = 1 - (1 - n_{Ly})(1 - n_{Lz}) + n_{Ly}n_{Lz}$$

$$n_{Ay} = 1 - (1 - n_{Lz})(1 - n_{Lx}) + n_{Lz}n_{Lx}$$

$$n_{Az} = 1 - (1 - n_{Lx})(1 - n_{Ly}) + n_{Lx}n_{Ly}$$

The volumetric porosity (n) can be similarly calculated:

$$n = 1 - (1 - n_{Lx})(1 - n_{Ly})(1 - n_{Lz}) + n_{Lx}n_{Ly}n_{Lz}$$

The areal and volumetric porosity values determined on the basis of the investigation discussed here as an example, are also listed in Table 1.

ESTIMATION OF HYDROGEOLOGICAL PARAMETERS USING WELL-FIELD EXPLOITATION DATA

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Abstract

The hydrogeological parameters in the region of operating well-fields can be estimated by using steadily recovered head data of separately stopped pumped wells and available observation well data. The initial water head, transmissibility and recharge parameters, in case of bank filtration and vertical leakage, can be determined by minimizing the average absolute deviation between measured and computed heads using the FORTRAN IV program PAROP.

1 Introduction

The most reliable values of hydrogeological parameters can be determined from the data, measured at the waterworks discharging for a long time with a high rate. In this case the nearly steady-state cone of depression has a large area and depth, respectively, and characterises all the main recharging factors and processes.

The hydrogeological parameters to be determined, are as follows:

- the initial water level or head, H_o, of the aquifer influenced only by the external factors (changes in natural discharge and recharge, water-level fluctuations in rivers, other water-withdrawals etc.). It is assumed, that H_o is a constant value both over the area, where observations are made and during the time interval of measurements.Practically H_o is the head wich is to be reached theoretically after a long term stopping of all the pumped wells in the analysed well-field;

- the transmissibility, T, of the aquifer in the area of well-field;
- the recharge factors, L, such as the additional distance of the effective recharge boundary from the river bank in case of waterworks using bank filtration, and the vertical leakage coefficient b = K/m for the leaky aquifers (K,m - hydraulic conductivity and thickness of the covering semipermeable layer, respectively).

2 Theoretical background

In order to determine the above parameters the pumped wells must be stopped separately to measure the recovery-curve up to the stabilization of the piezometric head H_i. Water-level measurements should also be made in the available observation wells. The local values of the transmissibility and the screen-resistance of the wells can be calculated from non-steady-state recovery data. The regional values of H_o, T, L or b can be estimated by minimizing a functional F of the absolute mean difference between measured and calculated heads:

$$F = n^{-1} \sum_{i=1}^{n} |S_i - H_o + H_i| \qquad (1)$$

where

n = number of steady-state water-level or head measurements

H_i = measured water-level or head in the i-th observation or stopped pumped well

S_i = calculated value of the drawdown in the i-th well caused by the other wells, depending on the spacing and discharge of the pumped wells and the parameters T, L or b

$$S_i = \sum_{j=1}^{g} (Q_j/2\pi T)W(x_i,\ y_i,\ x_j,\ y_j,\ P) \tag{2}$$

where

g = number of wells operating during the measurement of H_i with coordinates x_i, y_i (axis y is directed along the river bank)

Q_j= discharge of the j-th pumped well with coordinates x_j, y_j

P = hydrogeological parameter; for bank filtration P = L, for leaky aquifer P = b/T

In case of recharge from the river (Forchheimer, 1886):

$$W(x_i,y_i,x_j,y_j,L)= (1/2)\ln\{[(x_i+x_j+2L)^2+(y_i-y_j)^2]/r_{i,j}^2\} \tag{3}$$

$$r_{i,j}^2 = [(x_i-x_j)^2+(y_i-y_j)^2] \tag{4}$$

For leaky aquifers (Steggewentz and Van Nes, 1939):

$$W(x_i,y_i,x_j,y_j,b/T) = K_o[r_{i,j}(b/T)^{1/2}] \tag{5}$$

where

$K_o(u)$ = zero order modified Bessel-function of second kind

A similar method was developed by Gudz and Polshkova (1981) for the interpretation of nonsteady-state drawdown and recovery data of test pumpings.

Using Eqs. (1) - (5) and by changing parameters in the real intervals by steps ΔH_o, ΔT, ΔL or $\Delta(b/T)$ the three-dimensional distribution of F with minimum F_{min} can be generated. The parameters obtained at this minimum give the minimal deviation of the computed heads from the measured data and are to be considered as the optimal values. The character of the distribution of F near F_{min} shows the sensitivity of the head distribution to the different parameters.

The results of special test computations enable one to analyse how the spatial distribution and the possible errors of field data do influence the value and reliability of the hydrogeological parameters. For example, it was found, that the accuracy of the recharge parameters L or b increases with the area of cone of depression, covered by observation data, while reliable value of T can be determined using data, measured in the central part of the cone of drawdown.

3 Computer method of parameter identification

A FORTRAN IV. computer program PAROP (Parameter Optimization) was developed for the computation of F as the function of H_o, T and L or b/T. The following input data are required:

- the coordinates, discharges and (recovered) heads of the observation or stopped pumped wells;
- the coordinates and discharges of pumped wells having no measured head data;
- the initial values and steps of increments for unknown parameters H_o, T, L or b/T.

During the procedure the previous value of each parameter increases by the step of increment 10 times. The output data are as follows:

- the list of input data;
- the list of coordinates and heads H_i of observation or

stopped pumped wells;

- the list of coordinates and discharges Q_j of pumped wells;
- the table of values S_iT for each well having head measurement and for each increment of L or b/T;
- the three-dimensional matrix F is printed as the series of tables. Each table, characterized by a given value of H_o, shows the distribution of F between minimum and maximum values of T and L or b/T.

The program PAROP was used to determine parameters H_o, T and b for well-field II. of municipal waterworks of the town Debrecen in Hungary. The well-field consisting of 22 wells occupies a relatively small area with a diameter of 2 kms. The mean value of the transmissibility from the nonsteady--state part of the recovery curves of the wells gives T=1770 m^2/day. During the optimization procedure the parameters were varied in intervals $82 \leq H_o \leq 92$ m, $1100 \leq T \leq 2100$ m^2/day, $10^{-9} \leq b/T \leq 1,1 \cdot 10^{-8}$ m^{-2} with F_{min} = 0,74 and F_{max} = 11,88 m. The optimal parameters were chosen at H_o = 92 m, T = 1400 m^2/day, b/T = $2 \cdot 10^{-9}$ m^{-2}, b = $2,8 \cdot 10^{-6}$ 1/m with F = 0,89 m as equally reliable for each parameter.

References

Forchheimer,Ph.1886. Über die Ergiebigkeit von Brunnenanlagen und Sickerschlitzen, Z.Architekt. Ing.-Verein. Hannover, vol.32. No.7.

Gudz V.I., Polshkova I.N. 1981. Computer method of test pumping data interpretation. In "Algorithms and Programs" VIEMS, Moscow pp. 43-54. (in russian)

Steggewentz J.H., Van Nes I.N. 1939. Calculating the Yield of a Well, Taking into Account Replenishment of the Ground Water above. Water and Water Eng., Vol.41. pp.561-563.

PUMPING TEST
SINGLE HOLE TECHNIQUE

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Abstract

Hydrogeological surveys and pumping test in igneous rock areas often have to be based on single boreholes.
A theory for evaluation of fractured aquifers based on data from a single borehole has been developed. Also a measuring device, with high metric and time resolution has been constructed for this purpose.
The paper will describe this technique and give examples from different types of hydrogeological surveys. Besides aquifer analysis this technique has also been used in controlling new as well as old water-wells both in igneous rocks and sedimentary deposits.

1 Introduction

Testpumping is a well known method in hydrogeological investigations. Analysis of aquifer conditions are normally based on data from piezometer wells (observation wells). In areas with hard rocks or deep drillings this will cause great costs to conduct a proper pumping test. To reduce these costs, single hole pumping tests have been developed.

During a pumping test the cone of depression will penetrate the aquifer and act like a "geophysical probe". All events during the development of the depression cone will be recorded as a change in the rate of drawdown.

This fact will be used when evaluating the data from a single hole pumping test. The problem is to get sufficient time resolution and accuracy in leveling ground water surface at the very beginning of the pumping test.

In Sweden the single hole pumping test is used for determining the continuous discharge from wells in fractured bedrock and for determining the efficiency of wells in sedimentary deposits. According to our experience the pumping test will give a sufficient amount of data within about 5 hours pumping of a well in sedimentary deposits and within 20 hours for a well in fractured bedrock.

2 Pumping test technique

A pumping test is a controlled disturbance of the hydrological balance in order to determine the hydraulic properties of the aquifer.

The traditional way is to measure the drawdown in a number of piezometer wells. The drawdown is a result of the development of the cone of depression, which is illustrated in figure 1. When using piezometer wells manual measurements are sufficient for the evaluation of the hydraulic properties.

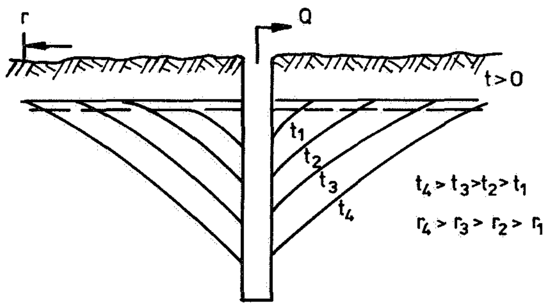


Figure 1 Development of the cone of depression

The single hole technique demands high time resolution and accurate recording of drawdown at the very beginning of the test to enable a relevant evaluation. For this purpose a special equipment, described below, has been developed.

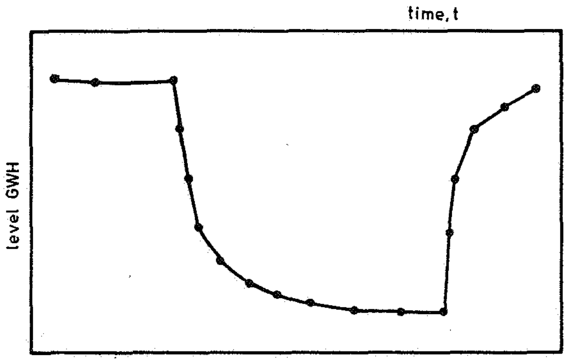


Figure 2a. Drawdown in a linear graph

Small differences of drawdown data will be obscure in a linear graph.

It is a well known fact that drawdowndata plotted in a semilogaritmic diagram compared to linear graphs will give more information. This is true especially in the beginning of the pumping test and of utmost importance when using a single hole technique, fig 2a and 2b. Together with a graph in a double logaritmic diagram it is possible to evaluate the hydraulic properties from a single hole pumping test.

The drawdown curve can be divided into four different parts shown in figure 2b. When the pumping starts most of the water comes from the borehole itself (well bore storage). Next part of the curve shows the connection between the well and the aquifer. Later on the hydraulic properties of the aquifer influence the drawdown in the well and at last the effect of the hydraulic boundaries will appear.

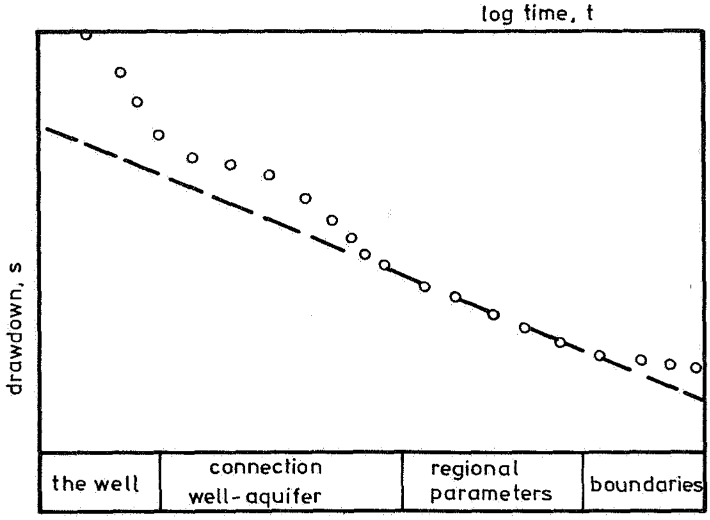
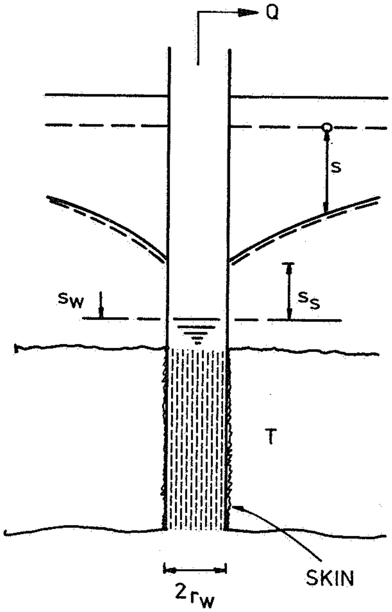


Figure 2b. Drawdown in a semilogaritmic graph

The hydraulic properties that can be determined from a single hole pumping test is the well bore storage (WBS), transmissivity (T), skinfactor (ξ), elastic storage coefficient (Sa) and leakage coefficient (P'/m'). Two examples to describe the technique are given at the end of this paper.

The figures 3a, 3b and 3c illustrate the concept of skin. Normally it is a loss in piezometric level caused by the resistance of flow into the well, positive skin, (figure 3a). It can be caused by clogging or improper well design. In fractured bedrock the contact between well and water bearing fissures can be very good and cause a negative skin, (figure 3b). A partially penetrating well will also give a skineffect, pseudoskin (figure 3c).



THE SKIN CAUSES ADDITIONAL DRAWDOWN

BY DEFINITION

$$\xi = \frac{2\pi T}{Q} \cdot s_s$$

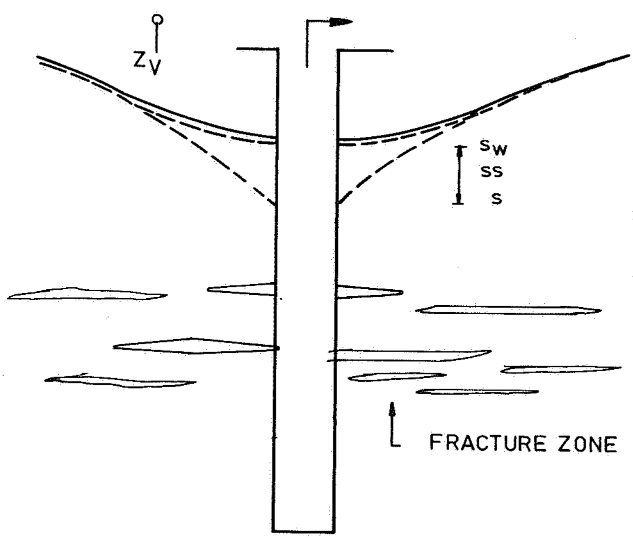
$$\Rightarrow C_\xi = \frac{2\pi T}{Q} = C_\sigma$$

$$\Rightarrow s_w = \frac{Q}{2\pi T}\,(\sigma + \xi)$$

$$\Rightarrow r_{wf} = r_w \cdot e^{-\xi}$$

Figure 3a. Concept of the skinfactor

NEGATIVE SKIN



$s_s < 0$

BY DEFINITION

$$\xi = \frac{2\pi T}{Q} \cdot s_s$$

$$s_w = \frac{Q}{2\pi T}\,(\sigma + \xi)$$

$$r_{wf} = r_w \cdot e^{-\xi} > r_w$$

Figure 3b Skinfactor in fractured bedrock

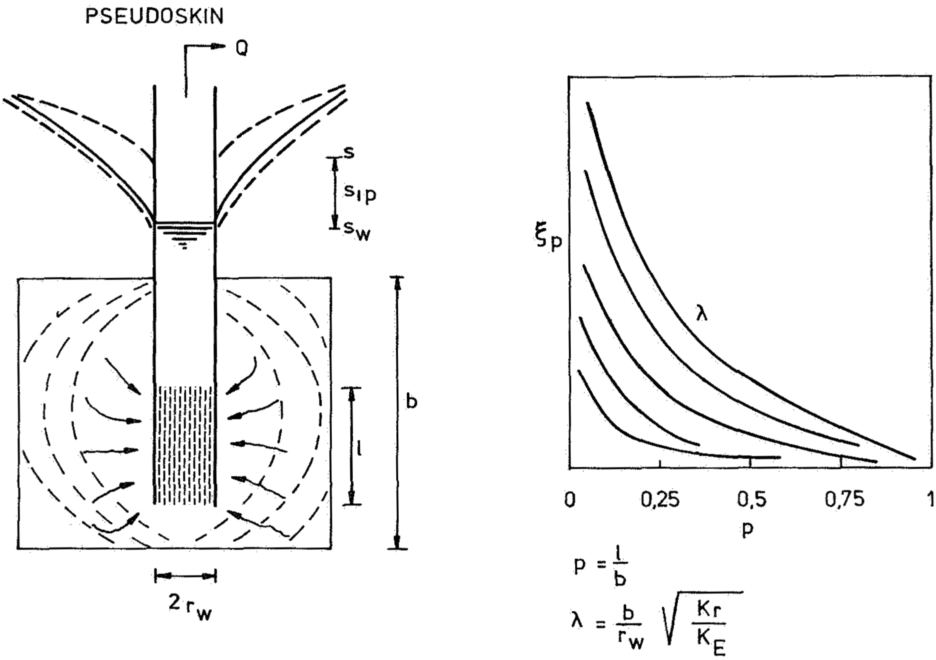


Figure 3c Skinfactor due to partial penetration

2 The field equipment

The field equipment for recording the drawdown (or recovery) is built up around a computer. In the well there is a piezoresistive transducer, often situated near the submersible pump. The transducer is connected via an amplifier to the multimeter which has a digital output. The com- computer is programmed to read the multimeter value after a special time schedule fitting the log-log and semilog scales.

During the first minute of a pumping a value is recorded every 5th seconds and after one minute the time interwals increase following a logaritmic scale. The measuring data is stored on a tape.

An equipment of this design can also be used to record several piezometers at the same time.

Figure 4 shows the field equipment and the equipment in the office, where the data automatically are drawn up using the same computer and a plotter.

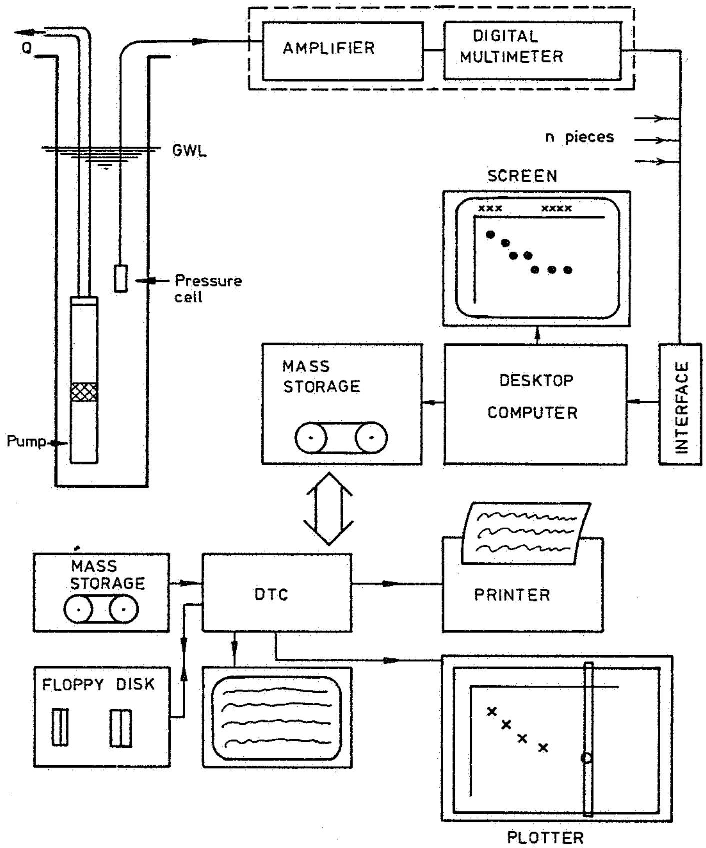


Figure 4 Computerized equipment for field and office

4 Accuracy

All the recorded values stored on the tape will pass several units, which can affect the data. Control of the accuracy of the equipment has to be performed by manual measurements.

As the technique of evaluation makes use of the difference between the starting value and the following values only this difference has to be controlled. The accuracy has been determined to about 1 % of the difference, which has been sufficient for our purposes.

5 Use of the single hole pumping test

In order to determine the continuous discharge of a well in fractured bedrock this technique can be used. It can also be used as a complementary geophysical method in the tectonic survey during site investigation for rock caverns, mines etc.

Wells in sedimentary deposits can be examined by this technique and the efficiency of the well can be determined. This means that the single hole technique can be used for control of well design, well development and to detect chemical or mechanical clogging.

Example 1

This example shows a pumping test of a well in a sedimentary deposit. The screen of the well is located in a confined aquifer. The test is a part of a general study of wells in sedimentary deposits sponsored by the Swedish Council for Building Research.

Figure 5 shows the drawdown curves in semilog and log-log graphs. In the double logaritmic diagram the well bore storage can be determined from an arbitrary chosen point on the straight line with a slope 1:1.

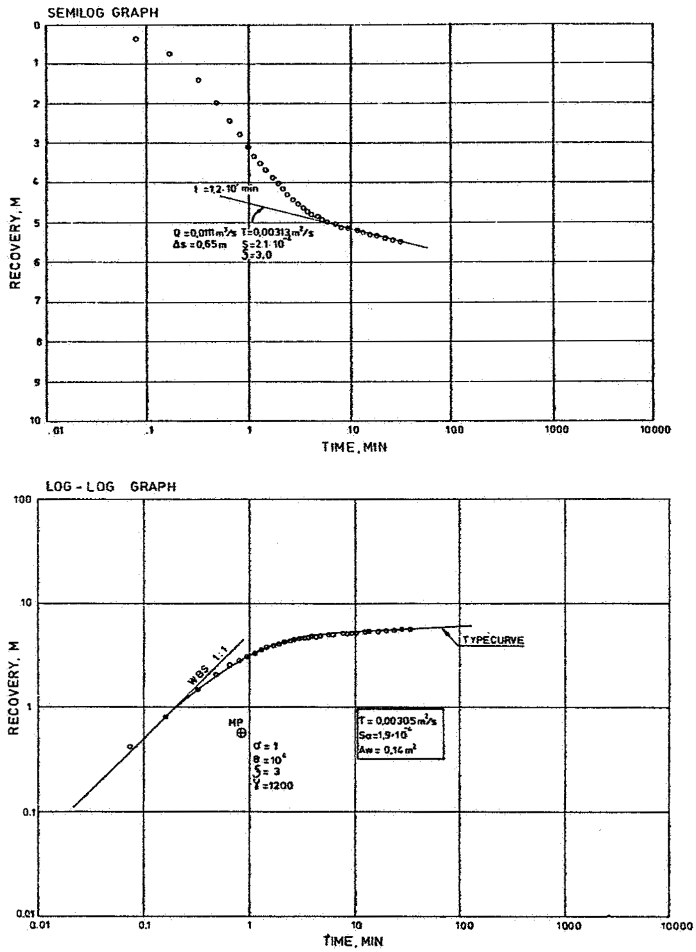
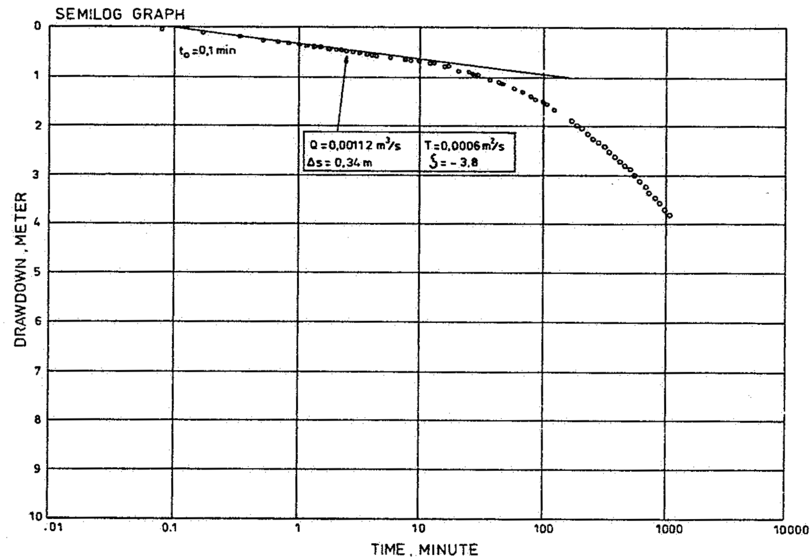


Figure 5 Recovery data for a well in a sedimentary deposit

This value can be compared with the well bore storage calculated from the geometric data of the well and the installation. At this stage you have to assume a relevant Sa-value and use the semilogaritmic diagram to evaluate the transmissivity and the skinfactor. Using the formulas for dimensionless well bore storage and the time the transmissivity and elastic storage coefficient (Sa) can be calculated from the double logaritmic diagram using typecurves.



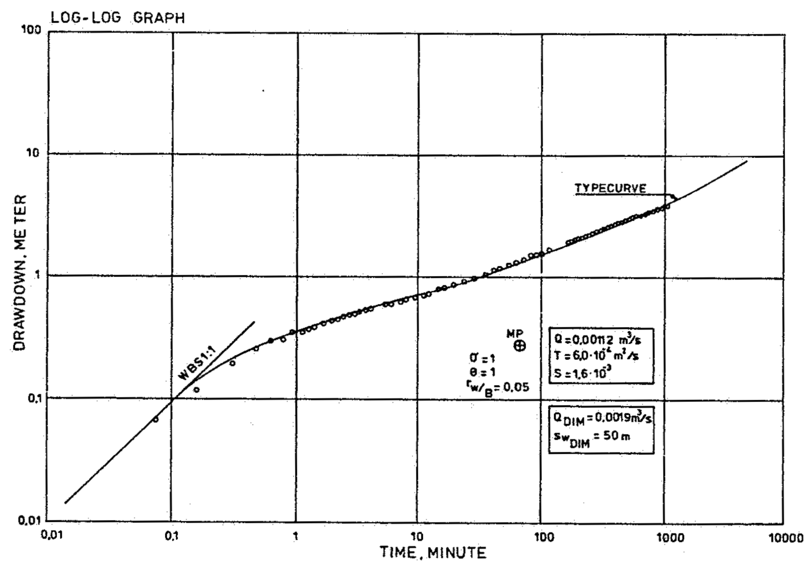


Figure 6 Drawdown data for a well in a fractured bedrock

If the assumed and calculated elastic storage coefficient and the transmissivity from semi and double logaritmic diagrams correspond the evaluation is finished.

Otherwise you have to make a new assumption of Sa and make another calculation loop. As the dimensionless drawdown, σ, for the aquifer and the skinfactor, ξ ,is determined, the efficiency of the well can be calculated as $\eta = \sigma / (\sigma + \xi)$.

Example 2

This example is taken from a pumping test of a well in fractured bedrock. The well is a part of a water distribution system for a city using the groundwater from fractured bedrock.

Figure 6 shows the drawdown from this well in a fractured bedrock. The evaluation technique is the same as for example 1.

In this case the drawdown reveals an aquifer acting as a single channel.

At the end of the pumping period the curve seems to turn into steady state conditions due to leakage effects, i.e. from the secondary fissures.

Assuming that waterbearing fissures are below the final level of drawdown, the continuous discharge of the well can be calculated.

EVALUATION OF AQUIFER PARAMETERS IN LARGE DIAMETER WELLS - A CASE STUDY

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Abstract

In most of the developing countries of Asia and Africa, including India, the exploitation of groundwater is mainly through large diameter dug wells. About two third of the surface area in India is covered by hard rocks and nearly 50 percent of replenishable groundwater resources occurs in these rocks. There is a lack of mathematical solutions to evaluate the hydraulics of shallow large diameter wells. Papadopulos and Cooper have suggested a method of analysing pumping test data from large-diameter wells tapping confined aquifers. But it is applicable only to a fully penetrating aquifer. Boulton and Streltsova (1976) derived equations for the drawdown in partially penetrating wells under unconfined (water-table) conditions. In India, the Papadopulos and Cooper method has been used for the evaluation of aquifer parameters (T and S). In the present study, the method suggested by Boulton and Streltsova has been used to determine the Transmissivity values for large diameter wells tapping the Miliolite and the Gaj Formations of Quaternary and Tertiary ages. The results obtained are quite comparable and hence,this method can be tried in other areas.

1 Introduction

Large diameter dug wells which are used to tap groundwater is a very common feature in India. This paper embodies the results of 14 pumping test analyses in wells of finite diameters having storage capacity, in a carbonate terrain along the southwest coast of Saurashtra, India. The wells tap two formations viz. the Miliolite Formation of Pleistocene age and the Gaj Formation of Miocene age. The limestones belonging to the Miliolite Formation are karstified. Solution openings are numerous and both intergranular and intragranular porosity is present. The limestones belonging to the Gaj Formation are ocherous and hard and compact with lot of fossils. Fractures and joints are developed in this rock. The wells tap the upper portions of the Miliolite Formation and the Gaj Formation which are generally under water table conditions.

The applicability of standard methods of aquifer test analysis in hard rock aquifers has long been a subject of debate. Crystalline rocks, carbonate rocks and other consolidated rocks as a medium of groundwater flow possess anisotropy and heterogeneity. The hard rocks are characterised by secondary porosity due to the presence of joints, fissures, foliation planes and solution cavities.

In karstic rock terrains, the circulation of water takes place both in individual isolated fractures and voids where a hydraulically single flow is formed. Hence, generally under natural conditions, a single flow system prevails.

Some quantitative evaluation and interpretation of flow in fissured rocks started only recently. In developing the analytical models of flow through fissures, it is considered that the flow properties are due to the presence of fissures and the storage of water is associated with intergranular porosity. The aquifer is considered to be made up of blocks and fissures in a regular pattern and in having a double porosity i.e., porosity of blocks and that of fissures. Based on this, models have been developed by various workers including Papadopulos (1967), Rofail (1967), Snow (1966), Babushkin et al. (1975), and Boulton and Streltsova (1976a,b).

The finding out of the aquifer parameters in large diameter dug wells is problematic, because there is a lack of mathematical solutions to evaluate the hydraulics of shallow large diameter wells. Many workers have attempted to estimate the aquifer parameters by using various methods of pumping test analysis. Sammel (1974) has reviewed the different methods of aquifer tests in large diameter wells and came to the conclusion that the method of Papadopulos (1967) and Papadopulos and Cooper (1967) are the best available approaches. Papadopulos and Cooper's method, although, takes into account the effect of well storage, it is applicable only to a fully penetrating well in a confined aquifer.

2 Analysis of pumping test data

The pumping test data from 14 wells of finite diameters have been made use of in the study. The wells are either dug ones or dug-cum bore wells. Most of the wells are rectangular in shape and the average cross-section is 3.20 x 2.10 m. From the bottom of the well bores are drilled varying from 10 to 15 cm in diameter. The wells are partially penetrating.

Boulton and Streltsova (1976) have derived equations for the drawdown in partially penetrating wells under unconfined (water table) conditions. The drawdown at any point in the aquifer is a function of the depth of that point below the watertable.

The assumptions in this method are:

a) The aquifer is compressible and in general anisotropic the horizontal and vertical permeabilities being constant.
b) The aquifer is underlain by a horizontal impermeable bed; which may be at any depth below the bottom of the pumping well.
c) The aquifer is pumped at a constant rate from the instant t = 0.

The equation in the abbreviated form can be written as

$$S = Q/4\pi T \; W(U_A, \beta, S, \rho, 1', y') \qquad (1)$$

where $W(U_A, \beta, S, \rho, 1', y')$ = well function for water table aquifer with partially penetrating wells and production well storage capacity (dimensionless).

Values of $W(U_A, \beta, S, \rho, 1', y')$ are given by Boulton and Streltsova (1976) in terms of a selected range of $1/U_A$, β, S, ρ, 1', and y'. From these a family of type curves can be drawn.

The data curve plotted on a double log sheet with drawdown versus time was matched with one of the type curves and a suitable match point was selected. The values of $1/U_A$, $(W(U_A, \beta, S, \rho, 1', y')$, and ρ were read from the type curve and t and s from the data curve. These values were then substituted in equation (1) and the transmissivity values were calculated (Tables 1 and 2).

Table 1 Computed values of T for the Miliolite Formation

Serial No.	T (m^2/day)
1	554
2	137
3	728
4	682
5	909
6	971
7	51
8	109

Table 2 Computed values of T for the Gaj Formation

Serial No.	T (m^2/day)
1	102
2	111
3	62
4	32
5	138
6	175

3 Conclusion

Tables 1 and 2 indicate that the transmissivity determined by this method for the Miliolite Formation varies from 51 to 971 m^2/day and for the Gaj Formation it varies from 32 to 175 m^2/day. These values are comparable with the results obtained by various methods of aquifer test analysis in similar rocks of other areas. Hence, this method can be employed for the determination of the aquifer characteristics in large diameter dug wells. The wide range in the T values for the Miliolite Formation is due to the aquifer anisotropy and the high T values can be attributed to the karstic nature of the limestones. In the case of the limestones belonging to the Gaj Formation the lower values of T is due to the lower permeability of the rocks because of the presence of clay.

Acknowledgements

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References

Babushkin, V.D., Borevsky, B.V., and Kovalevsky, V.S. with contributions by Bröcker, 1975. Regime of subterranean water flows in karst regions. In Burger, A, and Dubertret, L. (ed.) Hydrogeology of karstic terrains. Published by International Association of Hydrogeologists.

Boulton, N.S., and Streltsova, T.D., Flow to a well in an unconfined fractured aquifer. Intern Symp. Hydrol, 1976. Problems in karst Regions, Bowling Green, Kentucky.

Boulton, N.S., and Streltsova, T.D., 1976. The drawdown near an abstraction well of large diameter under nonsteady conditions in an unconfined aquifer. Jour. Hydrology, 30, 29-46.

Papadopulos, I.S., 1967. Non-steady flow to a well in an infinite anisotropic aquifer. Proc. Dubrovnik Symposium on Hydrology of Fractured Rocks, Vol.1, 21-31.

Papadopulos, I.S., 1967. Drawdown distribution around a large diameter well. Proc. Nat. Symposium on Groundwater Hydrology, AWRA, California, 157 -168.

Papadopulos, I.S., and Cooper, H.H. Jr., 1967. Drawdown in a well of large diameter. Water Resources Res. Vol.3, 241-244.

Rofail, N., 1967. Analysis of pumping test in fractured rocks. Proc. Dubrovnik Symposium on Hydrology of Fractured Rocks, Vol. I, 81-88.

Sammel, E.A., 1974. Aquifer tests in large diameter wells in India. Ground Water, 12, 265-273.

Snow, D.T., 1969. Anisotropic permeability of fractured media. Water Resources Res., 5, 1273-1289.

APPLICATION OF FLOW NETS TO THE
DECCAN BASALTIC AQUIFERS OF
MAHARASHTRA, INDIA, TO EVALUATE
GEOHYDROLOGICAL PARAMETERS

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Abstract

The basaltic terrain of Rahuri area, Maharashtra, India, is an irrigated tract. The variation in transmissivity values as inferred from well tests indicate the anisotropic nature of the aquifers. A water-table contour map of the area is constructed which assume to represent the hydrologic system. Idealized isotropic conditions are assumed and the ground water flowlines are initiated at points where the transmissivity is known. The stream tubes are constructed by extending the flowlines on either sides. By assuming the same discharge in a stream tube and by using the Darcy's law, the spatial variation in transmissivity is calculated. The anisotropy of aquifers is indicated by spatial variation of transmissivity of the range 23 to 123 m^2/day.

1 Introduction

Flow net comprises of sets of equipotential lines and flow lines. The net is two dimensional graphical representation of cross section of a three dimensional groundwater flow system. The analytical solutions of flow nets can be used to study hydrologic behaviours of groundwater flow system, such as boundary conditions, spatial variation in transmissivity etc. In the present paper an attempt has been made to study the feasibility of application of flow nets to basaltic terrain of

Rahuri area, to study the spatial variation in transmissivity.

2 Geohydrology of the area

Rahuri Agricultural University area is situated in Ahmednagar district, Maharashtra. This area is an irrigated tract with Mula right bank canal passing through the area. The eastern half of the canal is unlined. The geohydrology of this area has been studied by Deolankar (1978).

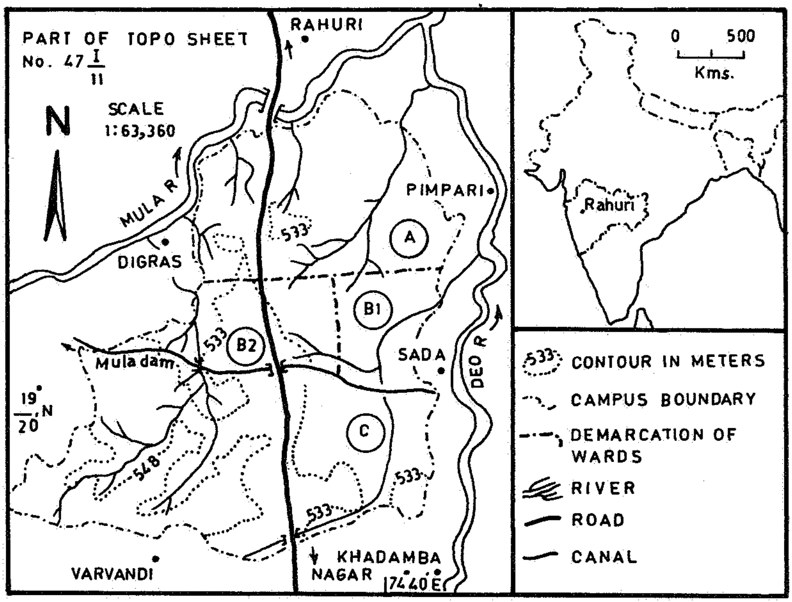

Figure 1 Location map of the Rahuri area.

The two main basaltic flows exposed over the area are greyish black basalt and pinkish amygdaloidal basalt. The junction between these two flows is marked by a 1 m thick red layer exposed at approx. 530 m above M.S.L. The well inventory shows that grey basalt is jointed and fractured at places while the amygdaloidal basalt, which is more susceptible to weathering, is weathered at places. Thus, in general, the jointed-fractured grey basalt forms the aquifer above 530 m M.S.L. and weathered amygdoloidal basalt forms the aquifer below 530 m. The groundwater occurs under watertable conditions and is exploited by dug wells.

The well tests analysis indicated the potential of aquifers. The pumping test data was analyzed by using Cooper-Jacob (1946), Papadopulos-Cooper (1967) and Walton (1970) methods to evaluate the transmissivity and specific yield. The transmissivity of aquifer varies from 25 to 130 m^2/day while specific yield from 0.8 to 10%. These point determinations of transmissivity indicate the anisotropic nature of the aquifers.

Table 1 Transmissivity values from well tests and Flow Nets

Well No.	Ave. Transmissivity from well test m^2/day	Transmissivity from Flow Net m^2/day
B1W3	115	115
B2W2	36	30
B2W3	37	37
B2W7	26	30
CW7	63	63
CW9	67	63
CW31	130	123
CW35	112	100

The waterlevel data analysis, in previous study, indicate that the unlined part of the canal recharges the groundwater, Deolankar (1978). Thus, besides rainfall, canal seepage forms main source of groundwater recharge. This canal recharge has increased the groundwater storage and raised the watertable beyond critical limit in south eastern part of the area giving rise to salination of soils, Deolankar (1982). Such facts indicate that the unlined canal forms a recharge boundary in the southeast part of the area. This southeast part of the area is chosen for Flow Net study.

3 Application of Flow Net

The graphical technique of Flow Nets has been described by Harr (1962), Davis and DeWiest (1966), Freeze and Cherry (1979), Bouwer (1978) etc. Skibitzke and da Costa (1962) have applied the Flow Net technique to basaltic terrain of Snake River basin, Idaho, U.S.A. to study the groundwater Flow system. Bouwer (1978) has described the use of Flow Nets to evaluate the transmissivity if the point determinations of transmissivity are available.

A watertable contour map of 2 m interval, conforming to the unlined canal as recharging boundary, for the southeast part of the area is prepared (Figure 2). This map is assumed to represent the hydrologic system of this part. For simplicity, the idealized isotropic conditions are assumed to construct the orthogonal network of groundwater flowlines. The flowlines are initiated at points where the transmissivity values from well tests are known. The rate of flow of groundwater is inversely proportional to distance between two watertable contours. Therefore two flowlines are drawn so that the length of flowlines is equal to distance between watertable contours. Thus the squares or unit areas created will carry definite amount of water. This quantity can be calculated by using Darcy's law for steady horizontal flow (Bouwer 1978).

$$Q = T \times H \times W / L \tag{1}$$

where

Q = flow per unit area in m^3 per day.

T = Transmissivity in unit area m^2/day as inferred from well test.

H = drop in level across the unit area in m.

W = ave.width of unit area in m.

L = ave.length of unit area in m.

In this equation for initial unit areas the T and H are constant and ratio of W/L will be one. This ratio of W/L can be termed as Index ratios. Now the stream tubes can be constructed by extending the flow

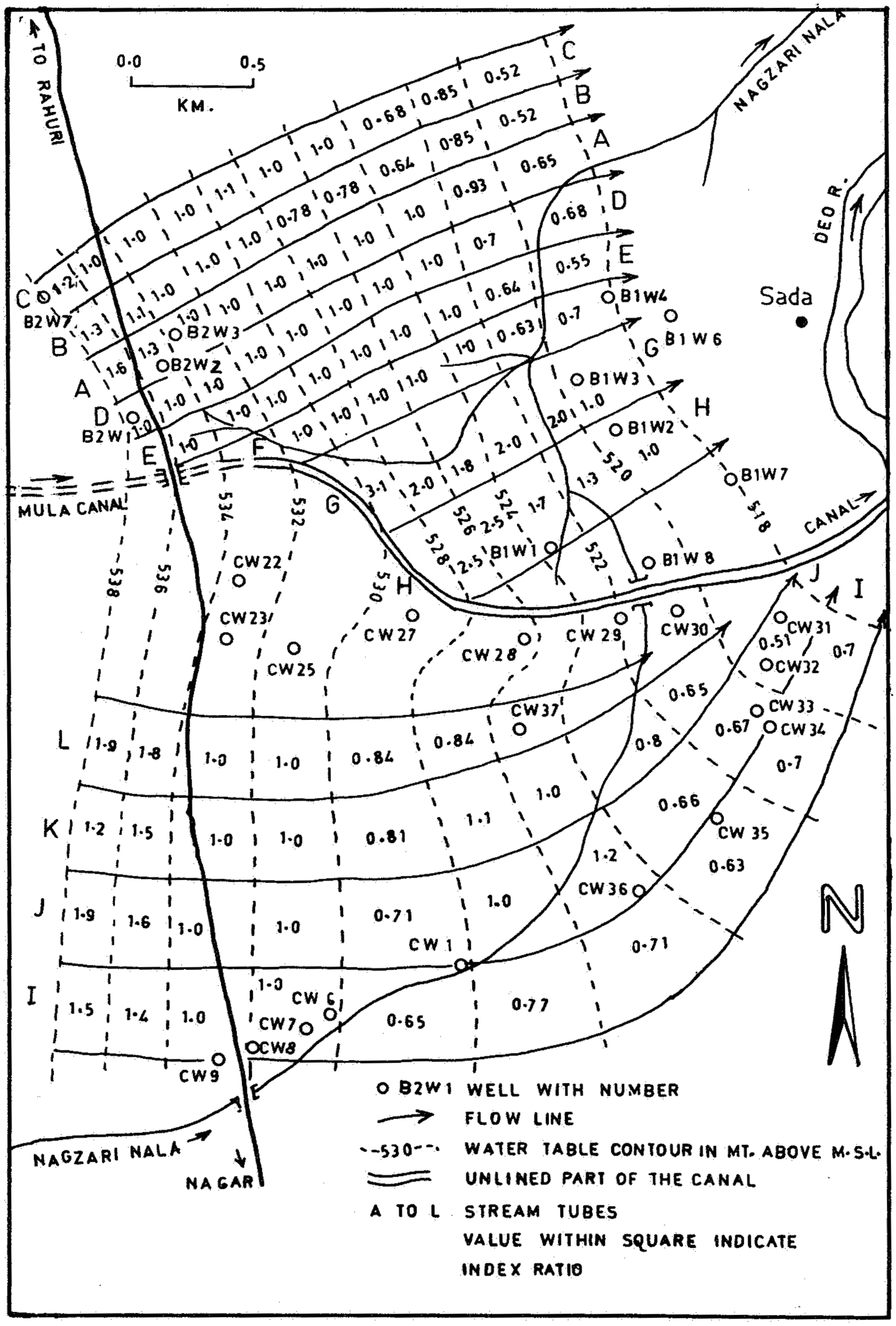

Figure 2 Idealized Flow Net map of Southeast Part of Rahuri Area

lines on either sides obeying the thumb rule that the flow line intersects at right angles to the watertable contourThus the successive unit areas can be created.

For simplicity, under isotropic conditions, the same quantity of water is assumed to flow in successive unit areas of the stream tube. Since H remain constant for each unit area, the departure of index ratios from unity for successive unit areas will indicate variation in transmissivity.

The stream tubes are initiated at well Nos.B2W3, B1W3, CW7 and the stream tubes A, G, I are completed (Figure 2). The other stream tubes are constructed by considering the discharges given in Table 2. The variable discharges are used with assumption of variation in recharge at places. The index ratios are calculated for different unit areas and are shown on the map.

Table 2 Flow net data for stream tubes

Stream tubes	Discharge per unit area m^3 per day	Range of variation in Trans.m^2/day
A,B,C,D,E,F.	74	23 to 71
G,H.	230	37 to 115
I,J,K,L.	126	33 to 123

4 Discussion

In order to analyze the subsurface flow system by Flow Nets, it is necessary to know internal state of the aquifer, external boundary conditions, input-output data etc. However, the present Flow Net is evolved with the idea to evaluate spatial variation in transmissivity. Hence only canal as recharge boundary is considered and few representative stream tubes are drawn.

The apparent qualitative analysis of the Flow Net shows that the groundwater moves towards the Deo river flowing along eastern boundary. The artificial recharge boundary of canal is indicated by the bending of watertable contours along the trend of the canal. The apparent convergence of stream tubes K and L can be explained by the water sump noticed in Nagzari nala, where it crosses the overhead canal. The steepening of hydraulic gradient in stream tubes G and H near the canal could be due to necessity to move additional water seeped from the canal.

By considering the appropriate discharge, index ratios shown on the map and using the equation one, the transmissivity values are calculated for different stream tubes. Table 2 shows the range of spatial variation of transmissivity for different stream tubes. If more than one value of transmissivity from well test is available in the stream tubes, it is used as check over the Flow Net. Table 1 shows that the Flow Net values of transmissivity matches well with that of well tests. Now if we assume that all the stream tubes are of unit depth, the variation in transmissivity at places indicate the variation in Hydraulic conductivity. The spatial variation in transmissivity thus calculated can be used for giving sites to new wells etc.

5 Conclusions

The study shows that certain idealized assumption like isotropy are necessary to evolve the Flow Nets. The departure from assumed isotropic conditions indicate the anisotropic nature of the aquifers. In the present case the anisotropy is indicated by the spatial variation in transmissivity. Inspite of idealizations, the Flow Net can form a useful tool to evaluate geohydrologic parameters if the geohydrologic data at some places is available. The validity of Flow Net increase manyfold if accurate hydrologic model is predicted. For that purpose certain data like groundwater inputs-outputs, hydrologic behaviours, accurate well inventory data is necessary. However, this study indicate that the Flow Net technique can be applied to small area of basaltic terrain,if data is available.

Acknowledgement

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References

Bouwer, H. 1978. Groundwater Hydrology. McGraw-Hill Book Co. N.Y.480

Cooper, H.H. and C.E.Jacob. 1946. A generalised graphical method for evaluating formation constant and summarising wellfield history. Trans. Am. Geophys. Un., 27:526-534.

Davis, S.N. and R.J.M.DeWiest. 1966. Hydrogeology. John Wiley and Sons. N.Y. 463 pp.

Deolankar, S.B. 1978. Hydrogeology of the Mahatma Phule Krishi Vidyapeeth area, Rahuri, Ahmednagar Dist., Maharashtra. Recent Researches in Geology, Vol. 4, Hindustan Publ. Co., 287-312.

Deolankar, S.B. 1982. Salination of soils - A environmental Hazard for agricultural landuse in Irrigated tracts. Proc. of Int. Symp. LANDPLAN-1, April 1-3, 1982. Bangkok, Thailand, A 4.1-4.6 pp.

Freeze, R.A. and J.A.Cherry. 1979. Groundwater. Prentice Hall. Inc. N.J. 604 PP.

Harr, M.E. 1962. Groundwater and Seepage. McGraw Hill Book Co. N.Y.316P.

Papadopulos, I.S. and H.H.Cooper. 1967. Drawdown in a well of large diameter. Water Reso. Res., 3:241-244.

Skibitzke, H.E. and J.A.da Costa. 1962. The groundwater flow system in the Snake River Plain, Idaho - An idealized analysis. U.S.G.S.Water Supply Paper No. 1536-D : 47-62.

Walton, W.C. 1970. Groundwater Resources Evaluation. John, Wiley and Sons. N.Y. 664 pp.

DETERMINATION OF THE ELASTIC STORATIVITY FROM BAROMETRIC EFFICIENCY

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Abstract

In the dune area, along the Dutch coast, atmospheric pressure and hydraulic head changes have been recorded in seventeen observation wells, during a three week period. The data have been analyzed by time series analysis, to determine the barometric efficiency. From the barometric efficiency the elastic storativity of an aquifer may be derived.
The elastic storativity values determined from the barometric efficiency using Jacob's expression, are smaller than values obtained by a pumping test on this site. According to Jacob, the difference may be caused by leakage from or into contiguous beds. Another explanation follows from the theory on the elasticity of aquifers given by Verruijt. Verruijt proves that the compressibility of the grain skeleton is different for a vertical deformation (atmospheric pressure) compared to radial flow towards a well (pumping test).
A modified relation between storativity and barometric efficiency is given where the theory of Verruijt is taken into account.

1 Introduction

In most regional groundwater studies the storage coefficient or storativity of elastic aquifers does not play a significant role. However, in well-test analysis the storage coefficient takes a key position.

Transmissivity may be determined by simple tests on a pumping well, without additional observation wells. If the storage coefficient is known, these so-called well-tests may also provide useful information, e.g. on the leakage factor in semi-confined aquifers and the well-losses (e.g. clogged pumping wells). Therefore an independent method to determine the storage coefficient could be valuable.

The elastic storativity of an aquifer can be derived from the so-called barometric efficiency. Up to the present this method has been used rarely in the Netherlands. In february 1981 the Municipal Waterworks of Amsterdam invited the National Institute for Water Supply to test this method on a site near the pumping station at Leiduin. A short description of the site and discussion of the test results are given. At first various definitions are given.

2 Definitions

Jacob (1940) defines the storage coefficient as the volume of water released from storage within a column of aquifer underlying a unit-surface area, during a decline in piezometric head of unity.

If ρ is the density of the water (kg/m^3), n and D (m) the porosity and thickness of the aquifer and g (m/s^2) the acceleration due to gravity, then the storage coefficient S (dimensionless) may be written, according to Jacob, as:

$$S = \rho g D\ (\alpha + n\beta) \qquad (1)$$

where the compressibility of the grain skeleton is denoted by α and the compressibility of water by β (both in m^2/N). The storage coefficient is a parameter related to the total height of the aquifer. In three-dimensional flow it is more convenient to use the specific storage coefficient S_s(m^{-1}) that is defined as (Jacob, 1950):

$$S_s = \rho g\ (\alpha + n\beta) \qquad (2)$$

The barometric efficiency BE may be defined as the ratio of the change in piezometric head in a well ($\Delta\Phi$) to the corresponding change in atmospheric pressure (Δp):

$$BE = -\rho g \ \Delta\Phi/\Delta p \qquad (3)$$

In this equation barometric pressure must be expressed in Pascal (1 Pa = 1 N/m^2). The barometric efficiency of an aquifer can be determined when simultaneously water level fluctuations and barometric fluctuations are recorded.
The barometric efficiency may equivalently be expressed as a function of the water compressibility β, the compressibility of the grain skeleton α and the porosity n:

$$BE = n\beta/(\alpha+n\beta) \qquad (4)$$

Combination of (2) and (4) leads to:

$$S_S = n\beta\rho g/BE \qquad (5)$$

From this equation the specific storativity may be calculated. The parameters ρ, g and β are all known, while the porosity may be estimated at 0.35 for most Dutch aquifers. Barometric efficiency is determined by recording water level and barometric pressure.

3 Field description and data analysis

In the dunes along the coast, artificial recharge is applied by the Municipal Waterworks of Amsterdam. Near the pumping station Leiduin an injection well is situated that at the present time is out of use. Around this well a number of observation wells are located with screens at various depths (Figure 1). The meaning of the numbers at the screens is as follows. The first number denotes the horizontal distance to the injection well; the second number denotes depth below datum level. Seventeen filter screens have been selected and pressure transducers were installed in it. A great number of screens has been chosen to check

whether a relation exists between specific storativity and depth.
The type of pressure transducer used here, measures pressure change with regard to the barometric pressure, so the recorded data indicate pressure changes due to water level changes only. The fluctuations of water level in observation well 40-44 and the corresponding fluctuations of barometric pressure are shown (Figure 2). In the vertical scale of upper and lower graph the term ρg has been taken into account, so the barometric efficiency may be determined graphically as the ratio of the slopes of the graphs at the left and right side. In this way BE is estimated roughly at 0.25.
A powerful technique to investigate the relation between two series of time dependent data is time series analysis. One of the series (the barometric pressure) is considered as input for a system (the aquifer), while the second series (water level in observation well) forms the response.
Time series analysis is based on the well-known principle that each signal, input as well as response, can be expanded into a Fourier series. Each Fourier component is characterized by a frequency f, a phase shift $\varepsilon(f)$ and an amplitude $A(f)$. The relation between response and input is shown in a so-called bode-diagram, where the amplitude ratio of response and input signal A_r/A_i is plotted versus f.
From the theory of time series analysis (Papoulis, 1965) it is known that this diagram also represents the fourier transform of the response to a unit impulse inputfunction $\delta(t)$. Equation (3) suggest that in our case the response (piezometric head) to a unit impulse input $p = \delta(t)$ will be

$$\Phi(t) = (- BE/\rho g)\ \delta(t) \qquad (6)$$

The fourier transform of this function is a constant:

$$\hat{\Phi}(f) = - BE/\rho g = A_r/A_i \qquad (7)$$

The barometric efficiency, therefore, may be calculated from the bode diagram.

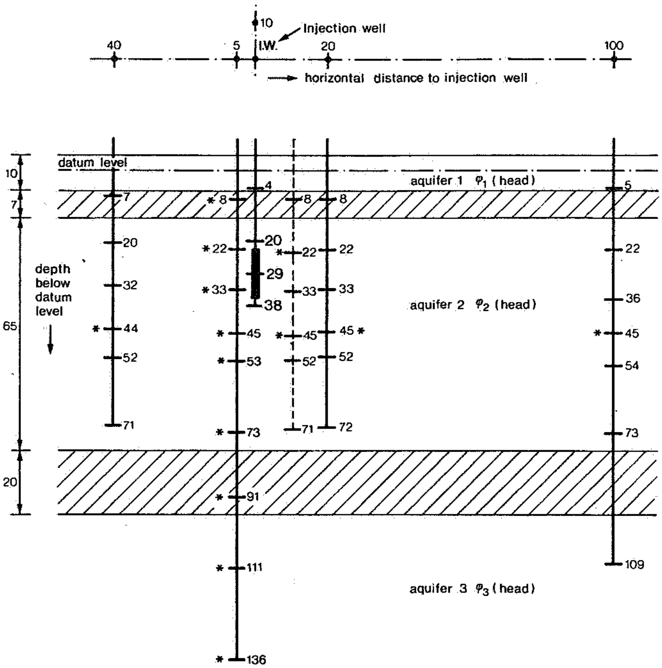

Figure 1 Geohydrological scheme and location of observation wells with screens at various depths; selected screens are marked with*

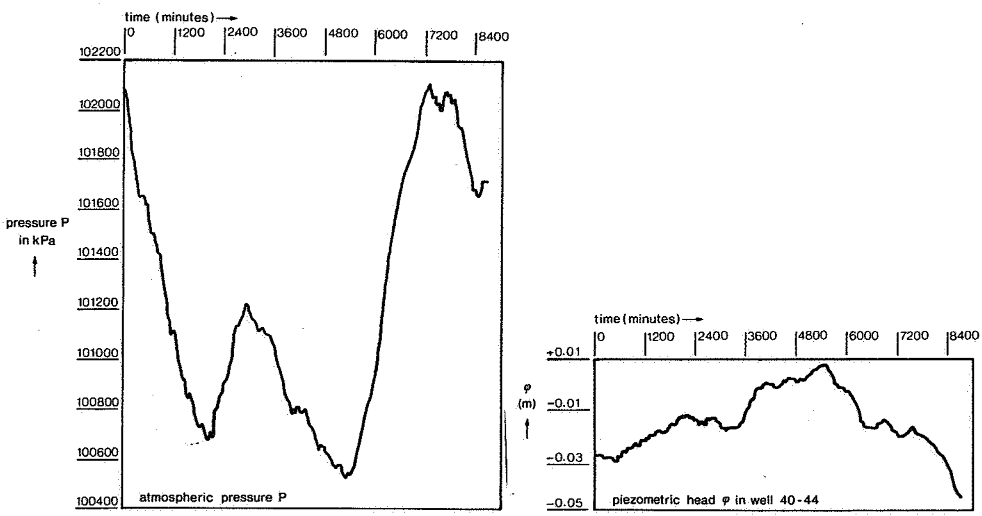

Figure 2 Fluctuations in atmospheric pressure and piezometric head (well 40 - 44)

4 Results

With a computer-program TRARID bode-diagrams for all observation wells were calculated. For observation well 40-44 the bode-diagram is given (Figure 3). On the vertical axis A_r/A_i is plotted on a logaritmic scale, while horizontally the relative frequency $\bar{f}$ of the Fourier component is given. The relative frequency is the frequency compared to a component with a period of 20 minutes. A relative frequency of e.g. 0.2 corresponds to a period of 20/0.2 = 100 minutes.

Before calculation of the bode-diagram the atmospheric pressure has been divided by ρg to convert pressure from pascal (N/m^2) to meters of water. Consequently the value of A_r/A_i corresponds directly to the barometric efficiency, according to equation 3.

It appears (Figure 3) that the ratio of amplitudes is almost constant as a function of the Fourier component. The averaged value for observation well 40-44 is 0.227. For the specific storativity is found $S_s = 0.76 \times 10^{-5}$ (m^{-1}). Results for all observation wells are given in Table 1.

The following numerical values are used:

- porosity $n = 0.35$
- density of water $\rho = 1000$ kg/m³
- compressibility of water $\beta = 0.5\ 10^{-9}$ m²/N
- acceleration due to gravity $g = 9.8$ m/s²

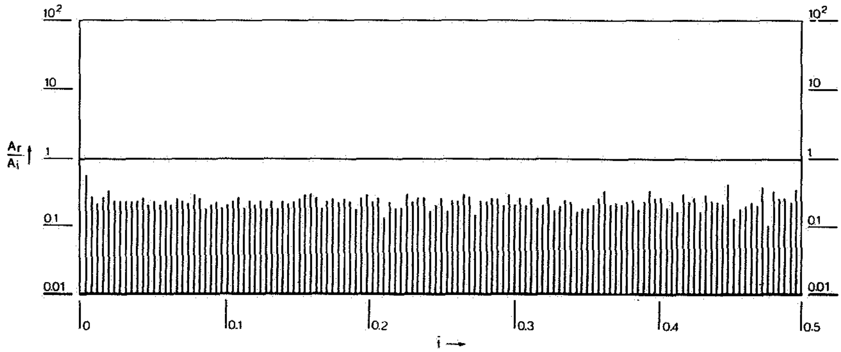

Figure 3 Bode diagram for observation screen 40 - 44

Table 1. Barometric efficiency and specific storativity (IW = injection well)

Observation well	BE	S_s m^{-1}	Aquifer
5 - 8	0.167	1.03×10^{-5}	(clay)
IW - 38	0.229	0.75×10^{-5}	
IW - 29	0.222	0.77×10^{-5}	
IW - 20	0.241	0.71×10^{-5}	
5 - 22	0.242	0.71×10^{-5}	
5 - 33	0.229	0.75×10^{-5}	
5 - 45	0.222	0.77×10^{-5}	
5 - 53	0.223	0.77×10^{-5}	
5 - 73	0.196	0.88×10^{-5}	2
10 - 22	0.221	0.78×10^{-5}	
10 - 45	0.220	0.78×10^{-5}	
20 - 45	0.216	0.80×10^{-5}	
40 - 44	0.227	0.76×10^{-5}	
100 - 45	0.224	0.77×10^{-5}	
5 - 91	(1.45)	--	(clay)
5 - 111	0.092	1.87×10^{-5}	
5 - 136	0.088	1.94×10^{-5}	3

5 Discussion

It is evident (Tabel 1) that all observation wells in aquifer 2 yield essentially the same value for S_s. It may be concluded that within an aquifer no relation exists between storativity and depth. This implies that one observation well may be sufficient. A second conclusion is that the value for S_s in aquifer 3 is greater than in aquifer 2. Van der Knaap (1959) gives a relation between storativity and depth based on data from oil-reservoirs. According to this relation, however, storativity decreases with depth. The relation assumes the same aquifer material on various depths. Probably this assumption is not correct for the case described in this paper.

In 1976 a pumping test was carried out at the injection well. The specific storativity in aquifer 2 was found to be about 1.5×10^{-5}. The values of S_s determined from the barometric efficiency are smaller than the values, obtained from the pumping test. In his original paper Jacob compared storage coefficient from pumping tests with the values computed from the tidal efficiency TE. Tidal efficiency is related to barometric efficiency by the expression BE = 1 - TE. Jacob also found that the storativity values computed from the tidal efficiency were smaller than those obtained by pumping tests. From equation (5) is it clear that the storativity is proportional to the value assumed for porosity. However, the difference with the pumping test result can only be partially explained in this way, since a porosity higher than 0.40 seems very unlikely.

According to Jacob the difference is due to leakage from or into contiguous beds, while tidal and barometric efficiency is based on the assumption that no leakage from contiguous bed takes place.

Since aquifer 1 is a phreatic aquifer, barometric pressure changes do not affect the water level Φ_1. On the other hand, in aquifer 2 the piezometric head Φ_2 varies. Due to differences in head increasing and decreasing leakage may occur indeed. Another explanation is given by Verruijt (1969).

Verruijt has shown that for radial flow towards a pumping well horizontal displacement of the sand grains is no longer negligible. For one-dimensional vertical displacement, as occurs when only barometric pressure is varying, the coefficient α is different from the radial flow case. When α' denotes compressibility for the radial flow case, the ratio α'/α appears to be equal to $2(1-\nu)$, where ν is the poisson ratio (Terzaghi, 1943). Consequently α should be substituted by α' in equations (1) and (2), in case of a pumping test. Equation (4) remains unchanged, since atmosphere pressure changes cause vertical deformation only. This leads to a modification of equation (5):

$$S_s = \rho \frac{gn\beta}{BE} \left[2(1-\nu) - BE\ (1-2\nu) \right] \qquad (8)$$

If $\nu = 0.25$ is assumed, the value of the specific storativity from barometric efficiency changes to 1.05×10^{-5}.
This figure is still below the average value determined by the pumping test. However, there is a great variation in the pumping test values for the different observation wells, i.e. from $1 * 10^{-5}$ till $3 * 10^{-5}$, while the results derived from the barometric efficiency are more consistent. It may therefore be concluded that the described method is very useful to determine elastic storativity. The method can be applied in observation wells as well as in pumping wells.

References

Jacob, C.E., 1940. On the flow of water in an elastic artesian aquifer. Transactions Am. Geophys. Union, 2: 574 - 586.

Jacob, C.E., 1950. Chapter 5 in: H. Rouse (editor), Engineering Hydraulics. J. Wiley & Sons, New York, pp 321 - 386.

Terzaghi, K., 1943. Theoretical Soil Mechanics. Chapman and Hall, London.

Papoulis, A., 1965. Probability, Random Variables and Stochastic Processes. McGraw-Hill Kogakuska, 583 pp.

Van der Knaap, W., 1959. Nonlineair behaviour of elastic porous media. Transactions AIME, 210: 179 - 187.

Verruijt, A., 1969. Elastic storage of aquifers. In: R.J.M. De Wiest (ed), Flow Through Porous Media, Academic, New York, pp 331 - 376.

GROUNDWATER INVESTIGATIONS IN THE SEMI-ARID PARAIBA

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Abstract

Results of the aquifer tests conducted in the water supply district of CAGEPA, Catolé do Rocha of semi-arid Paraiba were analysed by way of time-drawdown and distance-drawdown methods of Jacob. Impervious bedrock boundaries and recharge sources have been located using the method of well images. Transmissivity and storage co-efficients of the aquifers were determined to find the feasibility of utilising these aquifers for water supply schemes in the micro-region. The drawbacks of existing procedures have been discussed. Analysis of the test results indicate that aquifers located in the alluvial deposits of rivulet Agon, north of township are promissing, with transmissivity values in the range of 25 to 43 m^2/h and specific yields of the order of 0.001 to 0.003.

1 Introduction

The micro-region of Catolé do Rocha lies in Paraiba, Brazil near the 6^o - 21' lat' and the 37^o - 45' long', with a medium altitude of 250 m above sea level. The effectively irrigable areas total 245,260 ha out of a 300,000 ha of the micro-region. Of this area, only 17 percent forms irrigable land with little or no limitations, while a major part is unfit for use in agriculture. Aquifers tests were conducted by the CAGEPA in the micro-region. Earlier studies made by SUDENE, organisation of the north-east Brazil were inconclusive, without quantificating the yields of aquifers. The low storage capacity of most of the aquifers in the region is because of the geological conditions in the north-east and the low precipitation rates, low as 600 mm in drought years. High evaporation rates of the order of 1600 mm with a hydraulic deficiency of 850 mm/yr make the groundwater levels go down, causing salt accumulation, due to medium annual temperatures of 26,5^{o} C.

2. Hydrogeology of the Region

The hydrogeology of the region is characterised principally by two types of formations. The crystalline terrains practically impervious occupy a major part of the region, where the possibility of water reserves is restricted to fractured zones with less storage capacity. The others are the sedimentary deposits in valley depressions in long natural drains, where the likelihood of occurance of aquifers with good yield is more. The thickness of sedimentary formations varies from 10 to 60 m which are presently explored for water of good quality in reasonable quantities for use in the small irrigation schemes.

3 Aquifer Tests Conducted near Catolé do Rocha

The state agency, CAGEPA of Paraib conducted during Oct./Nov., 1979 bore-hole and pumping tests north of the township (Figure 1). The lithological details are given in their annextures of 6 to 9 of the 1979. But no efforts were made either to locate the aquifers or to analyse the effects of nearness of impervious boundaries and recharge sources on the discharge and drawdown characteristics. Such information is needed because of restricted extention of aquifers in the region, simultaneous pumping of water through other wells and the proximity of river beds and lakes to the pumping sites. In the present analysis, only the pumping well PT 2 (Figure 1) was considered (PT stands for tube well). PT 2 has two observation wells, PT 1 and PT 3 at 46 m and 80 m from PT 2, while only one observation well, PT 4 at 57 m distance,was available for

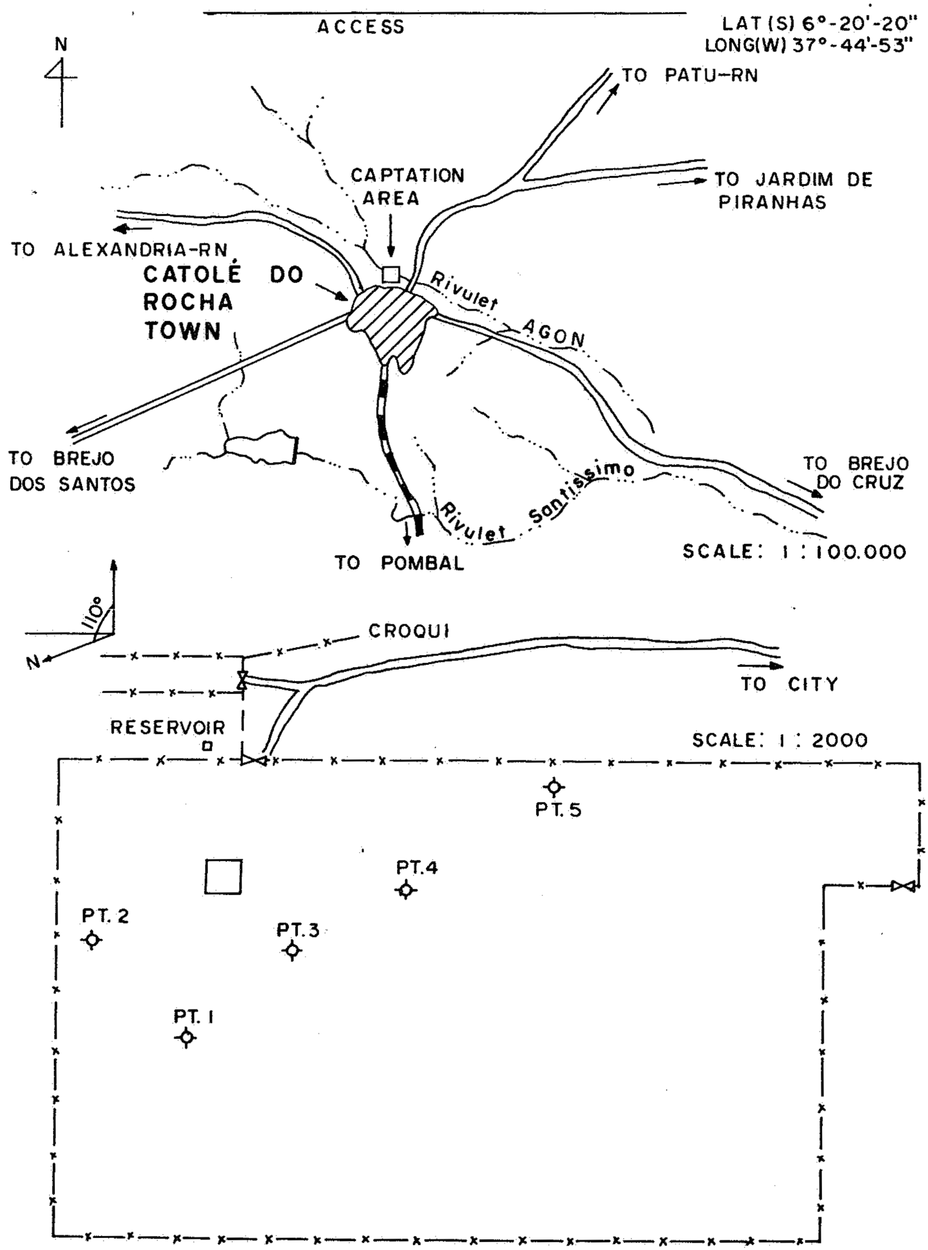


Figure 1 Location map of CAGEPA water supply district at Catolé do Rocha (PB) - showing pumping and observation wells (after SUDENE)

consideration for PT 5.

The pumping test is a useful means of determining the hydraulic properties of water bearing layers and confining beds, but the reliability of the results depends on the accuracy of the values of characteristics of water-bearing layers and properly assumed boundary conditions. It is obvious that the results of any groundwater computation will be erroneous when these values are insufficiently known.

4 Choice of the Method Used

One of the objectives of the pumping test is to obtain information about the yield and drawdown characteristics of the aquifer (Table 1) which data is used to get the specific yield of the formation, a measure of the productive capacity of the well. The other objective is the determination of the characteristics of water-bearing layers. The latter is more aptly called the aquifer test because it is the aquifer, not the pump nor the well that is being tested. The data obtained from the recovery test permits the evaluation of transmissivity, this serving as a check on the results. Of the many methods available for analysis of field data on aquifers, the Jacob method is presently adopted which permits an aproximate solution to Theis' transient equation using the straight-line method which is simple and may offer advantages over the type-curve methods. In Jacob's method, in the plots of drawdown, s vs log of distance, r of the well and s vs log of time, t, the points will not fall on a straight line, unless pumping has continued for such a long time that U, in the well equation becomes less than 0.01. The time required to reach this condition increases with increasing distance from test well. Further, the interception of the boundary by the ever-widening cone of depression results in change in slope of the drawdown curve. For an impermeable (negative) boundary, the slope theoretically increases, and for recharge (positive) boundary, it reduces. Because of the additional geologic and hydrologic information which can be obtained through Jacob's method, its use is demonstrated in the present analysis.

Table 1 Pumping and observation well data on PT 2, PT 1, and PT 3.

Pumping well: PT 2; $Q = 9.6\ m^3/h$: $r_w = 75$ mm; $R_1 = 46$m; $R_3 = 80$ m

Date & time hour mts	PT 1 Dynamic Level, m	PT 1 Drawdown m	PT 2 Dynamic Level, m	PT 2 Drawdown m	PT 3 Dynamic Level, m	PT 3 Drawdown m
21.11.1979						
08:00						
08:01	6.575	0.025	6.185	0.005		
08:02	6.583	0.233	6.186	0.006		
08:03	6.585	0.235	6.189	0.009		
08:04	6.590	0.240	6.191	0.011		
08:05	6.595	0.245	6.193	0.013		
08:06	6.601	0.255	6.194	0.014		
08:08	6.605	0.255	6.197	0.017		
08:10	6.608	0.258	6.200	0.020		
08:15	6,619	0.269	6.205	0.025		
08:20	6.628	0.278	6.213	0.033		
08:25	6.637	0.287	6.218	0.038		
08:30	6.645	0.295	6.223	0.043	6.540	0.010
08:40	6.662	0.312	6.233	0.053		
08:50	6.673	0.323	6.241	0.061	6.550	0.025
09:00	6.681	0.331	6.249	0.069		
09:10	6.685	0.335	6.255	0.075		
09:20	6.688	0.338	6.261	0.081		
09:40	6.692	0.342	6.270	0.090		
10:00	6.708	0.358	6.281	0.101	6.560	0.030
10:30	6.728	0.378	6.302	0.122		
11:00	6.744	0.394	6.315	0.135		
12:00	6.775	0.425	6.340	0.160	6.580	0.050
13:00	6.795	0.445	6.360	0.180		
14:00	6.813	0.463	6.378	0.198	6.597	0.067
15:00	6.810	0.460	6.384	0.204	6.597	0.067
16:00	6.865	0.515	6.408	0.228	6.605	0.075
17:00	6.905	0.555	6.432	0.252	6.620	0.090
18:00	6.930	0.530	6.458	0.278	6.632	0.102
20:00	6.875	0.525	6.455	0.275	6.645	0.115
22:00	6.815	0.465	6.428	0.248		
24:00	6.760	0.410	6.391	0.211		
02:00	6.719	0.369	6.358	0.178	6.606	0.076
04:00	6.675	0.325	6.324	0.144		
06:00	6.688	0.338	6.310	0.130		
Static w/l	6.350		6.180		6.530	
Dynamic w/l	6.930		6.458		5.645	

5 Analysis of the results

5.1 Calculation of parameters of the aquifer

The results of tests are presented in way of tables and figures. Figure 1 shows the location of pumping and observation wells near the township, together with a location map of the approachroads and the rivulet Agon passing through the area under study. Table 1 shows time-drawdown data on PT 1 and PT 3 being the observation wells for the pumping well PT 2. Figures 2 and 3 give the drawdown in arith. & log scale to time in log scale, which serve to locate the impervious boundaries, using the straight-line analysis of Jacob, and using the time-drawdown is used to obtain the recharge boundary location by type curve analysis.

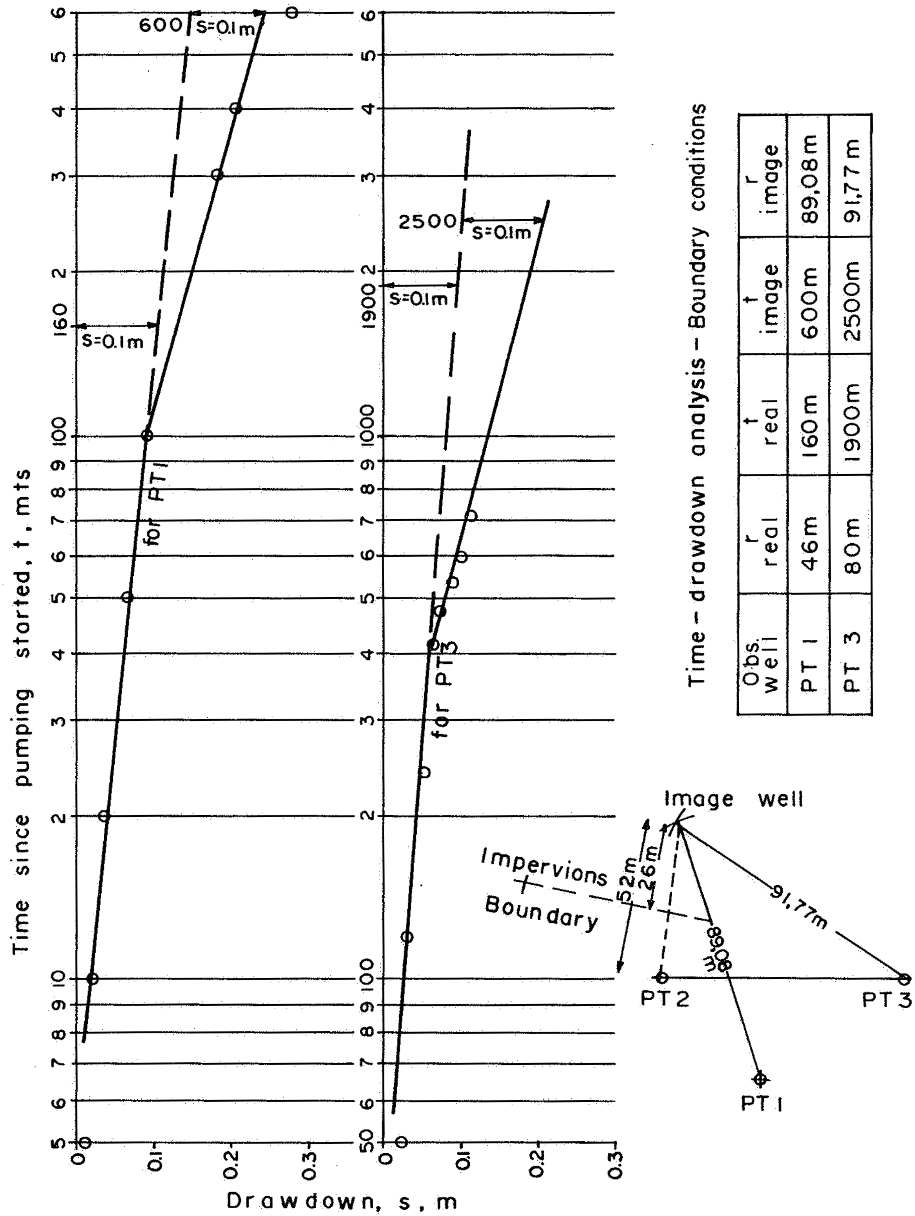
From the aquifer test data on PT 1, the transmissivity co-efficient (T) and the storage co-efficient (S) are respectively calculated as 23.46 m^2/h and 0.0029. The recovery analysis for this test, yielded T = 25.87 m^2/h, using a value of 0.00665 for Δ (s - s') which is the recovery along one log-cycle. The storage co-efficient from the recovery data works out to be 0.0033, which value is slightly higher than that originally obtained from data on PT 1. Correspondingly, for the data (not presented herein) on test well PT 4, T = 42,80 m^2/h and S = 0.0001 and the values for the recovery test data are T = 36.82 m^2/h and S = 0.00068.

5.2 Boundary Location by type curve analysis

Figures 2 and 3 gives plots between s-log t and log s vs log t on PT 1 and PT 3, which are the observation wells for PT 2. While the s-log t plot would give information on the existence or not of the impervious boundaries, depending upon the change in inclination of the plots, the determination of recharge boundary demands data to be plotted on bi-log scale. PT 4 being the only observation well for the pumping well PT 5, the location of recharge front can only be made from data on PT 1 and PT 3.

5.3 Determination of the impermeable boundary

When the area of influence of discharging well reaches an impermeable boundary, the rate of drawdown changes. This results from the influence of a hypothetical image well over the real well. The transition of one leg to another is however not sharp, but follows a curve. Time, t_i is determined for a certain drawdown difference between the second leg and the extension the first leg, which point should not be where the plotted points are on a curve. The distance to the image well causing the same amount of drawdown (or recharge for a recharge boundary) can be determined by the relationship $r^2_{real}/t_{real} = r^2_{image}/t_{image}$. In figure 2, $r_{real} = 46$ m, $t_{real} = 160$ mts, $t_{image} = 600$ mts. The radius of image well equals 89,08 m for the pumping well PT 2. Correspondingly for $r_{real} = 80$m, $t_{real} = 1900$ mts, $r_{image} = 91.77$ m for $t_i = 2500$ mts.



Time - drawdown analysis - Boundary conditions

Obs. well	r real	t real	t image	r image
PT 1	46m	160m	600m	89,08m
PT 3	80m	1900m	2500m	91,77m

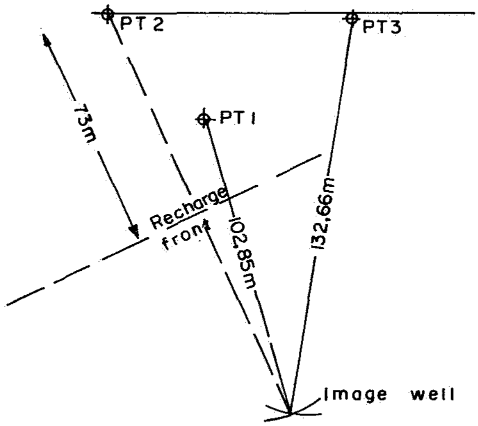
Figure 2 Impermeable boundary determination by straight-line analysis for observation wells, PT 1 and PT 3

5.4 Recharge well location

Figure 3 shows the relation between the type curve and the data from observation wells PT 1 and PT 3. To determine the distance to a recharge boundary from the pumping well, the data curve is fitted to the type curve as shown in Figure 3. The successive steps to read the data are: a) - difference in drawdown between the type curve and the data curve s_i is noted for any time t_i; b) - this difference is spotted on s axis and t_r noted where s_i value intersects the trace of the type curve. The values of t_i and t_r, times of equal drawdown result from the image and real wells respectively. At a radius of 46 m from test well, s_i = 0.20 m when t_i = 250 mts. Transferring s_i = 0.20 m to the s axis of the graph and reading across to the type curve, t_r = 500 mts. With r_{real} = 46 m, with the relation given between r and t, r_i works out as 102.85 m. Similary, for the r_r = 80 m, the corresponding r_i = 132.66 m. To locate the image well and the boundary of the recharge front, the construction is shown in the inset of Figure 3. Thus the intersection of the circles marks the approximate location of the image well, and is located at mid-point and is normal to the line. The existance of the recharge boundary 73 m from PT 2 is evident from Table 1 which shows decreased drawdowns after 600 mts of pumping and by 1320 mts, 0.278-0.130=0.148 m of recovery has taken place because of the effect of recharge.

6 Drawbacks in Existing Procedures

In the present study, the installation of wells and related details are given in anextures 6 to 9 of the SUDENE/CAGEPA report. In that, little consideration is given to spacing of wells as per radius of influence of wells, and consequent interference effect, as also to systematic collection of data. While the data on some wells is missing, more so in the first 10 to 15 mts of pumping, for certain other wells, recovery tests were not conducted till the full recuperation of the freatic condition of the aquifer. Further, the initial (static) water level was taken for granted to be horizontal. While altogether 7 pumping tests were conducted during the last week of November, 1979, only one pumping well was having two observation wells. In this too, the recuperation



Calculations for recharge front location

Obs. well	s ' (m)	r real	t real	t image	r image
PT I	0,2	46 m	50 m	250 m	102,85 m
PT 2	0, 17	80 m	200 m	550 m	132,66 m

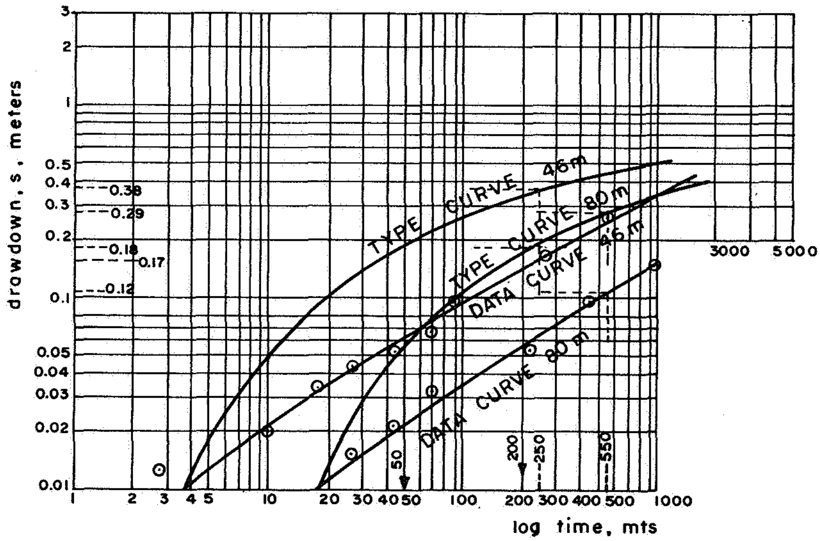


Figure 3 Recharge boundary location by type-curve analysis PT 1 and PT 3

test results were not complete. Such lacunes in aquifer tests are not however uncommon in developing countries and the present analysis is only an attempt to analyse the data with the existing tools of analysis.

7 Conclusions

The area north of rivulet Agon near Catolé do Rocha seems to be a good source for groundwater exploration in the semi-arid Paraiba. In view of the good water quality which was tested by the CAGEPA, advantage is to be taken of the existing aquifers of reasonable storage capacity and transmissivity co-efficients. However, the tube well technique is costly in the north-east Brazil. Alternative methods to adapt shallow wells in alluvial deposits are on way, specially to meet the needs of the small and medium level agriculturists. The author's involvement in the projects of Polonordeste and the CNPq would endeavour to utilise the more recent mathematical techniques and experimental procedures, thus avoiding the lacunas pointed out in the tests.

Acknowledgments

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References

FIPLAN-UFPb Report 1980 - Irrigation Potential in Paraiba State, vol I, Natural Resources, pp 1 - 3027.

U.S.D.I. - 1981 Ground Water Mannual. Denver, U.S.A. pp 85-167.

INTERPRETATION OF FIELD MEASUREMENT

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Abstract - Resumé

The interpretation of field measurements to determine geo-hydrological properties is based on calibration of a simulation model which is chosen to fit best the observed behaviour. This choice is rather arbitrary. To avoid complexity one is apt to apply simple mathematical models accepting the nature of the phenomenon to be covered only in a schematic way. Fortunately, the agent looked at, in particularly the pore pressure, is not seriously affected. However, the behaviour of related features, such as surface leakage and shrinkage, is not that insensitive. The present article elucidates this aspect.

L'interprétation des résultats de mesures in-situ pour la détermination des caractéristiques géo-hydrologiques est basée sur l'étalonnage d'un modèle qui répond le mieux possible au comportement observé. Le choix du modèle est cependant plutôt arbitraire. Afin d'éviter une certaine complexité on se sert souvent d'une modèle mathématique simple tout en acceptant que le caractère transitoire des phénomènes ne soit pas traîté rigoureusement.Heureusement, il apparaît que la grandeur considérée, notammant la pression interstitielle, n'est pas influencée outre mesure par cette approximation. Toutefois le comportement d'autres phénomènes, comme le débit sortant en surface et la dilatation du sol, n'est pas aussi insensible au choix du modèle. Cet aspect est traîté dans cette contribution.

1 Introduction

A frequent geo-hydrological system is a leaky aquifer system consisting of an aquifer and an aquitard on top. The aquitard contributes to the groundwater flow in the aquifer by leakage. For stationary situations this contribution is expressed in terms of the so-called leakage factor λ, a fundamental geo-hydrological constant. Time-dependent behaviour is usually formulated by means of the storativity S of the aquifer, whereas the concept of a constant leakage factor is sustained. This approach is the base of most conventional interpretation methods suited to determine geo-hydrological parameters, such as transmissivity KD, storativity and leakance (Hantush, 1964; Kruseman and de Ridder, 1971).
For time-dependent flow conditions the contribution of an aquitard can not be properly described by a constant leakage factor. The aquitard will shrink or swell due to a change in the aquifer pore pressure, and this process controls the actual leakage to the aquifer. The physical explanation of this aspect is described elsewhere (Barends, 1982). Here, the attention is focused to the influence on the interpretation of field measurements. Two particular situations are addressed: seepage under coastal dykes and pumping test evaluation.

2.1 Tydal porous flow in a leaky aquifer system

In an estuary or a tydal river area the inland is usually protected by dykes. If the subsoil is stratified with on top a semi-pervious layer (aquitard) the influence of fluctuation in the free water outside will extend underneath the dyke in the permeable layer (aquifer). The pore pressure in the aquifer will respond to these fluctuations damped out by the storativity of the subsoil. This damping character is related to the area of influence expressed by the leakage factor. Therefore, it is essential to simulate this damping phenomenon in a correct way. To show how this aspect is covered by various simulation models a particular situation is discussed in detail. It concerns the response of a leaky aquifer to a cyclic plane-symmetric boundary condition, according to:

$$H(x,t) = H_0 \cos(\omega t) \tag{1}$$

MODEL I: a rigid aquifer and a rigid aquitard with a constant leakage factor. The response in the aquifer becomes:

$$H(x,t) = H_0 \exp(-x/\lambda)\cos(\omega t) \tag{2}$$

in which λ represents the leakage factor: $\lambda = \sqrt{KDC}$, and C is the aquitard resistance: $C = d/k$.

MODEL II: a deformable aquifer (elastic storage) and an impervious top layer. The response in the aquifer becomes (Verruijt, 1982):

$$H(x,t) = H_0 \exp(-x/\lambda_\omega)\cos(\omega t - x/\lambda_\omega); \quad \lambda_\omega = \sqrt{2c/\omega} \tag{3}$$

in which c represents the aquifer compaction coefficient, related to the aquifer storativity S by: $c = KD/S$.

MODEL III: a rigid aquifer and a deformable aquitard. The vertical consolidation process in the aquitard is formulated according to Terzaghi, and controlled by the consolidation coefficient c' of the aquitard. The response in the aquifer becomes:

$$H(x,t) = H_0 \exp(-x/\lambda_\omega)\cos(\omega t - \alpha x/\lambda_\omega); \; \lambda_\omega = \lambda/\sqrt[4]{2\delta^2}\cos(\pi/8); \tag{4}$$
$$\alpha = \tan(\pi/8); \; \delta = d\sqrt{\omega/2c'}$$

MODEL IV: a deformable aquifer and a rigid aquitard. The leakage is directly related to changes in the aquifer piezometric head H; no time delay due to processes in the aquitard are considered. This model is popular for the evaluation of pumping tests (Hantush, 1964). The response in the aquifer becomes:

$$H(x,t) = H_0 \exp(-x/\lambda_\omega)\cos(\omega t - \alpha x/\lambda_\omega); \; \lambda_\omega = \lambda\sqrt{1+\alpha^2}\,/\sqrt[4]{1+\beta^2}\,; \tag{5}$$
$$\alpha = \tan(\tfrac{1}{2}\mathrm{atan}\beta); \; \beta = \omega\lambda^2/c$$

MODEL V: a deformable aquifer and a deformable aquitard. The response in the aquifer becomes:

$$H(x,t) = H_0 \exp(-x/\lambda_\omega)\cos(\omega t - \alpha x/\lambda_\omega); \; \lambda_\omega = \lambda\sqrt{1+\alpha^2}\,/\sqrt[4]{\delta^2(1+\varepsilon^2)}\,; \tag{6}$$
$$\alpha = \tan(\tfrac{1}{2}\mathrm{atan}\varepsilon); \; \varepsilon = 1 + \beta/\delta; \; \beta = \omega\lambda^2/c; \; \delta = d\sqrt{\omega/2c'}$$

The derivation of formulas (4), (5) and (6) is outlined in an appendix.

To verify the applicability of these simulation models a particular site measurement is worked out. The situation concerns the tydal response of a leaky aquifer, presented in Figure 1. The graph shows the correlation of the piezometric head at two positions, one at the outside of a protection dyke and the other at the innerside. Several high tides have been recorded. The plotted data correspond to tilted ellipses, which incorporate amplitude damping and phase shift. From these measurements the parameters of the geo-hydrological system can be determined, but the values depend on the simulation model chosen.

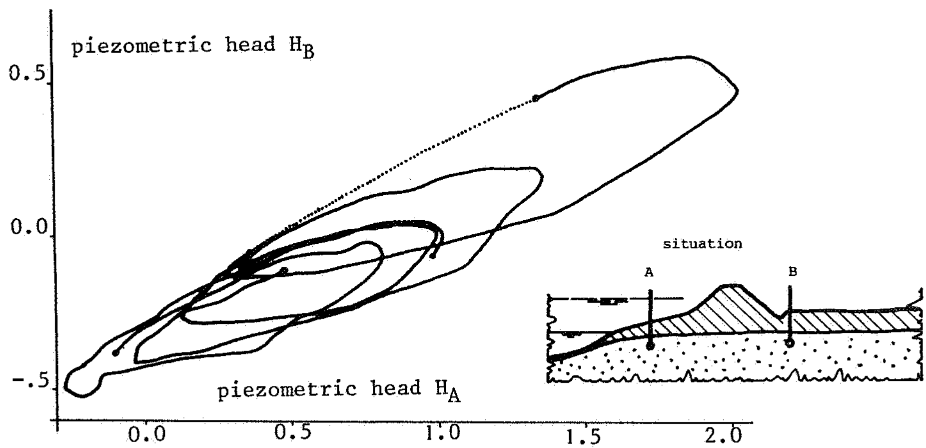

Figure 1 Measured behaviour of the tydal response in a leaky aquifer

The ellipses can be expressed by the following formula (see appendix):

$$H(x,t) = H_0\exp(\ln(f))\cos(\omega t - g) \tag{7}$$

in which f and g are functions of the inclination angle θ of the ellips, and of the ratio m of the main radials of the (average) ellips; $m < 1$:

$$f = \sqrt{(1 + m^2 tg^2\theta)/(1 + tg^2\theta/m^2)};$$
$$g = atan(1/(2(1/m - m)\sin 2\theta))$$

For the situation in Figure 1 it is found: $\theta = 21^\circ$ and $m = 0.3$.

Comparison with the formulas (1) to (6) leads to the following values:

$$x/\lambda_\omega = -\ln(f) = 0.42; \qquad \alpha = -g/\ln(f) = 0.58$$

Since the distance between both positions is about 50m, one finds that:

$$\lambda_\omega = 119.1\mathrm{m}$$

On the base of these measured values, the fundamental parameter λ of the geo-hydrological system can be determined. Table 1 compiles the results of the evaluation by means of the previously posed simulation models. For the interpretation of MODEL IV and V more information is required, and the following values are assumed (based on other information): $d = 3\mathrm{m}$; $c' = 10^{-8}\,\mathrm{m}^2/\mathrm{s}$; $\omega = 2.3\ 10^{-5}$ rad/s; ($\lambda \simeq 400\mathrm{m}$).

Table I. Evaluation of measured tydal fluctuation in a leaky aquifer.

MODEL	measured α	calibrated: ε	β	δ	c	λ_ω/λ
I	(0.00)	-	-	-	∞	1.00
II	(1.00)	-	-	-	0.16	1.00
III	(0.41)	-	-	10.0	∞	0.29
IV	0.58	-	1.75	(1.0)	2.10	0.81
V	0.58	1.75	7.50	10.0	0.49	0.26

values between brackets are conditioned by the model itself.

Model I is not applicable; it misses time delay. Model II cannot account for the measured value of α. In model III the fixed value for α is not correct, but as an approximation acceptable. Model IV results in a value for the aquifer storativity, which is rather low (high c value). Model V seems most complete, but it is complicated. The last column clearly shows the different effects of the various models. It deviates considerably. Extrapolation to other regimes of flow variation, a different ω, may lead to erroneous results. If one is interested in related features, which actually happen in the aquitard, a model should be chosen that incorporates this phenomenon correctly. In the considered case Model III or V are preferable. For the determination of geo-hydrological parameters the simulation model has to be chosen with great care.

2.2 Pumping test evaluation

Groundwater recovery established by wells evolves a porous flow field that may extend in a wide area inducing environmental consequences for land and water management. It is common practice to evaluate the behaviour of a geo-hydrological system by observing the response due to a pumping test. A short test is more economic; and often it is stated that short-test data are suited to determine the required system parameters accurately. Since in many systems a flow field established in a short period is not stationary, it is questionable whether correct values can be obtained. The previous chapter shows that for a cyclic agitation in a leaky aquifer with a period of about 12 hours the damping and delay properties are not easily determined. The model choice plays an important role.

To underscore this aspect a particular pumping test is evaluated. The test has been performed to determine the properties of a leaky aquifer, and to design a temporary artificial drainage system for the construction of a subsurface parkinghouse. At five positions the response in the aquifer has been observed, once due to a short test (40 min) and once to a long test (some days). In Table II the results of both tests are compiled. These values have been obtained from the measured data by application of the Hantush' well-function W, which is valid for a fully penetrating constant well in a leaky aquifer with a constant leakance λ:

$$H(r,t) = \frac{Q}{4\pi KD} W\left(\frac{r^2}{4ct}, \frac{r}{\lambda}\right) \tag{8}$$

The procedure followed is curve-fitting (nonlinear regression) in each observation point to obtain the transmissivity KD, the leakance λ, and the storativity S (or $c = KD/S$). On the base of observed deviation in measured data and the theoretical most correct response curve the accuracy of the determined geo-hydrological material properties could be determined, expressed in terms of a standard deviation. To find a reasonable accuracy a special iteration technique has been adopted, as the sensitivity of the applied well-function with respect to both variables $r^2/4ct$ and r/λ is essentially different (Villiers and Glasser,1981). The data from the first observation point are less reliable, because the filter clogged in the beginning of the first test.

Moreover, the first two observation points are influenced by the fact that the well partly penetrated the aquifer. Nonetheless, the data in Table 2 show a clear tendency: all parameters vary monotonically with the distance to the well. This can not be explained by inaccuracy, since the standard deviation is small. And it can not be explained by incompleteness of the test (too short pumping period), since the results of the short and long test do agree in absolute sense. It is more likely, that this tendency is due to a physical phenomenon which is not well covered by the chosen simulation model. In accordance with the findings of the previous section recalling that the Hantush' well-function falls in the class of model IV, which does not take into account the process in the covering aquitard in a correct way, the reason of the noticed coherent deviation in the results mentioned in Table 2 is probably due to the real effect of the aquitard. Some qualitative explanation, which stresses this conclusion, is given, but a quantitative proof is not presented here.

Table 2 Results of a short and long pumping test evaluation

r(m)	KD	m^2/day	λ	m	S	10^{-5}	c/λ^2	1/day
5	100(2)	105(2)	38(1)	43(2)	---	---	3.60	3.21
10	123(3)	138(3)	106(6)	144(10)	364(9)	273(9)	3.01	2.44
20	155(3)	166(4)	216(12)	259(18)	127(3)	118(6)	2.62	2.10
40	195(7)	197(7)	387(39)	397(35)	118(4)	111(6)	1.10	1.12
80	264(4)	265(7)	762(36)	762(54)	87(1)	78(3)	0.52	0.59
	short	long	short	long	short	long	short	long

the values between brackets denote the standard deviation

The leakage from the aquitard is due to the flux at the separation between the aquifer and the aquitard. This flux attains an instantaneous large value at the very beginning of the test nearby the well, and then decreases rapidly, but a constant value is obtained when the consolidation process in the aquitard is completed. For a deformable low-permeable layer this process may take years. When the aquitard contains sand lenses this period is strongly reduced (Vreeken and Van Duyn, 1983). This consolidation process explains that in the beginning the leakance nearby the well is abundant, and consequently a small leakage factor is obtained by a model IV evaluation. At further distance from the well the leakage is mobilised at later stage, thus averaging the response there over the total pumping period will result in a large leakage factor. The reason why piezometric data in the aquifer due to a short pumping test are hardly affected by the time-dependent leakage phenomenon is due to the insensitivity of the applied well-function with respect to the factor r/λ. This fact explains the success of the application of the Hantush' well-function to determine the transmissivity KD of a leaky aquifer, but the corresponding parameters related to the aquitard, to wit: the leakage factor and the consolidation coefficient or storativity are determined only in an approximate sense. They are underestimated by the Hantush' well-function. To correctly evaluate environmental consequences such as surface leakance and surface subsidence due to well production the evaluation of the geo-hydrological system by means of a well-function evaluation of a pumping test is not sufficient. Additional soil investigation is required to determine the fundamental material properties. In this regard the dipool sounding facility is recommanded (Rietsema, 1983).

References

Barends, F.B.J. 1982. Seepage Induced Surface Leakage in and around Polders, Int.Symp.'Polders of the World',ILRI Pub.,Vol.1,pp.686-699.

Hantush, M.S. 1964. Hydraulics of Wells, Adv. Hydrosc.,Vol.1,pp.281-699.

Kruseman,G.P. and De Ridder, N.A. 1970. Analysis and Evaluation of Pumping Test Data,Bulletin 11,ILRI Pub.,Wageningen,the Netherlands.

Rietsema, R.A. 1983. A New Method for in Situ Measurement of Permeability Int.Symp.'Methods and Instrumentation ..',Noordwijkerhout.

Verruijt, A. 1982. Theory of Groundwater Flow, 2nd.ed.,MacMillan Press.

Villiers, and Glasser, 1981. A Continuation Method for Non-Linear Regression, SIAM Jrn.,Numerical Analysis,Vol.18,no.6.

Vreeken, C. and Van Duyn, C.J. 1983. The Effect of Soil Heterogeneity on the Consolidation by Vertical Drains, to appear 8 ECSMFE, Helsinki.

APPENDIX

EVALUATION OF CYCLIC RESPONSE IN LEAKY AQUIFER SYSTEMS

Plane-symmetrical horizontal seepage in leaky aquifer systems involves two phenomena, compaction and flow in the aquifer and conslidation or shrinkage and swelling in the aquitard, described by:

$$H_{,xx} = H_{,t}/c + q_o/KD \qquad x\geq 0,\ t>0,\ z<0 \qquad (a1)$$

$$h_{,zz} = (h-\sigma/\gamma)_{,t}/c' \qquad x>0,\ t>0,\ z>0 \qquad (a2)$$

$$q_o = -K'h_{,z} \qquad x>0,\ t>0,\ z=0 \qquad (a3)$$

$$h = H \qquad x>0,\ t>0,\ z=0 \qquad (a4)$$

Here, c represents the compaction coefficient, c' the consolidation coefficient, q_o the leakage flux at the separation z=0, KD the aquifer transmissivity, K' the permeability of the aquitard, and H and h are the piezometric heads in the aquifer and the aquitard, respectively. The term σ/γ is related to the surface load, which is assumed constant.

For a cyclic boundary condition, $H(x,t) = H_o\cos(\omega t)$, the harmonic response is found by separation of variables, according to:

$$H(x,t) = \bar{H}(x)\exp(i\omega t);\ h(z,t) = \bar{h}(z)\exp(i\omega t);\ q_o(x,t) = \bar{q}_o(x)\exp(i\omega t)$$

The set of equations (a1-4) is transformed accordingly, and the solution in the aquitard becomes: $\bar{h} = \bar{H}\exp(-z\sqrt{i\omega/c'})$. The flux $\bar{q}_o$ becomes then: $\bar{q}_o = (\bar{H}/C)(1+i)\delta$; $\delta = d\sqrt{\omega/2c'}$; $C = d/K'$; where d is the aquitard size.

Substitution into equation (a1) leads to the following solution:

$$H(x,t) = H_o\exp(-x/\lambda_\omega)\exp(i\omega t) \qquad (a5)$$

$$\lambda_\omega = \lambda(\sqrt[4]{\delta^2(\varepsilon^2+1)}(\cos\tfrac{1}{2}\mathrm{atan}\varepsilon + i\sin\tfrac{1}{2}\mathrm{atan}\varepsilon))^{-1}$$

$$\lambda = \sqrt{KDC}\ ;\ \varepsilon = 1 + \beta/\delta\ ;\ \beta = \omega\lambda^2/c\ ;\ \delta = d\sqrt{\omega/2c'}$$

The response related to the imposed cyclic boundary condition is the real part of (a5), and elaboration results in:

$$H(x,t) = H_o\exp(-x/\lambda_\omega)\cos(\omega t - x/\lambda_\omega\ \tan\tfrac{1}{2}\mathrm{atan}\varepsilon) \qquad (a6)$$

If the aquifer is rigid, then $c \to \infty$, and $\beta=0$. The solution can be obtained from (a6). If the aquitard is rigid, the situation is different, since the leakage adjusts itself instantaneously with respect to the outside condition on top of the aquitard (at z =d), and $q_o = -H/C$. All damping in the aquitard is excluded, and the corresponding solution becomes like (a6), but with: $\lambda_\omega = \lambda(\sqrt[4]{\beta^2+1}\cos\frac{1}{2}\mathrm{atan}\beta)$; $\alpha = \tan\frac{1}{2}\mathrm{atan}\beta$. This solution is related to (a6) for $\delta=1.0$, which expresses that the relevant zone in which the cyclic response in the aquitard acts is in the order of d, the thickness of the aquifer. For a consolidating aquifer this zone is equal to $d/\delta = \sqrt{2c'/\omega}$; it is usually much smaller than d.

The response presented in Figure 1 can be expressed by: $H_A = y = y_o\cos\omega t$ and: $H_B = x = x_o\cos(\omega t-\psi)$. Elimination of ωt leads for $\psi<\pi/2$ to:

$$(xy_o)^2+(yx_o)^2-2(xy_o)(yx_o)\cos\psi = (x_oy_o)^2\sin^2\psi \qquad \text{(a7)}$$

Next, consider the analytical formulation of an ellips with m the ratio between the main radials co-axial with the axes x' and y':

$$(x'b)^2+(y'a)^2=(ab)^2; \quad m=b/a$$

Rotation over angle θ into the xy-system ($\theta<\pi/2$) leads to:

$$x^2(b^2\cos^2\theta+a^2\sin^2\theta)+y^2(a^2\cos^2\theta+b^2\sin^2\theta)-2xy\cos\theta\sin\theta(a^2-b^2)=a^2b^2$$

and comparison with (a7) results in four conditions, which all match:

$$x_o=\sqrt{(a^2\cos^2\theta+b^2\sin^2\theta)}; \quad y_o=\sqrt{(b^2\cos^2\theta+a^2\sin^2\theta)};$$

$$x_oy_o\cos\psi = \cos\theta\sin\theta(a^2-b^2); \quad x_oy_o\sin\psi = ab$$

This implies that the response y(x) can be expressed by a tilted ellips. The following relations hold:

$$\psi = \mathrm{atan}(m/(2(1-m^2)\sin2\theta)) = g(m,\theta) \qquad \text{(a8)}$$

$$x_o/y_o = \sqrt{(1+m^2\mathrm{tg}^2\theta)/(1+\mathrm{tg}^2\theta/m^2)} = f(m,\theta) \qquad \text{(a9)}$$

Finally the relation between H_A and H_B or y(x) can be expressed by:

$$x = y_o\exp(\ln(f))\cos(\omega t - g) \qquad \text{(a10)}$$

which can be evaluated on the base of determining the oblique angle θ and the ratio of the main radials of an average ellips representing a continuous cyclic response such as presented in Figure 1.

RELIABILITY AND ACCURACY OF THE DETERMINATION OF TRANSMISSIVITY WITH WELL TESTS

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Abstract

Compared with pumping tests, a well test provides only information about the transmissivity of an aquifer. A well-known method to obtain transmissivity values from pumping tests, is given by the Theis-Jacob method for unsteady, confined conditions.
Under some assumptions - expressed in time criteria - also problems under semi-confined and unconfined conditions can be solved from well tests with this method.
Also time criteria are determined to use the Theis-Jacob method after all, although the circumstances do not match with the assumptions underlying the Theis-Jacob method.
The Theis-Jacob method applied with the needed time criteria is tested on pumping test data to investigate reliability and accuracy of well tests evaluated with this method, compared with the pumping test transmissivity values.

1 Introduction

The most important geohydrological parameters in relation to groundwater flow are the transmissivity (T) and the storage coefficient (S) of aquifers and the vertical hydraulic resistance (c) of semi-pervious layers.

A pumping test is considered to be an accurate and practical method to determine these parameters. A more simplified method is given by the well test; groundwater is extracted from a well with a constant discharge rate and the response of the aquifer is only measured in the pumped well. As the well test provides only information about the time depending drawdown, only the transmissivity can be determined. Therefore, the execution and evaluation of a well test must be considered with respect to the other unknown parameters.

A simple and reliable evaluation method for pumping tests to obtain transmissivity values is given by Theis (1935), simplified by Jacob (1950) for unsteady and confined conditions.

The selection of the drawdown section, which has to be used with this Theis-Jacob method in order to determine the transmissivity, and the application of the Theis-Jacob method for semi-confined and unconfined conditions is subject of investigation in this paper.

2 Evaluation methods for confined, semi-confined and unconfined conditions

Combination of Darcy's law and the law of continuity results in a partial differential equation, which describes the flow of groundwater in a saturated porous medium. In case of two-dimensional radial flow to a pumped well it can be written as:

$$\frac{\partial^2 s}{\partial r^2} + \frac{1}{r}\frac{\partial s}{\partial r} = \frac{S}{T}\frac{\partial s}{\partial t} \qquad (1)$$

where

s = drawdown (L)

r = radial distance to pumped well or well diameter (L)

S = storage coefficient (-)

T = transmissivity (L^2T^{-1})

t = variable of time (T^{-1})

According to Theis the solution of s is given by:

$$s = \frac{Q}{4\pi T} = W(u) \tag{2}$$

where

Q = discharge rate (L^3T)

$W(u)$ = Theis' wellfunction

$u = \frac{r^2S}{4Tt}$

For $u < 0{,}01$ this can be simplified (error < 0,3%) to (Cooper and Jacob, 1946, Jacob, 1950):

$$s = \frac{2{,}3\ Q}{4\pi T} \log \frac{2{,}25\ Tt}{r^2S} \tag{3}$$

The transmissivity can now be deduced:

$$T = \frac{2{,}3\ Q}{4\pi\Delta s} \tag{4}$$

where

Δs = change in drawdown during one log cycle of time (Figure 1)

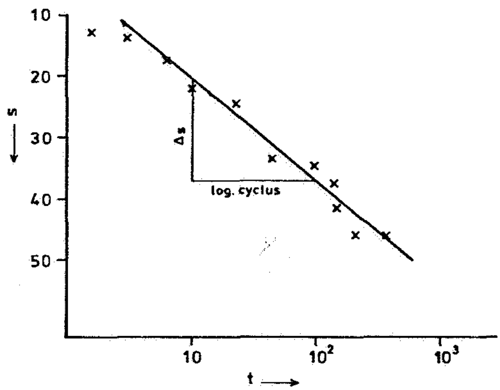

Figure 1 Drawdown versus the logarithm of time

This wellknown Theis-Jacob method is considered to be useful only under confined and unconfined conditions.
In case of semi-confined conditions a solution for s is given by

Hantush and Jacob (1955) analogous to Theis:

$$s = \frac{Q}{4\pi T} W\ (u, r/\lambda) \tag{5}$$

where

$\lambda = \sqrt{Tc}$ (L)

c = vertical hydraulic resistance of semi-perveous layer (T)

According to Hantush (1964) $W(u, r/\lambda) \simeq W(u)$ if $u > 5r^2/\lambda^2$ under the condition $r/\lambda < 0,1$.
If data of the pumped well are used, r_o (= well radius) will be small and the boundary condition of $r/\lambda < 0,1$ will be satisfied.
Due to this simplification the Theis-Jacob method can also be used under semi-confined conditions.
As mentioned before, the Theis-Jacob menthod can also be used for unconfined conditions. In some cases, however, delayed yield occurs, due to loamy and silty material in the aquifer. Analogous to Theis, Boulton (1963) gives the following solutions:

$$s = \frac{Q}{4\pi T} W(u_a,\ r/B) \tag{6a}$$

where

$u_a = \frac{r^2 S}{4Tt}$

B = indication for leakage conditions

$$s = \frac{Q}{4\pi T} W(u_y,\ r/B) \tag{6b}$$

where

$u_y = \frac{r^2 S'}{4Tb}$

S' = water storage coefficient = $n_e + S \simeq n_e$ (-)

n_e = effective porosity (-)

Equation 6a deals in case of elastic storage in the aquifer only. The well functions $W(u)$ and $W(u_a, r/B)$ are almost equal if $r/B < 0,1$ and $u_a > 10^{-2}$ (error < 6%).
Boundary condition $r/B < 0,1$ will be satisfied if data of the pumped well are used.

$u_a > 10^{-2}$ results in a time criterium $t < 25r^2S/T$ which normally means that already after at most several minutes this limitation is exceeded. Therefore, determination of transmissivity according to the Theis-Jacob method is not possible from equation 6a.

If the water is mainly provided from effective porosity the drawdown is given with equation 6b. Now the well functions $W(u)$ and $W(u_y, r/B)$ are almost equal if $r/B < 0{,}1$ and $u_y < 2{,}5 * 10^{-3}$ (error < 4%).

Using these boundary conditions also for unconfined conditions with delayed yield the Theis-Jacob method can be used.

The value of the transmissivity can also be determined with the Theis-Jacob method using the recovery period after the pump is shut down. The piezometric or phreatic level will then rise to the initial state. This rise can be considered as the result of superposition of a discharge from $t = t_o$ and a recharge from $t = t_o''$. The residual drawdown s'' is the difference between the original water level prior to pumping and the actual water level measured at the moment t'' (Figure 2):

$$s'' = s(t_o) - s(t'') \tag{7}$$

According to Theis:

$$s'' = \frac{Q}{4\pi T}\left[W(u) - W(u'')\right] \tag{8}$$

where

$$u'' = \frac{r^2S''}{4Tt''}$$

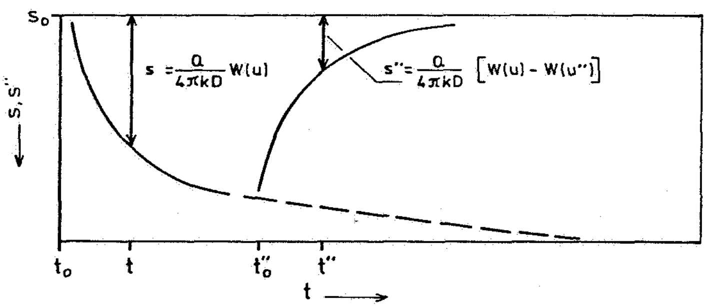

Figure 2 Recovery before drawdown reaches steady state (superposition)

Assuming that the storage coefficient does not change (i.e. $S = S''$) the simplification of Jacob can be applied and the transmissivity can be determined:

$$T = \frac{2,3\ Q}{4\pi\Delta s''} \tag{9}$$

where

$\Delta s''$ = change in residual drawdown during log cycle Δt

In case there is already steady state condition the recovery can be considered as a "negative well test" and the Theis-Jacob method can be applied, replacing s for s', the rise of piezometric level after the pump is shut down and replacing t for t'', the time after the pump is shut down at t_o'' (Figure 3):

$$s' = \frac{2,3\ Q}{4\pi T} \log \frac{2,25\ Tt''}{r^2 S} \tag{10}$$

and consequently:

$$T = \frac{2,3\ Q}{4\pi\Delta s'} \tag{11}$$

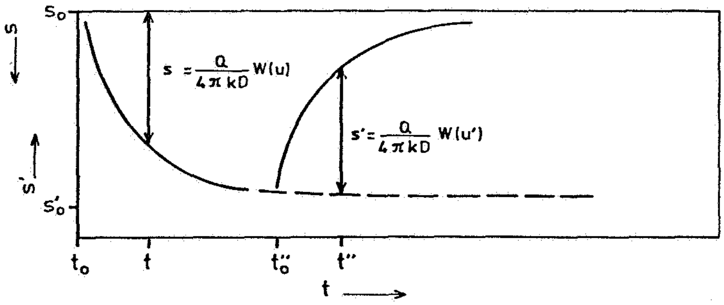

Figure 3 Recovery after drawdown reaches steady state (negative well test)

If the recovery period starts before steady state is reached, the recovery can be considered as a negative well test anyhow, if the extrapolated drawdown is neglectable in respect to the rise of the piezometric level after the pump is shut down.

3 Limiting conditions

3.1 Introduction

The Theis-Jacob method is subject to a number of limiting conditions, but in reality it is not always possible to meet all these conditions. Under some assumptions, however, it is possible to use the Theis-Jacob method anyway.
The aquifer is considered to be of infinite areal extent, because of the relatively small influenced area around the pumped well, due to the limited duration of the well test.
As the well test in general does not provide information about homogenity and isotropy of the aquifer, these characteristics are assumed by force.
The storage capacity of the aquifer is assumed to be constant. In case of unconfined conditions with delayed yield there is a transition from the elastic storage coefficient to the effective porosity; the evaluation method takes this in account however.

3.2 Well storage

In the Theis-Jacob method the storage in the well has to be neglectable. Papadopulus and Cooper (1967) have considered well storage. For a confined aquifer they give the relation:

$$s = \frac{Q}{4\pi T} F(u,\alpha) \tag{12}$$

where

$F(u,\alpha)$ = well function

$$u = \frac{r_o^{\,2} S}{4Tt}$$

$$\alpha = \frac{r_o^{\,2} S}{r_c^{\,2}}$$

r_o = effective radius of well screen

r_c = radius of well casing in the interval over which the water level declines

The well functions $W(u)$ and $F(u,\alpha)$ are almost equal if $u/\alpha < 10^{-2}$ (error < 2%). Consequently if $u/\alpha < 10^{-2}$:

$$s = \frac{Q}{4\pi T}\ F(u,\alpha) \approx \frac{Q}{4\pi T}\ W(u) \tag{13}$$

and now with the Theis-Jacob method the transmissivity of a confined aquifer, and because of the foregoing also of semi-confined and unconfined aquifers, can be determined.

3.3 Well losses

Due to friction losses in and around the well screen there will be an added drawdown Because of the limited disturbed zone around the screen, flow conditions are almost immediately in steady state. Consequently, well losses will not change in time.
In case of well losses equation (3) will change in:

$$s = \frac{2{,}3\ Q}{4\pi T} \log \frac{2{,}25\ Tt}{r^2 S} + s_{wl} \tag{14}$$

where

s_{wl} = added drawdown in consequence of well losses

To determine the transmissivity of an aquifer with the Theis-Jacob method Δs and Δt are used and consequently equation (14) will change in:

$$\begin{aligned}\Delta s &= \left[\frac{2{,}3\ Q}{4\pi T} \log \frac{2{,}25\ T(t+\Delta t)}{r^2 S} + s_{wl}\right] \\ &\quad - \left[\frac{2{,}3\ Q}{4\pi T} \log \frac{2{,}25\ Tt}{r^2 S} + s_{wl}\right] = \\ &= \frac{2{,}3\ Q}{4\pi T} \log \frac{t+\Delta t}{t}\end{aligned} \tag{15}$$

The term s_{wl} disappeared from the equation; the added constant drawdown due to well losses apparently does not influence the determination of the transmissivity. This is, however, only true if the discharge rate is kept constant, because of the relation between discharge rate and well losses.

3.4 Partial penetration

If the pumped well penetrates the entire aquifer it receives water from the total thickness of the aquifer by horizontal flow. In case of partial penetration, however, there will be three-dimensional flow. This results in an added drawdown, which will be constant after some time. Acoording to Hantush (1964) the transmissivity can be calculated with the Theis-Jacob method if $t > SD/2k_v$ (k_v = vertical hydraulic conductivity).

It has to be assumed that the aquifer is homogenous. If not, the calculated transmissivity value may deviate from the real transmissivity, since the mean hydraulic conductivity in the screened section may deviate from the mean hydraulic conductivity of the total thickness of the aquifer.

4 Time criteria

From the foregoing it is obvious that for the determination of transmissivity values of aquifers under several conditions the Theis-Jacob method can be used:

a) The simplification Jacob made on the Theis equation can only be made under the assumption $u < 0{,}01$ ($u = r_o^2S/4Tt$). This results in:

$$t > \frac{25r_o^2S}{T} \tag{16}$$

b) For semi-confined conditions the Theis-Jacob method may be used if $u > 5r^2/\lambda^2$ resulting in:

$$t < \frac{cS}{20} \tag{17}$$

c) In case of unconfined condtions with delayed yield the relation between s en T can be approximated by the Theis-Jacob equation if $u_y < 2.5 * 10^{-3}$, which means:

$$t > \frac{100r_o^2S'}{T} \tag{18}$$

d) The effect of well storage can be taken in account in the Theis-Jacob method if $u/\alpha < 10^{-2}$ ($\alpha = r_o^2 S/r_c^2$) and consequently:

$$t > \frac{25r_c^2}{T} \qquad (19)$$

e) The Theis-Jacob method can be used in case of partial penetration if:

$$t > \frac{SD}{2k_v} \quad \text{or} \qquad (20a)$$

$$t > \frac{SD^2}{\frac{k_v}{k_h} 2T} \qquad (20b)$$

where:

k_h = horizontal hydraulic conductivity

D = thickness of aquifer

In case of isotropy $k_v/k_h = 1$ and so:

$$t > \frac{SD^2}{2T} \qquad (20c)$$

For the determination of the transmissivity from the recovery period the time criteria have to be used for the total time after the pump was started, if there is a considerable extrapolated drawdown (case of superposition).
If the recovery period is looked upon as a negative well test the time criteria have to be used for the period after the pump is shut down.
When the time criteria are determined, the pumping time can be estimated beforehand, or afterwards it can be decided which data obtained from the well test may be used for the determination of the transmissivity value with the Theis-Jacob method.
It is necessary however to know the order of magnitude of the parameters, which determine the time criteria: T,D, S, S' and c; an impression of k_v/k_h is needed to determine the time criterium in case of partial penetration.

a) <u>T value</u> The transmissivity can be estimated from well logs. Afterwards these values must be compared with the calculated value;

eventually the time criteria have to be revised.

b) D The thickness of an aquifer can be deduced from well logs or maps indicating the thickness of an aquifer.

c) S The elastic storage coefficient of sandy aquifers may be estimated from figure 4 (v.d. Gun, 1979) if thickness (D) and depth of the top of the aquifer (d) are known.

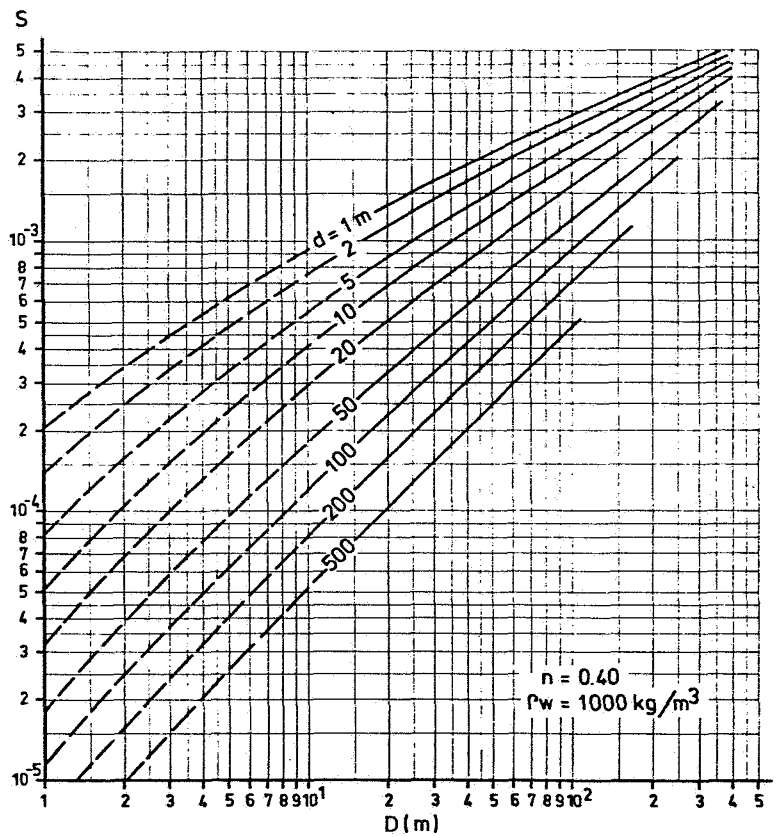

Figure 4 The storage coefficient can be determined with data of thickness (D) and depth (d) of the aquifer

d) S' The storage cofficient of an unconfined aquifer corresponds with the effective porosity ($S' = S + n_e \simeq n_e$). The effective porosity is about 0,3 to 0.4 for Dutch sandy aquifers.

e) c The vertical hydraulic resistance of semi-pervious layers is very hard to determine in an accurate way. This parameter will cause the most problems in fixing the time criterium incase of a semi-confined aquifer.

To get a rough impression the following relation is suggested:

$$c = 100 * D \text{ (D is thickness semi-pervious layer in m)} \tag{21}$$

5 Trial

Thirtytwo pumping tests in the Netherlands evaluated as well tests with the Theis-Jacob mehod are compared with the results of the complete pumping tests in order to trial the utility of the Theis-Jacob method under the restrictions of the before mentioned time criteria. For this purpose data in respect of discharge rate and drawdown in the pumped well are collected. With these data the transmissivity values are determined with the Theis-Jacob method using the time criteria.

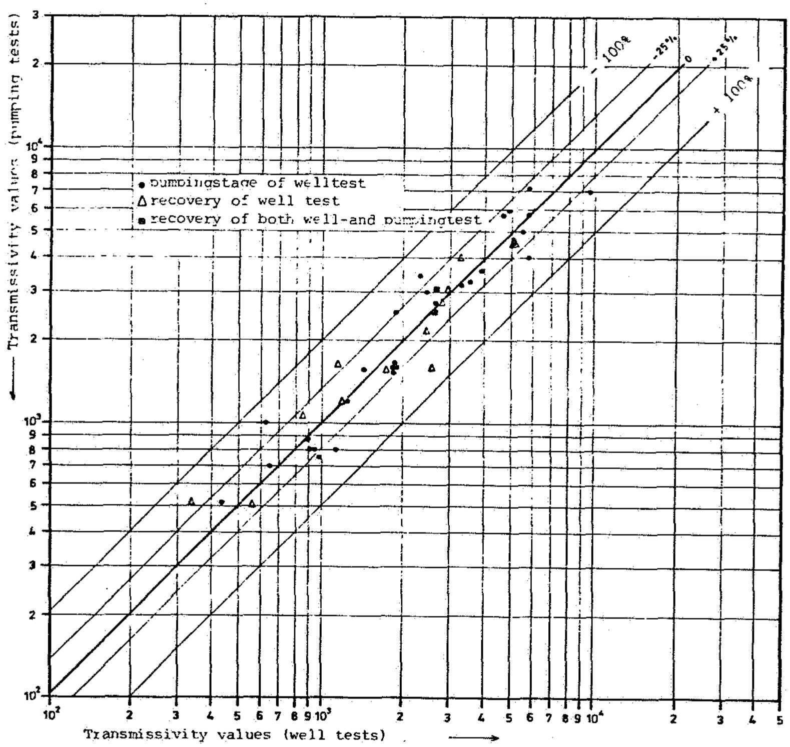

Figure 5 Transmissivity values obtained from pumping tests versus the transmissivity values obtained from well tests

Because it was to compare transmissivity values, the other parameters, needed to determine the time criteria, were adopted from the results of the pumping tests. The results obtained in this way and the transmissivity values calculated from the complete pumping tests are in general comparable.

In all cases the difference of the results obtained in both ways was less than 100%; in threefourths of the cases even less than 25% (Figure 5).

The observed differences may be linked with the inaccuracy of the pumping test results as well as with the Theis-Jacob method. But also due to heterogenity in the aquifer there will always be differences in transmissivity values obtained from data of the pumped well and from data of an observation well at some distance from the pumped well.

6 Conclusion

The Theis-Jacob method seems to be a good applicable way to determine the transmissivity of confined, semi-confined and unconfined aquifers in spite of the simplifications which are used to adapt the several evaluation methods to the Theis-Jacob method.

To use the right section of the drawdown curve in order to determine the drawdown Δs in a log cycle t, several time criteria have to be used. These time criteria have to be estimated before the well test is executed in order to decide how long there have to be pumped.

The lower limits are easy to estimate and in general these limits are satisfied after some minutes. To determine the upperlimit in the Theis-Jacob method is mostly less easy owing to the difficult way of quantifying the vertical hydraulic resistance of semi-pervious layers.

A constant discharge rate is essential in evaluating well tests with the Theis-Jacob method; using the recovery period of the well test, the consequences of little changes in the discharge rate are mostly met.

The obtained results of transmissivity values from both pumping tests and well tests are in most cases comparable and the well test is in relation to a pump test a cheap and applicable method to obtain a good impression of the transmissivity value of an aquifer.

References

Boswinkel, J.A. 1981. Betrouwbaarheid en nauwkeurigheid van de bepaling van het doorlaatvermogen door middel van putproeven. Dienst Grondwaterverkenning TNO, OS 81-3.

Boulton, N.S. 1963. Analysis of data from nonequilibrium pumping tests allowing for delayed yield from storage. Proc. Inst. Civil Eng. 26, pp. 469-482.

Cooper, H.H. and C.E. Jacob 1946. A generalized graphical method for evaluating formation constants and summarizing well-field history. Trans. Am. Geoph. Union 27, pp. 526-534.

Gun, J.A.M. van der 1979. Schatting van de elastische bergingscoëfficiënt van zandige watervoerende pakketten. Jaarverslag DGV-TNO, pp. 51-60.

Hantush, M.S. 1964. Hydraulics of wells. In: Advances in Hydroscience, vol. 1. V.T. Chow (ed) pp. 281-432.

Hantush, M.S. and C.E. Jacob 1955. Non steady radial flow in an infinite leaky aquifer. Am. Geoph. Un. Trans. 36, pp. 95-100.

Jacob, C.E. 1950. Flow of groundwater. In: Engineering Hydraulics. H. Rouse (ed), pp. 321-386.

Papadopulos, I.S. and H.H. Cooper 1967. Drawdown in a well of large diameter. Water Resources 3, pp. 241-244.

Theis, C.V. 1935. The relation between the lowering of the piezometric surface and the rate and duration of discharge of a well using groundwater storage. Trans. Am. Geoph. Un. 16, pp. 519-524.

APPLICATION OF A WATER BALANCE MODEL IN A HEAVY SALT-AFFECTED SOIL IN SEMI-ARID REGIONS

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Abstract

Attemps are made to scrutinize the available relevant field data, with the aid of a water balance equation to estimate the coefficients describing the hydraulic properties of the water-bearing layers in the Dujailah Project Area, located within the Tigris river alluvial plain (about 200 km south of Baghdad).

The project area is experiencing a severe saline condition, due to an excessive buildups of the water table as a result of infiltration from the fields and augmented by insufficient natural drainage.

The assembled water-levels records, from a network of 194 shallow wells and 14 piezometers, together with other relevant field and laboratory data are utilized in quantifying the hydrogeologic characteristics of aquifers. The results are as follows:

Transmissivity of semi-confined aquifer	= 100.6 m^2/day
Resistance coefficient of the covering semi-pervious layer to vertical flow	= 11842 day
Leakage factor	= 955 m
Storage coefficient of the semi-confined aquifer	= 7.5×10^{-7}

1 Introduction

The Dujaila Project Area, covering an area of 360 km^2, is considered part of the Tigris river alluvial plain and is with altitude ranging between 10.0 m to 15.5 m above mean sea level. Bore holes analysis have indicated the presence of an upper partially silty clay sediment with depth ranging between 13.5 m to 17.0 m that overlies the semi-confined aquifer with depth ranging between 14.0 m to 24.0 m. The later overlies a hard clay bed which is considered an impervious stratum. Because of the worsening drainage conditions, a new reclamation systems are planned in the area to be the premise for a unique and a vital pioneering agricultural-industerial complex.

Reclamation projects in general suffer from unexpected change due to some reasons which are either badly estimated or facts which are not included. To provide an adequate designs of the reclamation works to be undertaken in the project area, and in other areas within the Mesopotamian Plain with similar conditions, this groundwater investigation was initiated with the main objective of using the available data to make a quantitative analysis of the main hydrogeological parameters of the water bearing formations.

2 The water balance equation

The annual groundwater budget of the area could be established in terms of an equivalent water depth of deep percolation losses from the field surface (d), seepage from open water courses (q), leakage from the lower semi-confined aquifer (l_s), recharge from neighbouring areas (l_o), and the change in the subsurface soil water storage (ΔS). The conceptual model describing the functional relationship between the components mentioned above is assumed to take the following form:

$$\Delta S = d + q + l_s + l_o$$

Application of the model necessitatesfirst the finding of the hydrogeologic parameters describing the water bearing formations. Analysing the the field and laboratory data, the following results regarding these parameters were obtained.

2.1 Transmissivity

Analysis of auger hole and bore hole data have indicated the transmissivities of the upper phreatic and the lower semi-confined aquifers are 36 m^2/day and 100 m^2/day, respectively.

2.2 Storage coefficient

According to the graphical procedure established by Soliman (1972), in which the field records of water levels were utilized, the storage coefficient of the semi-confined aquifer is in the order of magnitude of 7.5×10^{-7}.

2.3 Leakage factor

Using the field data gathered from eight bore holes in an equation developed by Hantush (1960), the leakage factor was determined with a value of 955 m.

2.4 Hydraulic resistance

Knowing the saturated thickness and the average hydraulic conductivity (in the vertical direction) of the covering semi-pervious layer the resistance of the layer to vertical flow was determined. The calculated value for the resistance is 11840 days.

3 Assessment of groundwater fluctuations

Analysis of the fluctuation in groundwater levels, observed by the shallow observation wells, have provided an estimate of the annual changes in groundwater storage. The annual change in groundwater level ΔS is envisioned to compose of two parts, as seen in Fig. 1; the first part Δh is the difference between the lowest and highest levels of the hydrograph for the year, while the second part ΔZ is due to the assumption that if no rainfall existed then the recession part of the hydrograph will continue to fall in level. In order to estimate the average yearly value of ΔS , of the whole project area of 360 km^2 similar hydrographs to that shown in Fig.(1) were constructed and the ΔS value for each well was extrapolated. It was found that the average yearly water table fluctuations within the project area is in the order of 0.725 m, which is equivalent to 2.027 mm/day.

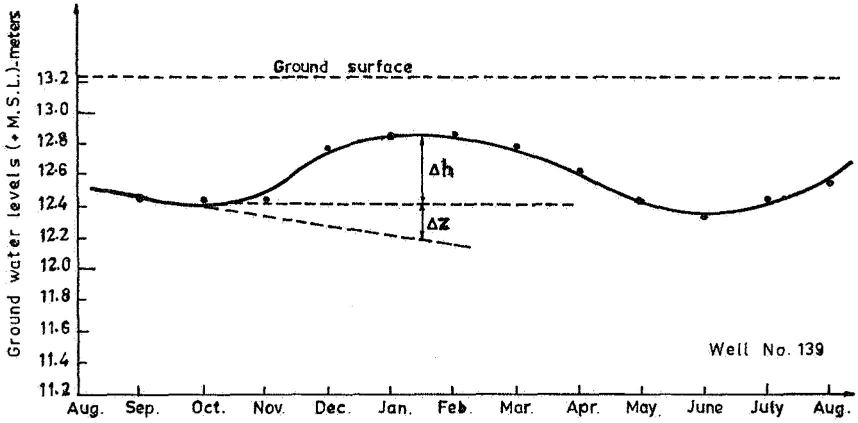

Figure 1 Representative hydrograph showing fluctuations of water table for the period Aug. 1978 to Aug. 1979.

4 Seepage from open-water courses

For the calculation of seepage losses from the Main Dujaila Canal, the only major unlined water course within the project area (with a designed capacity of 24 m^3/s), it is thoughtto be more proper to apply the method developed by Muskat (1937). It is worth noting that conditions required for the applications of Muskat approach are similar to those existed in the project area. The method requires the construction of graphs showing groundwater-level variations in shallow wells located on lines perpendicular to the Main Dujaila Canal . Typical graph is shown in Fig. 2 . The analysis of data has shown that the seepage rate is about 0.135 m^3/day per meter length of canal which is equivalent to a depth of recharge to groundwater of 0.005 mm/day. It appears that this value is negligible as compared to other sources of recharges to ground water.

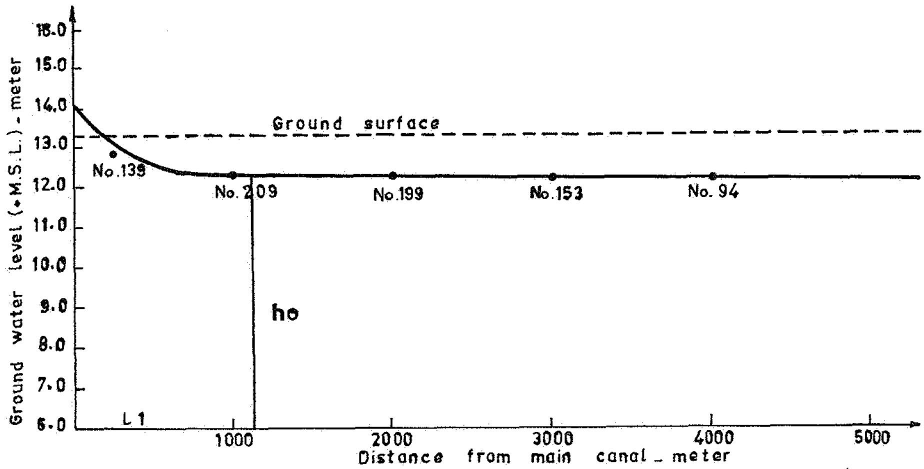

Figure 2 Ground water levels in observation wells along a line perpendicular to main Dujailah canal.

5 Recharge from semi-confined artesian aquifer

The average differential piezometric head, taken as the difference between levels in shallow and piezometric wells, was substituted in Darcy's law to calculate the vertical recharge from the semi-confined aquifer to the upper covering layer. This procedure was followed for all locations of piezometric wells. The results indicate that the recharge based on the average yearly differential piezometric head is about 0.068 mm/day.

6 Deep percolation losses due to irrigation

Recharge to groundwater due to field losses through heavy-textured soils is calculated according to a criteria and elaborate methods of analysis relating potential evapotranspiration, precipitation, surface runoff, leaching requirements and deep percolation, as illustrated in Publication 16 and Lecture Notes of The International Institute For Land Reclamation and Improvement, (1978) and (1975). For this purpose, the soil profile is divided into three zones; namely, the upper rootzone layer of 50 cm thick and with a field capacity of 90 mm, the lower rootzone layer

of 50 cm thick and with a field capacity of 70 mm, and the bottom layer connecting the rootzone region and groundwater of 2 m in thickness and with a field capacity of 90 mm.
By trial and error procedure the water amount of each irrigation, to satisfy a preselected winter-crop and operational leaching requirements, together with the irrigation time intervals, were determined. The results show that the recharge to groundwater due to deep percolation of irrigation water is about 1.93 mm/day.

7 Contribution of groundwater flow from surrounding areas

Applying the afore mentioned groundwater balance equation, the amount of recharge to the project area from adjacent areas could now be determined. The analysis indicate that the amount of this source of recharge to groundwater is in the order of 0.024 mm/day.
It should be mentioned that the values obtained in this study could easily be correlated in order to make a quantitative evaluation of the drainage coefficient, an important parameter needed in the design of the field drainage system in the project area. Such analysis, however, is beyond the scope of this paper.

References

Hantush, M.S., (1960); Modification of the Theory of Leaky Aquifers, Journal of Geophys. Res., Vol. 65, No. 11, pp. 3713-3725.

International Institute For Land Reclamation And Improvement, Wageningen The Netherlands, (1978); Drainage Principles and Applications, Publication 16, Vol. 11, 374 p.

Internation Institute For Land Reclamation And Improvement, Wageningen, The Netherlands, (1975); Lecture Notes of the International Course on Land Drainage.

Muskat, M., (1937); The flow of Homogeneous Fluids Through Porous Media, McGraw-Hill Book Comp., New York, 763 p.

Soliman, M.M., (1972); Determination of Some Hydrological Factors With a Well Recorder. Proceedings, Second Internation Symposium in Hydrology, Fort Collins, Colo., U.S.A., pp. 74-79.

BASIC HYDROGEOLOGICAL DATA FOR THE INVESTIGATION OF GROUNDWATER SYSTEMS IN HARD ROCK AREAS IN SWEDEN

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Abstract

Groundwater investigations in Sweden has formerly mainly been focused on the quaternary deposits. They generally have both higher storativity and higher permeability compared to the hard rocks, the crystalline basement. However. the last decade has produced new hydrogeological tasks, with demands of deeper knowledge about the basement. The main objectives with these investigations are; site characterization for underground structures, and method development, site investigation and groundwater characterization for nuclear waste disposal. For both these objectives, the water-bearing ability of the rock mass is of crucial importance.

1 Introduction

Geologically, the main part of Sweden is characterized by the Precambrian crystalline rocks included in the Baltic Shield. In addition to the crystalline basement, minor areas with Cambrian or younger rocks also exist. The Caledonean Mountain Range, which is composed mainly of metamorphic, Cambro-silurian rocks is located in the western part of Sweden. In the southernmost part of the country, the bedrock is composed of sedimentary rocks, being a part of the European sedimentary basin. Some minor areas of the Baltic Shield are covered by thin remnants of Cambro-silurian rocks (Persson et al 1979).

The bedrock is generally covered with loose Quaternary deposits, mainly till, which occupy as much as 75 per cent of the area. Clay and sand/gravel formations also exist, the latter mainly as eskers. The glacial drift is generally shallow, on average about 5 - 10 m thick.

About 50 per cent of the water supply in Sweden is obtained from groundwater. The absence of extensive sedimentary reservoirs limits the possibilities for groundwater extraction, and the water supplies for the largest cities (population >250 000) are all based on surface water. However, the groundwater supply dominates for all other cities and municipalities. In this respect, the Quaternary eskers are the most important aquifers. Water extraction from the till cover and from the crystalline basement is most common for small villages and for single households. The importence of the latter is illustrated by the intensive drilling of wells, 6000 - 8000 new wells are completed each year.

Investigations of the groundwater resources for water supply are focused on the major aquifers, i.e. the eskers, while little or no effort is devoted to the investigation of the crystalline rocks. However, extensive hydrogeological investigation programmes focused on the basement have been carried out during the last decade. The reason for these programmes was not water supply, but engineering geological purposes and pollution transport minimization, although these investigations resulted in valuable information as regards the groundwater conditions in the low-conductivity, crystalline basement.

The aim of this paper is to review the investigations in the crystalline rocks, the development and possible future trends.

2 Investigation of hard rocks in Sweden

The investigations of the crystalline rocks are made with two main objectives:

- Site investigations for underground structures
- Site investigations, general characterization and method development

for disposal of nuclear waste or spent fuel in crystalline rocks

For both these objectives, the water-bearing ability of the rock mass is of crucial importance. The groundwater conditions are very important for the economy, operation and construction of underground plants. In the case of nuclear disposal, the groundwater will be the transporting medium for any radioactive nuclides (Olsson 1979, Carlsson et al. 1982)

As regards the water available in the low-conductivity, crystalline basement, new geophysical techniques have been adopted to detect major water-bearing zones (Müllern 1980). An increased use of these techniques has produced wells with high capacities.

The investigations carried out in the crystalline basement have been mainly focused on the determination of various hydraulic properties, such as:

- hydraulic conductivity
- storage coefficient
- kinematic porosity
- sorbtion

These have been carried out to obtain a characterization of the rock mass as regards its water-bearing and transporting properties, and to collect input data for continued modelling.

The water-bearing ability of a crystalline rock mass is governed by the fracturing, where the major flow paths consist of larger zones of fractured or crushed rock. These zones usually govern the flow pattern in a large area, acting as high conductive drains through the rock mass. In the more homogeneous rock mass, minor fracture zones and single fractures are of importance, while the rock matrix may be regarded as almost impervious, from a practical point of view (Carlsson and Olsson 1981).

This great variety in hydraulic conductivity, from the highly conductive highly fractured zones to the low-conductivity, unfractured rock matrix, calls for specially developed hydraulic tests. Ordinary pumping tests are only suitable for investigations in highly conductive formations because they become too time consuming in a moderately fractured rock mass.

At present the following are the tests most frequently used:

- Packer tests
  - water injection tests
  - pulse tests
  - interference tests
  - drainage tests
- Tracer tests

The different techniques cover different ranges of hydraulic conductivity (Carlsson and Olsson 1979). The pulse test makes it possible to test rock masses with very low conductivity, but is not suitable for more conductive rock masses. The detection limit of the water injection test is not as low as that of the pulse test, but on the other hand, this technique is suitable for highly conductive zones. Multiple hole tests, such as interference tests and tracer tests, are used for testing over long distances, along fractured zones or interconnected single fractures (Carlsson et al. 1983, Carlsson et al. 1979).

The evaluation techniques vary, depending on the purpose of the test. Simple water injection tests evaluated according to stationary theories are used for engineering geological purposes. For tests in the nuclear waste disposal programme, the evaluation is normally based on transient theories. These more accurate theories have lately also been adopted for engineering geological puposes.

The instrumentation for hydraulic tests in crystalline rocks, has been continously developed and improved, and at present, most tests are made with computerized registration and regulation devices. Most investigations are made in narrow boreholes (46, 56 or 76 mm diameter) at great depths (at most 800 - 900 m), which has made it necessary to obtain specially designed down-the-hole instruments as well as equipment for controlling and handling the probes (Jacobsson and Norlander 1981).

3 Conclusions

During the last decade, the determination of the hydraulic properties of the crystalline basement have become one of the main tasks of hydrogeological research in Sweden. Great efforts have been made to determine the hydraulic characterization of the rock mass, and the development and improvment of the investigation techniques and instruments has been a main goal. This trend will probably also continue in the future.

The general tendency is for control, data collection and evaluation to be computerized to a great extent. The increased knowledge of the groundwater conditions in fractured rock masses will result in groundwater models which are more suitable for simulation of flow in fractures than existing models.

References

Carlsson, A. and T. Olsson, 1981: Hydraulic properties of a fractured granitic rock mass at Forsmark, Sweden. Sver. Geol. Unders., C 783, 67 pp. Uppsala.

Carlsson, H., L. Carlsson, A. Jämtlid, H. Norlander, O. Olsson and T. Olsson, 1983: Cross-hole techniques in a deep-seated rock mass. In press.

Carlsson, L., G. Gidlund, K. Hansson and C. Klockars, 1979: Estimation of hydraulic conductivity in Swedish precambrian crystalline bedrock. Proceedings of Workshop on Low-flow, Low-permeability Measurements in Largely Impermeable Rocks, OECD-NEA, pp.97 - 114. Paris.

Carlsson, L., H. Norlander and T. Olsson, 1982: Hydrogeological investigations in boreholes. In press, OECD-NEA, Paris.

Carlsson, L. and T. Olsson, 1979: ALMA-Hydrauliska barriäregenskaper hos marknära bergmassor. National Council for Radioactive Waste, Technical report 1.24, 83 pp. Stockholm

Jacobsson, J. and H. Norlander, 1981: Equipment for hydraulic testing. Stripa Project Report 81-04, 13 pp. Stockholm.

Müllern, C-F., 1980: Airborne geophysical measurement used for hydrogeological mapping. Nordic Hydrological Conference, UNGI Report No 53, pp. 135 - 142. Uppsala.

Olsson, T., 1979: Hydraulic properties and groundwater balance in a soil-rock aquifer system. Striae, Vol. 12, 72 pp. Uppsala.

Persson, G. and J. De Geer, 1979: Explanatory Notes for the International Hydrogeological Map of Europe, Sheet C3 Oslo — Swedish part. Bundesanstalt für Geowissenschaften und Rohstoffe and UNESCO, 47 pp. Hannover and Paris.

DETECTION EN CONTINU DES PARAMETRES PHYSICO-CHIMIQUES DE L'EAU (TEMPERATURE, CONDUCTIBILITE, SODIUM) DANS LES SOURCES KARSTIQUES DU JURA SUISSE. TECHNOLOGIE DES MESURES ET INTERPRETATIONS HYDROGEOLOGIQUES.

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Abstract

During many years, the continuous detection of the temperature ($\pm$0.1°C), the electric conductivity ($\pm$3 μS/cm) and the activity of the sodium ion ($\pm$0.1 mg/l) is carried out by means of adapted sensors at the outlet of four karstic basins in the Swiss Jura.

In comparison with the hydrograph of springs, the fluctuations of these parameters permit to consider them as natural tracers and indicators of the different components of the karstic water-bearing rocks discharge. These continuous recordings allow to distinguish time-limited events, who should not be perceptible by an horary sampling.

Important results obtained by these studies increase our knowledge about:

- the rising mecanism of the karstic spring
- the transfert speed in the very permeable karst-net (fracture permeability)
- the rapid dry up of the surface infiltration
- the alimentation of the outlet from the low permeability "blocks" (primary permeability).

Certain artificial tracers and anthropogenuous pollutions are displayed with the help of adapted ion-selective electrodes.

Résumé

Durant plusieurs années, à l'exutoire de quatre bassins karstiques du Jura suisse, la détection en continu de la température (±0.1°C), de la conductibilité électrique (±3 µS/cm) et de l'activité de l'ion sodium (±0.1 mg/l) est réalisée à l'aide de sondes appropriées.

Comparées à l'hydrogramme des sources, les fluctuations de ces paramètres permettent de les considérer comme traceurs naturels et comme indicateurs des différentes composantes de l'écoulement des aquifères karstiques.
Ces enregistrements en continu permettent de cerner des événements limités dans le temps qui ne seraient pas perceptibles par un échantillonnage horaire.
Des résultats importants sont ainsi obtenus sur :
- le mécanisme de crue des sources karstiques
- la vitesse de transfert dans le réseau très perméable (perméabilité de fractures)
- le tarissement de l'infiltration rapide provenant de la surface
- l'alimentation de l'exutoire à partir des "blocs" peu perméables (perméabilité primaire)

Certains traceurs artificiels et pollutions anthropogènes sont aussi mis en évidence à l'aide d'électrodes sélectives appropriées.

1 Introduction

Le Centre d'Hydrogéologie de l'Université de Neuchâtel (Suisse) s'emploie depuis plusieurs années à développer les méthodes d'acquisition de données en continu. Les paramètres physico-chimiques enregistrés (conductibilité électrique, température, activité de l'ion sodium) servent à étudier le fonctionnement des aquifères karstiques jurassiens en relation avec l'hydrodynamique.

La figure 1 illustre la situation géographique des principales sources équipées, la durée d'enregistrements variant de 1 à 4 ans.

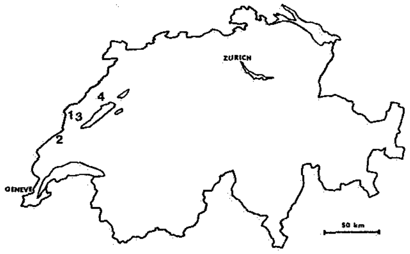

Figure 1
Carte schématique de la Suisse et position des sources karstiques équipées d'enregistreurs en continu.
1: Areuse, 2: Orbe, 3: Ubena, 4: Fontainemelon.

2 Matériels et méthodes utilisés

Les paramètres conductibilité, température, sodium sont choisis en raison de leur représentativité comme traceurs naturels des types d'écoulement.
Dans les eaux karstiques, la conductibilité traduit la minéralisation totale tandis que le sodium indique l'évolution de la phase non-carbonatée. La température, par contre, est un paramètre physique indépendant des variations chimiques.

Sondes utilisées:
température: thermistance Pt 100 ohms, précision ±0.1°C, plage de mesure 2 à 12 °C, reliée directement à l'enregistreur par un conducteur 3 fils
conductibilité: cellule WTW et Ingold, précision ±3 µS/cm, plage de mesure 0 à 333 µS/cm
sodium: électrode spécifique de verre Ingold et électrode de référence Tacussel, précision ±0.1 mg/l, plage de mesure 0.1 à 10 mg/l. La mesure de l'activité de l'ion sodium se fait par voie électrochimique basée sur la loi de Nernst.

Appareils de mesure:
conductibilité: conductimètre WTW modèle LF 56
sodium: ionmètre Orion modèle 407A.

Enregistreur:
Eurotherm Chessel type 301 à 3 canaux en continu, avancement du papier 1 cm/h.

Alimentation:
si l'alimentation 220 V n'est pas disponible, ce matériel est conçu pour travailler de façon autonome sur des batteries rechargeables, éventuellement à l'énergie solaire.

3 Difficultés rencontrées pendant l'acquisition des données

Les chaînes d'enregistrement sont placées dans des environnements différents, elles sont ainsi soumises à des contraintes d'humidité, de température et d'ensoleillement.
Un des principaux problèmes provient des distances entre les sondes et les appareils de mesure. Dans le cas de l'Orbe, toutes les sondes ont pu être placées 12 mètres au-dessous des enregistreurs sur un flotteur qui suit les fluctuations du niveau de la rivière souterraine.
Pour les sources de l'Areuse et de l'Ubena, seules les sondes de température étaient plongées à l'émergence, tandis que le reste de l'installation était alimenté artificiellement à plus de 300 mètres en aval.
Ces variétés d'emplacement ont causé les difficultés suivantes:

- fluctuations journalières parasites de la température de l'eau
- dérive des amplificateurs des appareils de mesure dues aux fortes variations de la température extérieure
- encroûtement des sondes
- interruption de l'alimentation des conduites d'amenée d'eau.

4 Fonctionnement des aquifères karstiques

La circulation dans les aquifères karstiques dépend de deux types de perméabilité. Des fractures et des chenaux très perméables et organisés découpent et drainent des masses calcaires formant des "blocs" peu perméables. Ces deux types de circulation se répercutent non seulement sur l'hydrogramme mais aussi sur les paramètres physico-chimiques des eaux.
Les enregistrements en continu permettent de détecter les différentes composantes de l'écoulement qui alimentent les exutoires.

5 Résultats et interprétations

Comme exemple, nous présentons ici les enregistrements en continu des paramètres météorologiques, hydrologiques et physico-chimiques de deux grandes sources karstiques.

La figure 2 donne la variabilité annuelle à la source de l'Areuse (Müller et Zötl 1980). On remarque que les fluctuations physico-chimiques suivent celles du débit . Les dilutions constatées correspondent à l'arrivée de l'eau fraîchement infiltrée à l'exutoire. Néanmoins, ces variations sont relativement atténuées par l'existence d'un volume d'eau considérable dans la zone non-écoulable se trouvant au-dessous du niveau de l'exutoire.

La figure 3 apporte plus de détails sur le fonctionnement de ce même aquifère. L'enregistrement correspond à la période d'injection des traceurs du 4ième Symposium sur l'Utilisation des Traceurs en Hydrologie (4ième SUWT).

L'arrivée massive du sodium est détectée à la source de manière très précise par les paramètres chimiques. La crue suivante chasse encore des masses d'eau salée qui étaient stockées dans l'aquifère.

Le 12 juin, un orage de grêle exceptionnel met en évidence l'arrivée des eaux nouvellement infiltrées, peu minéralisées, froides, retardées de 24 heures par rapport à la crue. Cet évènement permet également d'estimer les vitesses d'écoulement dans les chenaux karstiques.

La figure 4 illustre la variation annuelle à la source de l'Orbe. Les réponses des paramètres reflètent l'interférence de deux lac (10 km2) sur l'alimentation typique d'un aquifère karstique.

L'étiage est caractérisé par une alimentation préférentielle des lacs par l'intermédiaire de fractures et de chenaux très perméables. L'influence des eaux fraîchement infiltrées ne se fait pas sentir par une dilution à la source, mais par une augmentation de la minéralisation carbonatée due à la mise en charge de l'aquifère qui provoque un drainage des masses calcaires peu perméables.

La figure 5 décrit, à la même source, un phénomène particulier de courte durée. Lors de très fortes crues, les paramètres enregistrés indiquent une brusque modification de l'alimentation. Des siphons, situés dans la zone de battement de l'aquifère, reconnus par les recherches spéléologiques, sont vidangés.

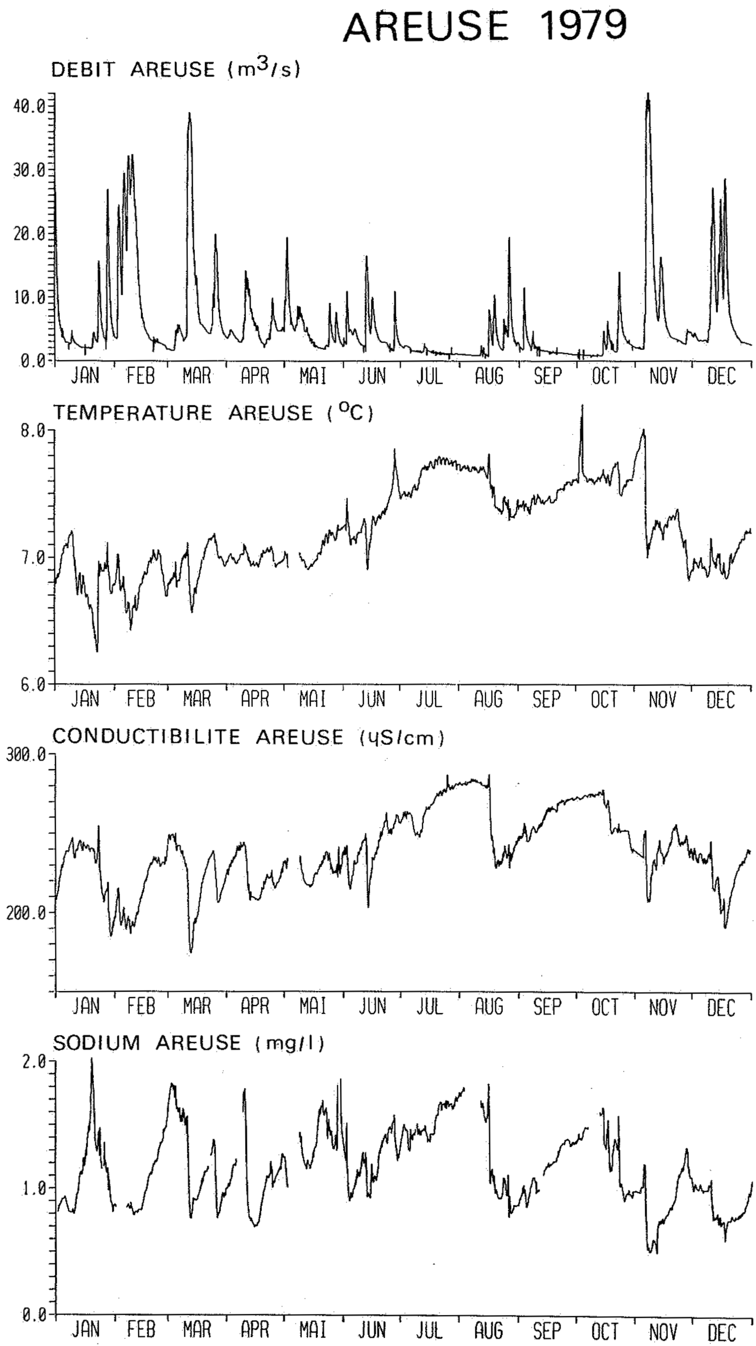

Figure 2. Enregistrements en continu à la source de l'Areuse durant l'année 1979

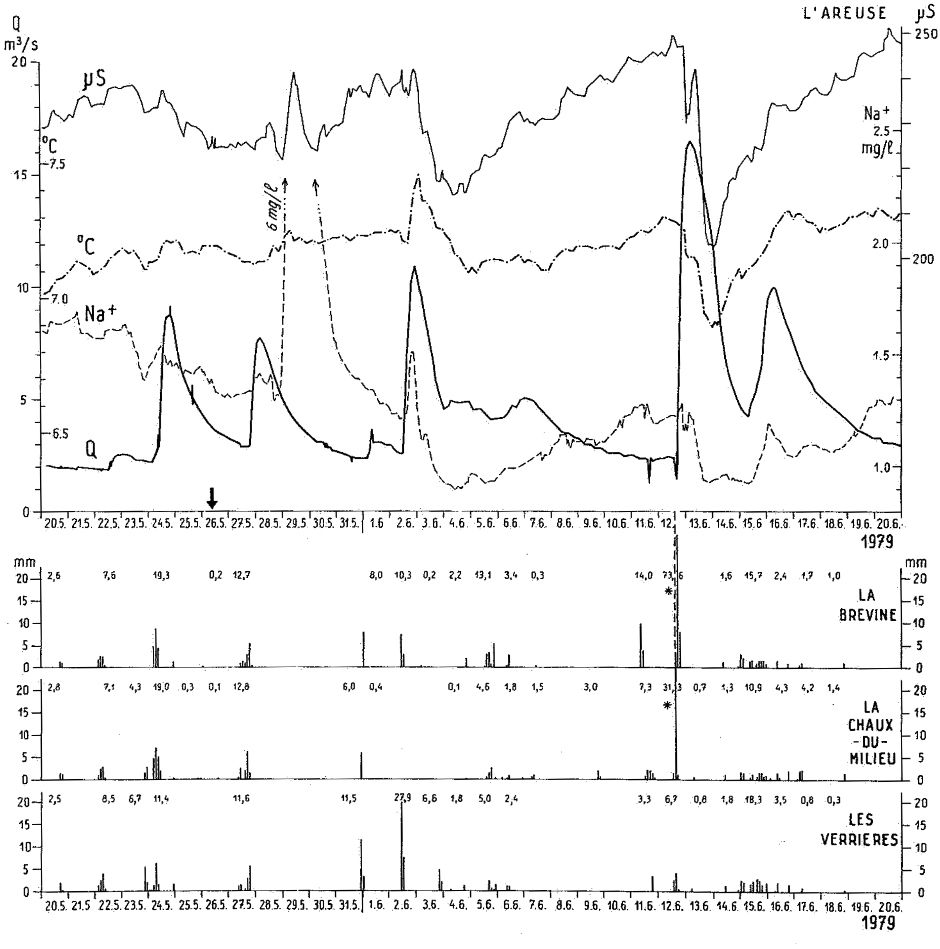

Figure 3 Synthèse des informations hydrologiques, météorologiques et de la variabilité des paramètres physico-chimiques (enregistrement en continu) à la source de l'Areuse, du 20 mai au 20 juin 1979

Flèche = injection des traceurs du 4ième SUWT

Etoile = orage de grêle

La Brévine, La Chaux-du-Milieu, Les Verrières = station pluviographique avec cumul de 2 heures des précipitations (mm) et indication chiffrée des totaux journaliers.

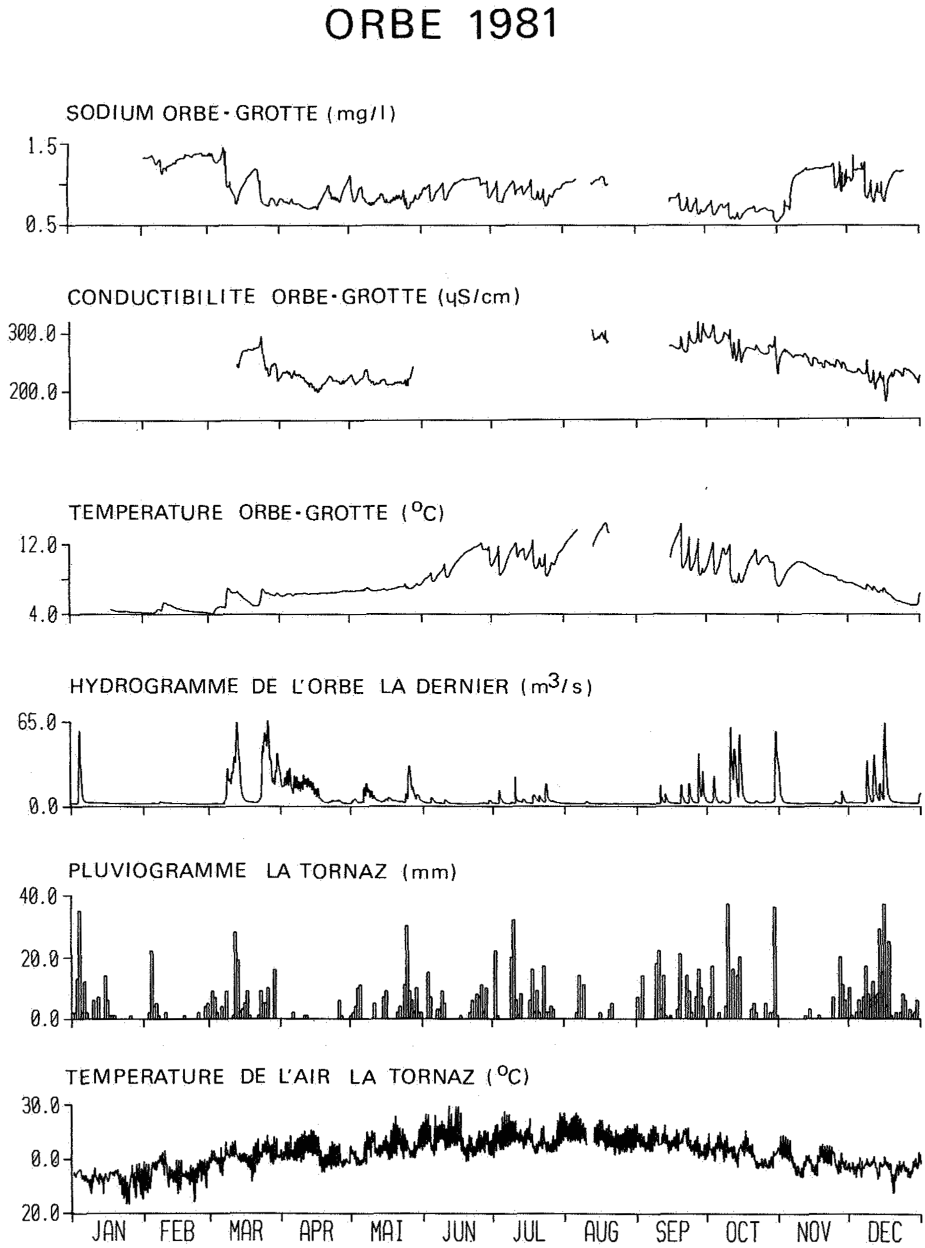

Figure 4 Enregistrements en continu dans la grotte de l'Orbe et hydrométéorologique de la région des lacs pendant l'année 1979

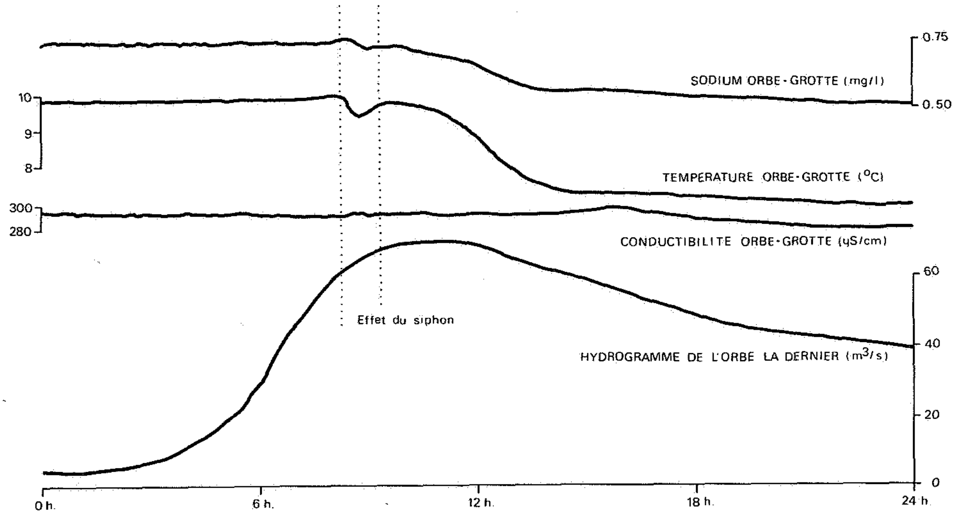

Figure 5 Enregistrement en continu journalier lors d'une grande crue à la source de l'Orbe. La vidange de siphons suspendus est mise en évidence pendant environ 60 minutes

6 Conclusions

Vu les difficultés d'acquisition des données sur le terrain, les enregistrements en continu ne peuvent pas fournir des valeurs absolues comparables aux analyses de laboratoire. D'après nos expériences, il est cependant préférable de posséder des enregistrements en continu à valeurs relatives qui peuvent être facilement calibrées grâce à des prélèvements manuels. Ces enregistrements apparaissent alors indispensables pour la compréhension du fonctionnement des aquifères karstiques car l'échantillonnage ponctuel,même très serré, n'arrive pas à restituer en détail les rapides variations des différents paramètres.

Remerciements

Le matériel qui a permis ces études est financé par le Fonds National Suisse de la Recherche Scientifique, dans le cadre des projets

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Références

Bürger, A. 1959. Hydrogéologie du bassin de l'Areuse. Bull. Soc. neuch. Géogr., 52/11(1):5-304.

Kiraly, L. et I. Müller 1979. Hétérogénéité de la perméabilité et de l'alimentation dans le karst: effet sur les variations du chimisme des sources karstiques. Bull. Centre d'Hydrogéol. Neuchâtel, 3: 237-285.

Müller, I., U. Schotterer et U. Siegenthaler 1982. Etude des caractéristiques structurales et hydrodynamiques des aquifères karstiques par leurs réponses naturelles et provoquées. Eclogae geol. helv., 75/1: 65-75.

Müller, I. et J.G. Zötl (eds.) 1980. Kartshydrologische Untersuchungen mit natürlichen und künstlichen Tracern im Neuenburger Jura (Schweiz). Steir. Beitr. Hydrogeol., 32: 5-100.

Tripet, J.-P. 1973. Etude hydrogéologique du bassin de la source de l'Areuse (Jura neuchâtelois). Mat.Carte géol. Suisse, 21:1-183.

Wacker, C. et C. Wittwer 1982. Approche hydrogéologique de la résurgence de l'Orbe (Vallorbe,Suisse). Actes 7ième Congrès nat. spéleol. suisse, Schwyz. à paraître.

TRITIUM AS A GROUNDWATER
TRACER IN ZIMBABWE

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Abstract

The disposition and flow rate of groundwater in the Sabi Valley Alluvial Plain in Zimbabwe was elucidated by use of serial measurements of tritium over a 15 year period. The Sabi Valley alluvial plain is the largest tract of alluvium in Zimbabwe. The tritium study revealed:

a) that the Sabi river contributes only minor recharge to the main groundwater basin.
b) that the flow rates through the perched Aquifer was higher than through the major and deeper aquifer, and
c) that the flow rate computed from the tritium study was in good agreement with that calculated by orthodox hydrogological techniques.

1 Introduction

Tritium is a radioactive isotope of hydrogen, naturally produced in the atmosphere by cosmic rays and present in nuclear bomb fall-out. Oxydised to water, it reaches the earth in minute concentrations in rain:

5 - 20 T.U. (1 T.U. = 3.2 pci 1^{-1} water).

On infiltration into the ground, the rainwater becomes to a greater or lesser extent isolated from the atmospheric source and the concentration drops according to the characteristic half-life of tritium (12.3 years). The tritium concentration therefore becomes a measure of the residence time of groundwater since the time of recharge or of the recharge/storage

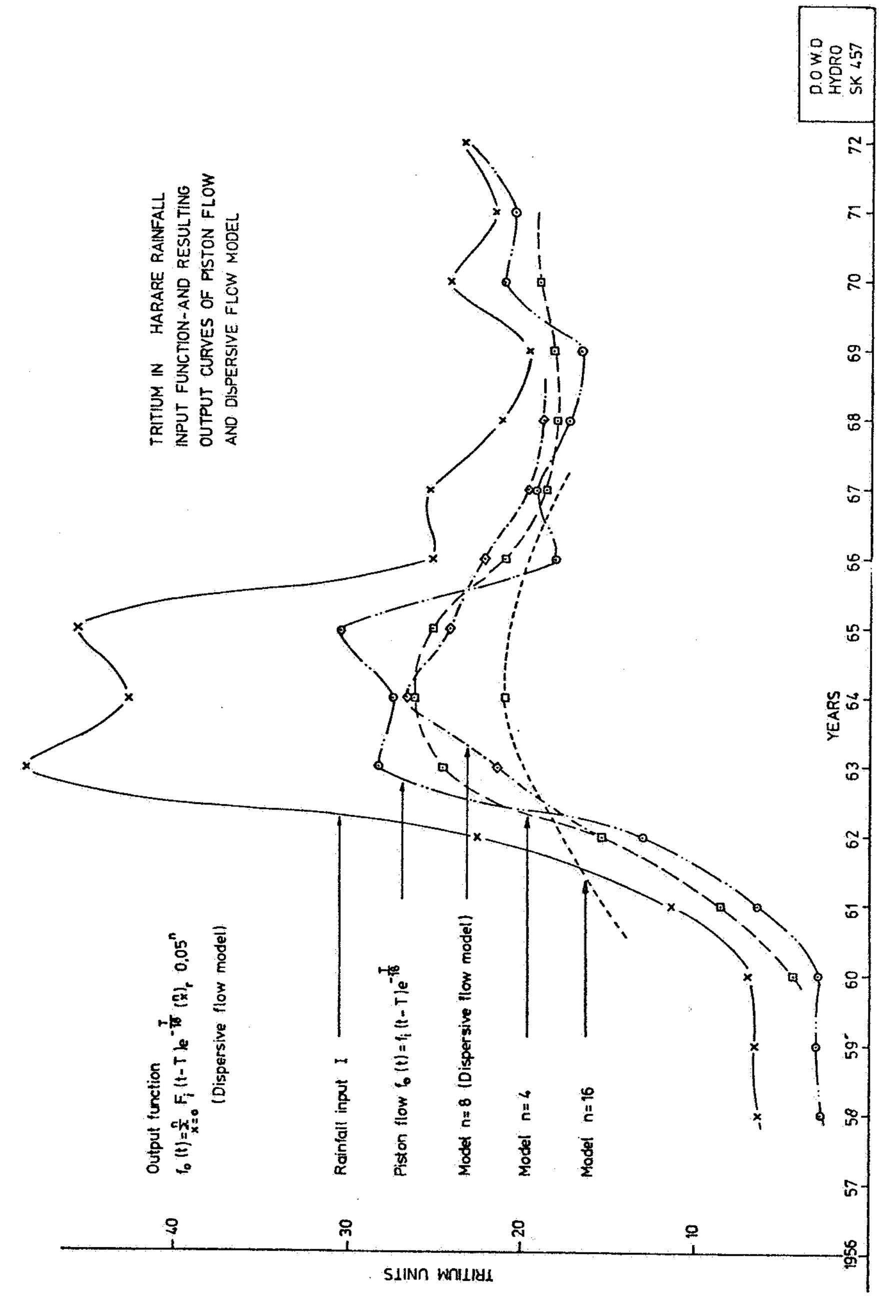

Figure 1 Tritium values in Harara rainfall. Also shown are different tritium output curves

ratio.

Routine measurement techniques allow for the detection of tritium down to about 0.5 T.U. which represents an effective "age" or residence time of about 50 years. The increase in concentration due to bomb fall-out over the last two decades allows for even finer time resolution. In Figure 1 the northern hemisphere hydrogen bomb fall out values in Harare rainfall are seen. These tritium values if subsequently detected in bore-holes that have been serially sampled will allow an accurate time reference for rainfall recharge into the ground.

2 Method of Measurement

Single stage electrolysis and subsequent gas counting is used for the analysis of tritium. The full details of the analytical procedure are given in Wurzel and Ward (1968).

3 Geology of Sabi Valley alluvial plain

The Sabi Valley alluvial plain situated in the extreme South East of Zimbabwe constitutes the largest tract of alluvium in Zimbabwe. (Figure 2). It is bounded by latitudes 20^{o} 05'S and 20^{o} 35'S and longitudes 32^{o} 10'E and 32^{o} 25'E, situated in the lowveld, and having an elevation above sea level between 400 and 500 metres. The climate is semi-arid, precipitation being confined to five months of the year and amounting to an annual mean of 455 mm. The alluvial plain has only gentle relief and is approximately 20 km wide by 50 km long. The Sabi river, the only perennial stream in the area, maintains a remarkably straight course through the centre of the plain in a braided channel which is confined within banks generally 1 km to 1.5 km apart and about 5 metres above the stream bed. The alluvium rests on essentially lower Karoo rocks (Permian sediments) and is confined on the east by the Umkondo System (precambrian rocks where a large fault plain divides the Umkondo rocks which form a massif on the east, with the underlying Karoo rocks on the plain), and on the West of the plain, by a gently eastwards sloping granite area. The alluvium is characteristically sand (varying from coarse to fine) silt and clay. Clay horisons and lenses

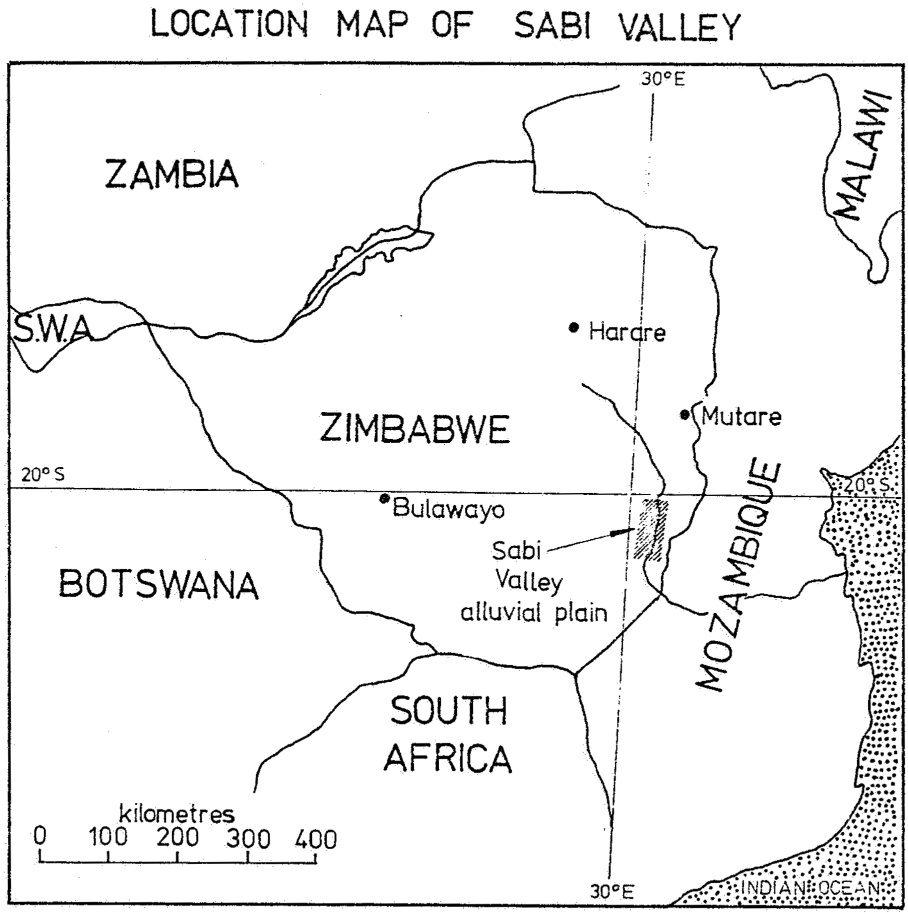

Figure 2 Location map of Sabi VAlley

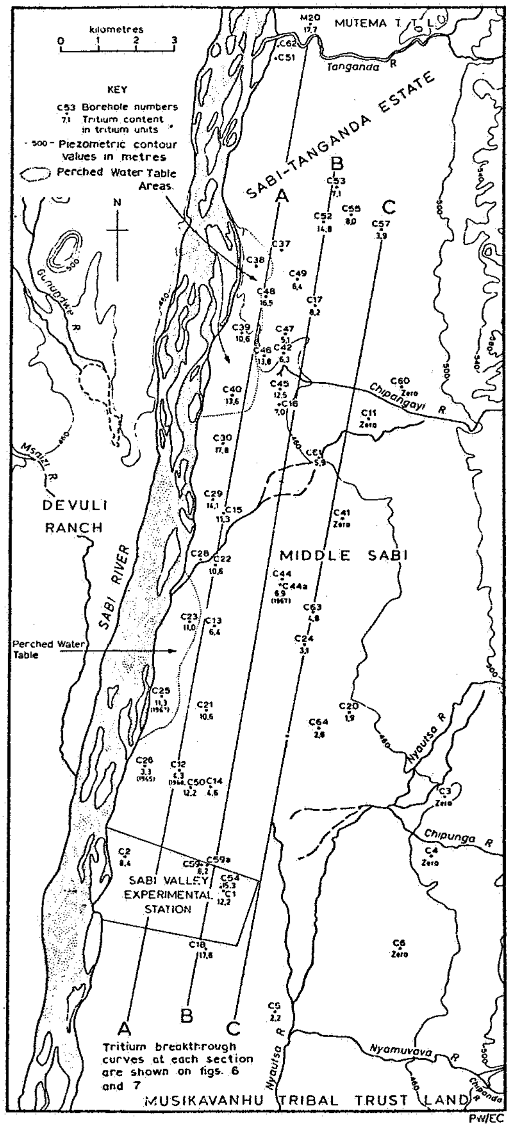

Figure 3 Sabi Valley - tritium content in boreholes in 1972. Also shown are Sections A, B, and C, for which breakthrough curves are shown in figures 4 and 5.

are universally present at the surface and the aquifer is confined. The alluvium varies in thickness from 30 - 40 metres near the Sabi river to 120 metres at the eastern boundary of the plain.
The existence of a groundwater reservoir within the alluvium had been known for several years; however, for development purposes it was essential to know whether the groundwater was "on the move" and being recharged and hence an early computation of a reasonable safe yield figure. The geohydrological investigations took place solely on the eastern half of the plain (east of the Sabi river); access to the western half was totally impossible.

4 The tritium results

The first samples of water for tritium analysis were taken in 1965. The open type of sampler was used to sample the boreholes. This was because it was important to determine whether horizontal stratification existed and also several of the boreholes did not have pumps mounted. Open samplers consist of a metal cylinder or tube with rubber stoppers or flap valves at both ends. In operation, the valves are held open by a spring and trigger mechanism. The sampler is lowered into the borehole; when the required sampling depth is reached a weighted messenger is dropped down the sampler line, strikes the trigger and closes the valves, thereby enclosing the water sample in the sampler tube. Most boreholes showed uniform tritium concentrations in the vertical profile, indicating that the boreholes act as short circuit conduits between aquifers.
It was evident early on that generally tritium values were low and decreased in an easterly direction concommitant with an easterly flow direction. In only one borehole were time dependent samples available (1965 and 1967), the results of which were correlated by Wurzel and Ward (1968) to the southern hemisphere wide rise in tritium content commencing in 1959. In the case of the other boreholes, Wurzel and Ward (1968) used the piston-model to deduce a filtration velocity, (the macroscopic or apparent velocity in contradistinction to the microscopic or interstitial velocity) Section A, Figure 3 of 0.3 - 0.6 metres/day. To compute the filtration velocity an effective porosity value had to be

assumed and a 20% value was considered to be realistic.
Since 1969 when the first phase of the investigation was completed, new data has accumulated, time dependent tritium samples are now available spanning 8 years and more newly drilled boreholes have been sampled. Further, in 1973, several radioisotope porosity determinations were carried out in the field which confirmed that the effective porosity estimate of 20% for the Sabi Sands was correct, (Wurzel 1974). Sampling ceased with the advent of war in Rhodesia.
The more recent tritium results are shown in Figures 3, 4 and 5. Overall higher values occur in 1972 than in 1967, and boreholes in mid-plain that contained zero tritium in 1965, show positive tritium values in 1972. Time dependent values are plotted in Figures 4 and 5 and the input values (Harare rainfall) are shown in Figure 1. The eastern bank of the Sabi Valley was arbitrarily divided into 3 sections and tritium values plotted for boreholes lying on or near each section. Thus each point on Figures 4 & 5 represents one borehole lying on or near each section being sampled in a particular year. As an example for boreholes lying on Section A: in 1965 eight boreholes were sampled, no sampling took place in 1966, ten boreholes were sampled in 1968 and so on. Boreholes (C38, C39 and C40) which have high water levels, and in which downward vertical currents have been measured with artificial tracers, reveal interesting tritium pulse curves. These boreholes lie in an area where two well defined aquifers exist, an upper perched aquifer and the lower and major groundwater reservoir. In two of these boreholes the complete tritium pulse curve is evident, from which we interpret that the perched aquifer is almost entirely fed from the Sabi river, that the throughflow velocity in the perched aquifer is less than in the major aquifer and indeed re-enters the river downstream, or that further to the east, a connection exists between the two aquifers. Indeed, it is likely the upper aquifer merges with the main water bearing body.
Such well defined break-through curves lend themselves to mathematical analysis. In interpreting tritium data, it is possible to use models to derive flow; the three most widely used are the piston-flow model, the exponential well mixed reservoir model and the dispersion mode. The dispersion model is one in which confined conditions are implicit and recharge which occurs in a given period mixes with water recharged in previous and subsequent years. It has been shown that a model in which

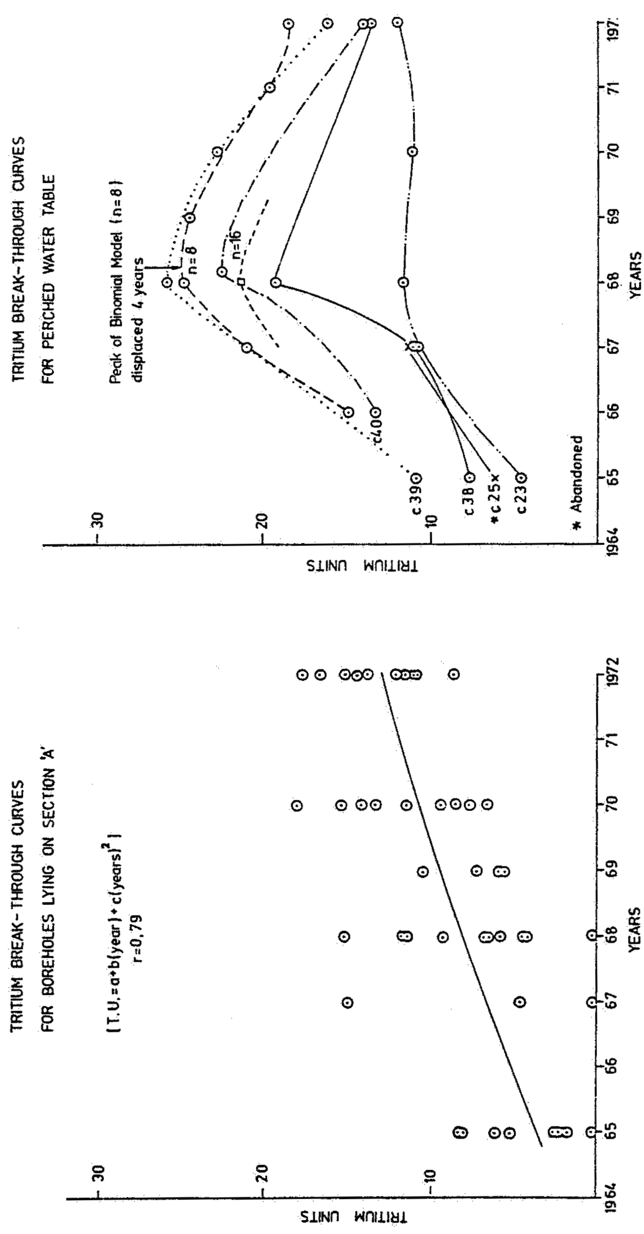

Figure 4 Breakthrough curves for section A and for perched water table.

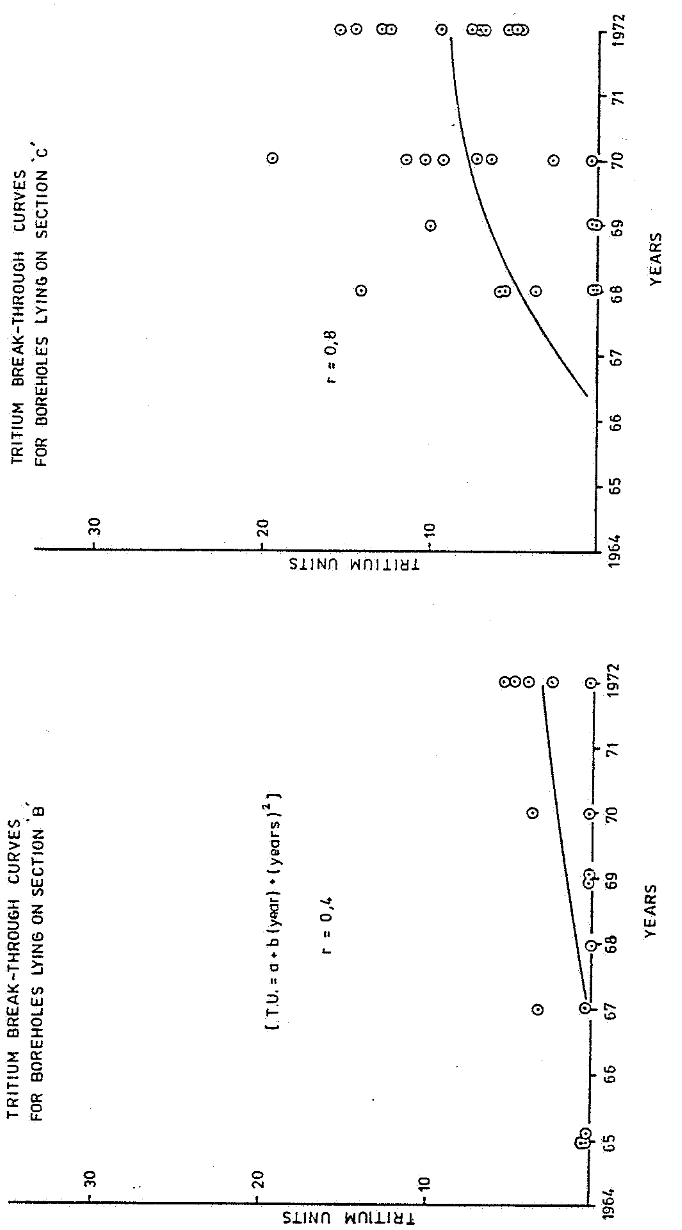

Figure 5 Breakthrough curves for Sections B and C

the age distribution of the samples collected at a distant point from the recharge is either normally distributed (Halevy and Nir 1962) or is gaussian. For simplicity of computation, a binomial model was used in this study.

A binomial model can be fitted with a variance of two displaced 4 years from the input date. An adequate fit for $\delta^2 = 2$ is obtained for boreholes C 40 and C 39. Indeed, the goodness of fit is startling. An apparent flow velocity in the upper aquifers of 0.4 metres/day is thus obtained. In the major aquifer the time dependent results are shown in Figures 4 and 5. The trend is eminently visible with bomb tritium steadily making its way eastward. The scatter of results is large and is to be expected. It is highly probable that boreholes tapping the main aquifer, but adjacent to the perched aquifer are receiving younger perched aquifer water, and this blend of water is reflected in the tritium values. Assuming a rather diffuse recharge area 4 km west of the Sabi river (a valid assumption; ephermal streams rising on the granite catchment which forms the western boundary to the plain, debauch into the alluvial plain), and making a further assumption that the tritium in rainfall input peak would reach Section A in 1975 a flow velocity of 1.4 metres/day is computed.

It is of interest to look at the variance of the pulse output curves in the perched aquifer. The variance in the binomial-model can be related to the dispersion coefficient, thus:

$$Dm = \frac{\bar{x}}{T^2} \delta_T^2$$

where

$\bar{x}$ is the mean distance between the recharge area and sampling point,

T is the transit time and

Dm is the dispersion coefficient.

A value for Dm of $0.4 m^2$/day was obtained. The identical calculation for the main aquifer using a δ^2 value of 4 yields a value for Dm of $0.9 m^2$/day.

Comparing the tritium results with the chemical analysis of the waters of the Sabi Valley is also of interest. There appears to be a broad correlation between water showing zero tritium values and water having high mineralisation content. This is to be expected. A high salinity content

'is generally taken to indicate that either the water has travelled a long way or that it has been underground for some considerable time.
No correlation exists between tritium content and salinity values if these values are below about 500 ppm. Also, zero values of tritium were obtained in boreholes with low salinity content. This is well illustrated by the table below.

Borehole No.	Total dissolved solids. ppm.	Tritium Units T. U.
C 21	225	Zero
C 22	108	Zero
C 23	186, 180 ('65, '67)	5.6, 14.1 ('65, '67)
C 29	132	2.0
C 39	138	14.0
C 41	360	Zero
C 42	186	3.9
P 11	480	Zero
P 12	480	Zero
R 25	1200	Zero

The variation in tritium content is seen to be high, while the salinity content varies by a factor 4 (excluding R 25). On salinity considerations alone, boreholes C 21 and C 22 would have been assumed to be in an area of fairly strong flow. But, as shown by the tritium content, this is not the case. Furthermore the concept of high salinity water being equated with length of travel time or time spent underground and vice versa is shown not to hold in this particular case.
No temperature differences were detected in 6 boreholes where temperature was measured in the vertical profile.

5 Conclusions

Classical tritium breakthrough curves in the Sabi Valley were obtained which revealed mass movement of water in the alluvial plain. The tritium study has shown that the Savi river does not contribute significantly to groundwater recharge. It is of interest to note that despite a much lower peak 1963 tritium in rainfall value in the Southern Hemisphere (2 orders of magnitude less than the Northern Hemisphere) the peak was still discernible in the groundwater reservoir.

References

Halevy, E. and Nir, A. 1962. The Determination of Aquifer parameters with the aid of radioactive tracers. J. Geophys. Res., pp. 2403-9.

Wurzel, P. and Ward, P.R.B. 1968. Groundwater Studies in the Sabi Valley, Rhodesia, using natural tritium measurements. J. Hydrology 8, pp. 48-58.

Wurzel, P. 1974. Radioactive isotopes in effective porosity determination. Atomkermenergie (ATKE) Bd. 23 (1974) Lfg. 4.

RECENT REMOTE SENSING TECHNIQUE IN FRESH WATER SUBMARINE SPRINGS MONITORING: QUALITATIVE AND QUANTITATIVE APPROACH

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Abstract

Due to its physical characteristics a water surface is easily prospectable by means of aerial thermal infrared surveys, in particular in the 9÷11 micron band. In over eight years of activity an operational methodology has been developed in order to locate fresh water submarine springs and to evaluate their yields using electronic data processing and mathematical models. More than 700 springs spread over 1500 km of italian shorelines have been pointed out with a total yield of about 100 m^3/s. Particular information is stressed out by the processing of thermal level slicing and the texture analysis. Such a method is also useful for the evaluation of a regional aquifer hydrologic balance and for the underground water prospection along the shores.

1 Introduction

Since the beginning of years '70 the detection of fresh water springs in coastal salt water bodies has become matter of remote sensing. Although there has been employed also the multispectral imaging in the photographic domain, the thermal infrared scanning demonstrated the highest reliability in any situation, being the thermal contrast, and not the transparency the indicator of the presence of springs in the salt water.
Among the advantages deriving from the use of thermal scanning and in particular from its processing, it has to be underlined the possibility

of getting a synoptic instantaneous view on hundreds of kilometres with the reconaissance of the coastal currents pattern, the detection of any outlets and the computing of their yields.

2 The interaction of the radiance with the water

The water shows a peculiar behaviour in respect to the radiance:opaque in the ultraviolet, is transparent in the visible with a peak at 0,52 micron. Then it begins to become opaque again in the dark read (0,7 micron) and even more in the near infrared (1÷2 micron) till reaching a complete opacity in the thermal infrared region (beyond 3 micron). The opacity in the domain of thermal emission is that peculiar characteristic of water which allows the acquisition of data strictly at the surface. Basing on the superficial seepages of these phenomena the upwelling fresh water springs can be throughly revealed and the outlet flow estimated.

3 The methodology of the infrared survey

In Italy and in many other parts of the world, where exist coasts formed by carbonatic deposits which spread towards the inland establishing aquifer (area of recharge) the phenomenon of submarine springs appears. The proper knowledge of these seepages can contribute to the solution of important hydrogeologic problems (the water balance and water supply) in areas where, at times, the discovery on underground water at depth is of difficult accessibility and too expensive. Following many surveys carried out up to date (1975-1981) we have developed a detailed methodology of survey along the coast of Italy, Sicily and Sardinia which is valid for both the technical and economical point of view. In this way we have located, misted and studied more than 700 fresh water submarine springs spread along 1500 km of coast (Fig. 1). Basing on the evaluation of the flow discharged in the sea (over 150 m^3/sec) hydrogeological regional studies have been carried out in detail and indicated inland areas where exploratory wells could be performed. Methodology bases on aerial thermography whose characteristic parameters stressed out after years of experience are the following:

- Flight season: late summer (low water table period)and, eventually, the spring (high water table period)
- Time of flight: at predawn, possibly in conditions of low tide
- Flying height : 1000÷1500 m above sea level
- Instruments : infrared thermal scanner operating in the 9÷11 micron band, inside the 8÷14 atmospheric window, where the absorption is minimum and there is the maximum thermal emission for a black body around 20°C. Other instruments of the photographic type have proved useless.

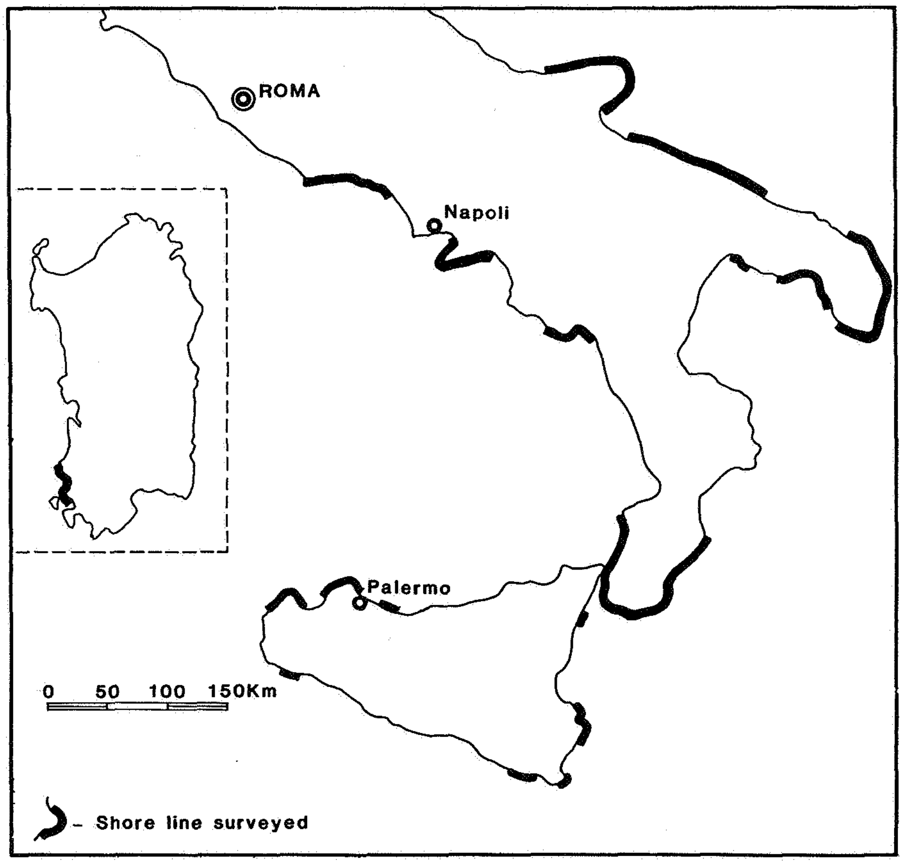

Figure 1 Frame of South Italy coasts surveyed with aerial thermal infrared scanning for the inventory of submarine springs

The data required from the flights are electronically treated to obtain the level slicing (Fig. 2) for the drawing up of the isotherm map for every coastal inflow with its relative topographic location (Figure 3).

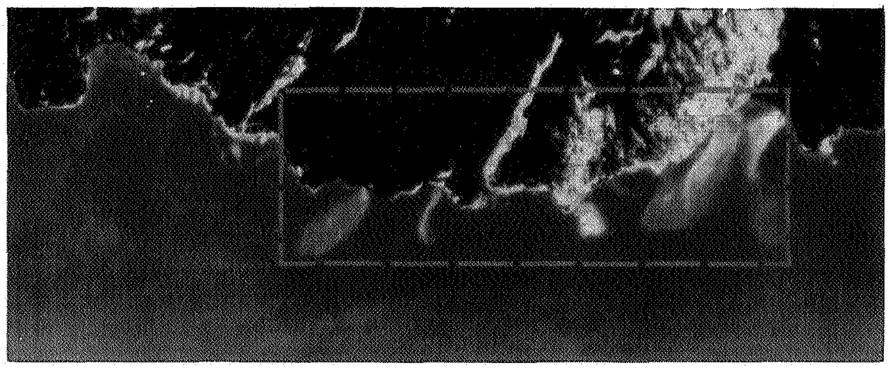

Figure 2 Level slicing of a thermal survey (Southern Italy). The thermal scale increases from white to black with 0.5 °C interval (approximate scale 1:25.000)Aut. S.M.A. N° 53 7.2.78

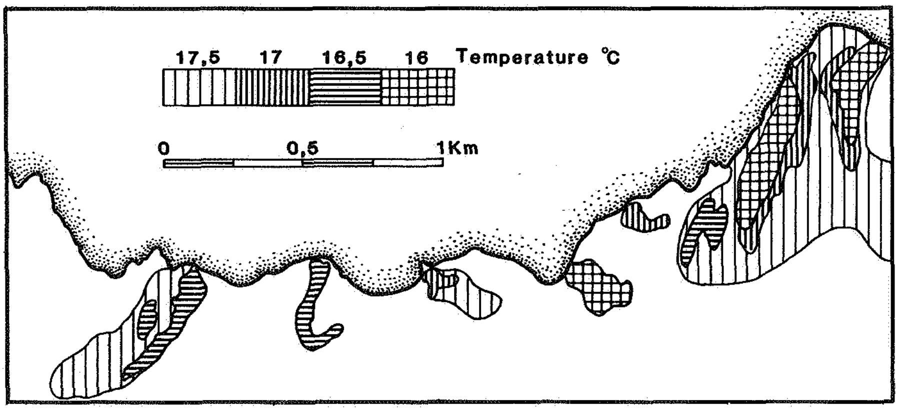

Figure 3 Example of isothermal map: the contouring includes isothermal areas with 0.5 °C interval. The shoreline is that inside the dashed frame of Figure 2

The final location of the submarine springs is carried out after a hydrological interpretation which begins with the checking of the original thermal data and is completed by field checks, in such a way to isolate those seepages (e.g. polluted discharge) which has nothing to do with the springs.

The map of isotherms, as defined, is used for computing the yield, basing on the thermal balance, occurring at the sea surface between spring and salt water, which we briefly describe.

4 The yield computing of submarine springs

The submarine spring of fresh water is usually colder than the sea it flows into. For this reason it would tend to lay at the bottom. What causes it emerge to the surface is the contrast of salinity (therefore of the density) between the two liquids. Normally the total salinity content is that of making the spring plume to float till the temperature difference doesn't exceed 16 °C. Practically a fresh water outlet floats always on the sea surface (normally the difference observed is less than 10°C). The yield of the spring is calculated evaluating the heat exchanged by the outlet with the sea and with the atmosphere.

The following formula is considered:

$$m/t = 1/c\ (Ts - Tm) \Big\{(5 + V/1100)\ \Sigma i\ Ai\ (Ti - Ta) + \\ + 4.95 \cdot \varepsilon \cdot \Sigma i\ Ai\ \left[(Ti/100)^4 - (Tm/100)^4\right]\Big\}$$

where

- m/t = mass of fresh water discharged in the unit time (m^3/h)
- c = thermal capacity of water = 1000 (Cal/m^3 °K)
- Ts = temperature of the spring outflow (°K)
- Tm = " of the sea surface (°K)
- Ta = " of the air at the sea level (°K)
- Ti = " of the i^{th} isoradiant surface (°K)
- V = wind speed at the sea surface (m/h)
- Ai = area of the i^{th} isoradiant surface of the plume (m^2)
- ε = emissivity of the water (for the 9÷11 micron band it is ~ 0.9)

In this formula the first term takes into account the heat exchanged by convection along the contact of the sea with air, the second one is concerned with the heat radiated by the plume towards the space. The thermal exchanges due to conduction and convection between the two liquids is negligeable. According to checks carried out on known outlets we can affirm that the calculations using the said formula approximates the actual value by a defect of 10÷15%.

5 The case of widespread springs or small discharges

At times large quantities of water flow into the sea in a widespread manner and across many springs each with a little discharge or with temperature contrast too low to be revealed by normal processing. In order to study the total amount of discharge it is important to define the location and area of influence also of these above mentioned seepages.
The used method bases on the fact that any discharge into the sea tends to deformate the field of the superficial currents. The horizontal movement of the current produces horizontal temperature gradients due to the mixing of layers of water at different depth. The pattern of the superficial thermal currents is stressed out by the image processing procedures able to perform the thermal texture analysis.
The texture analysis is a procedure with which are put in evidence the thermal gradients distributed coherently and in a continuous form (lines, curves) from those distributed in random manner (separation of the signal from the noise).
The method has given the most complete results being able to distinguish the alignment of very feeble gradients on the background formed by the noise. The sensitivity of this method is of about 0.1 °C.
The upwelling of a spring, even though weak, produces a disturbing effect on existing current lines or generates a typical thermal pattern. This is the reason why we can detect the presence of outlets whose flow or thermal contrast is too feeble to be observed directly on the original thermogram or on the usual enhancing treatment (level slicing, edge enhancement, etc.). Fig. 4 and Fig. 5 represent two interesting examples of texture analysis compared with the corresponding level slicing.

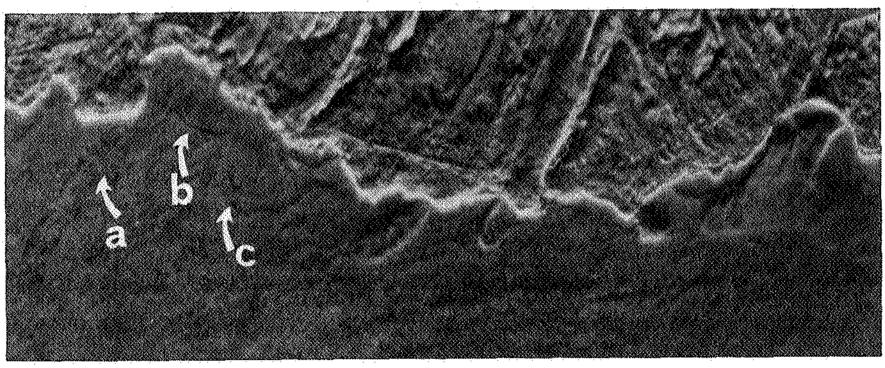

Figure 4 Comparison between level slicing (up) and texture analysis (down) of an infrared survey carried out in Southern Italy. Plumes with high temperature contrast are observable on both pictures. On the contrary the texture analysis quite enhances the typical circular pattern of very important offshore springs (known by local fishermen) without a thermal contrast (salinity profiles confirmed the complete mixing of the two liquids) sufficient to be revealed by level slicing (see arrows a,b,c)
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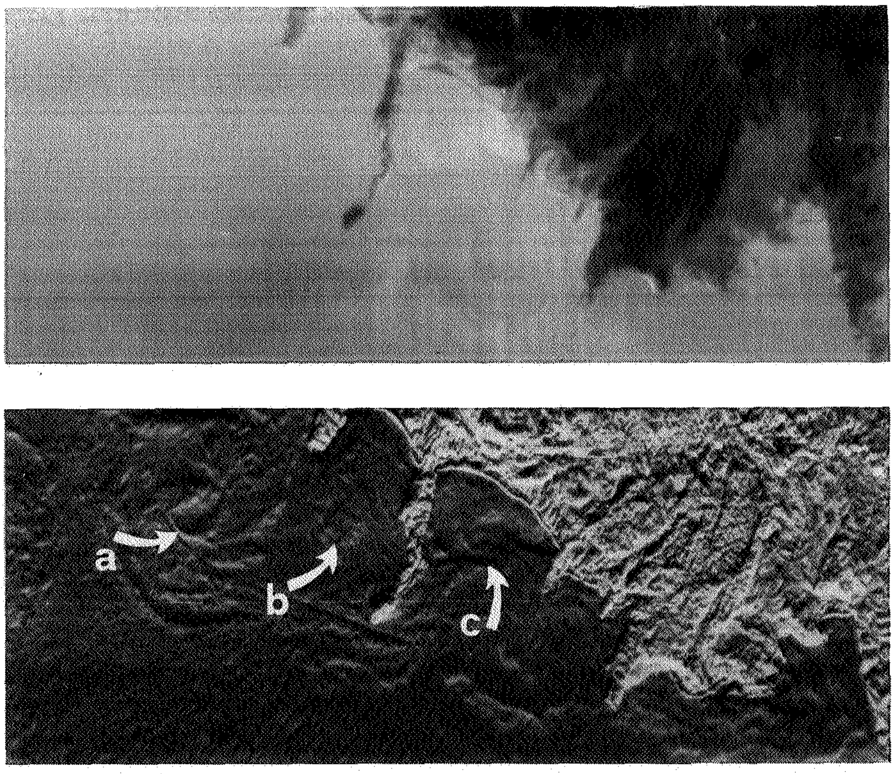

Figure 5 Thermal slicing (up) with 0.5 °C interval of Western shore of Vulcano island compared with the texture analysis (down). Arrow c indicates the thermal front due to the diffuse immission of a weak warm spring. Arrow a shows the typical spiral-shape center due to contrasting currents not linked to the presence of springs. Arrow b shows the typical thermal pattern of a very feeble cold spring not detectable by level slicing. Aut. S.M.A. N° 652 12.12.80

6 Possible applications of the method

Knowing the location of the springs and computing their flow are of great importance for two basic hydrological aims: the hydrologic balance

of the regional aquifer and the location of inshore areas where to develope research of underground waters.
In the first case taking into account the quantity of waters discharged into the sea means increasing of the same amount the direct recharge (infiltration) and therefore increasing the resource potentially exploitable. Important results have been achieved in the studies of large aquifers in Southern Italy and Sicily.
In the second case it is important to find a correlation between submarine springs and geological environment (geology and tectonics) to locate areas where to perform studies and explorative drillings. On the basis of past experience the spring is not always in relation with a fault or a fractured inland zone, however its position may increase the percent of success of explorative drilling.

7 Conclusions

In the study of regional carbonatic aquifers discharging important amount of water into the sea, the aerial thermal survey constitutes a valid tool of prospecting. For the synoptic gathering of the data and for the short time required for the survey, together with a relatively low cost, the thermographic approach is very important or indispensable for a proper study of a coastal aquifer.
The main results obtained in numerous surveys allow to consider the method useful for:

- locating and listing the fresh water sources
- computing the springs yield and performing a proper regional hydrological balance
- recognizing the zones where to try to do explorative drillings and inshore catchments.

Aknowledgements

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References

Kernan, Lee (1969). Infrared exploration for shoreline springs at Mono Lake, California, test site. Proceeding of the 6th Symposium on Remote Sensing of Environment, Ann. Arbor, Michigan (U.S.A.).

Scarpace, F.L. et al. (1972). Scanning thermal plumes. Proceeding of the 9th Symposium on Remote Sensing of Environment, Ann. Arbor, Michigan.

Lechi, G.M., Tincati, M. (1974). Quantitative evaluation of the minimum fresh water spring flow in the sea by means of thermal surveys. Proceedings of the 10th International Symposium on Remote Sensing of Environment, Ann. Arbor, Michigan (U.S.A.).

Guglielminetti, M. et al. (1974). Remote Sensing techniques applied to the study of fresh water springs in coastal areas of Southern Italy. Proceedings of the 10th International Symposium on Remote Sensing of Environment, Ann. Arbor, Michigan (U.S.A.).

Alberotanza, G., Lechi, G.M. (1977). Quantitative evaluation of water bodies dynamics by means of thermal infrared and multispectral surveys on the Venetian Lagoon. XI International Symposium on Remote Sensing of Environment, Ann. Arbor, Michigan (U.S.A.).

Tonelli, A.M. (1978). Surface texture analysis with thermal and near infrared scanner. Photogrammetric Engineering and Remote Sensing, Vol. 44, N° 10, pp. 1273-1278.

Boltri, R., Gandino,A. (1979). Multispectral aerial survey applied to fresh water submarine springs location. Proceeding of the 3rd World Congress on Water Resources, Mexico City - Mexico.

A NEW GEOPHYSICAL APPROACH USING REMOTE SENSING TECHNIQUES TO STUDY GROUNDWATER TABLE DEPTHS AND REGIONAL EVAPORATION FROM AQUIFERS IN DESERTS

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Abstract

Two new methods to determine variables relevant to the appraisal of groundwater resources are presented. First it is shown that the properties and thickness of the different soil layers can be inferred from the thermal behaviour of the soil surface. An application of remotely sensed thermal infrared data to estimate shallow groundwater table depths is presented. Secondly, a new method is discussed dealing with the determination of actual evaporation in large regions by combining visible and thermal infrared data with ground measurements.

Résumé

On présente deux nouvelles méthodes pour déterminer des variables importantes pour l'évaluation des ressources en eau souterraine. On montre d'abord qu'on peut dériver du comportement thermique de la surface les propriétés et l'épaisseur des différentes couches du sol. On présente une application de télédétection à l'infrarouge thermique pour évaluer le niveau de la nappe phréatique. La deuxième méthode s'agit de la détermination de l'évaporation actuelle des régions vastes à l'aide d'une combinaison de télémesures à l'infrarouge thermique aussi que visible avec des mesures prises à la surface.

1 Introduction

Both size and complexity of the large aquifer systems in deserts only recently have been appreciated. This applies especially to the North African deserts, where during the last ten years generally accepted views about structure and potentiality of groundwater reservoirs have been reshaped.
It appears that the accuracy required to judge projected water consumptions is not fully realized. A figure of 10^8 $m^3.year^{-1}$ is considered as representative of the gross water requirement of a typical modern sprinkling irrigation system. A conservative figure for the extension of outcrops towards the soil surface of a large aquifer system in North Africa is 10^4 km^2. Hence the corresponding water exchange rate is 0.03 $mm.d^{-1}$, which rate only can be measured by very accurately weighing lysimeters. Therefore the difference between 'groundwater mining' and 'managing a renewable resource' is a rather thin one.
In the very arid zones of the world, where it might be raining less than once a year, groundwater is the only source for agricultural development. Groundwater reservoirs loose water by evaporation through playas, which are present in every desert area. The role of such groundwater discharging areas (playas) in the water balance of large groundwater basins in desert regions has been poorly assessed. Evaporation can be relevant not only where open water is present, but also where the water table is situated at shallow depths. The variability of surface characteristics, both in space and time, plays a most important role in establishing the evaporation rate. It appears that it is not only difficult to work out a local estimate of actual evaporation, but it is even more troublesome to estimate evaporation losses in volume. An evident bottle-neck in the procedure is the estimation of the area which has a given and known evaporation rate.
The concepts and theories presented in this paper originated from a case-study on evaporation losses in playas, carried out as part of an operational hydrogeological study of the Fezzan region in Libya. Estimated evaporation losses had to be included in the specification of boundary conditions of a three-dimensional numerical model to simulate the aquifer's reaction to planned developments.

Evaporation from playas was estimated by combining ground experiments with remotely sensed data. Field experiments in the winter and summer of 1978 included an InfraRed LineScanning (IRLS) airborne survey both a day and a night one of a 1500 km^2 area, continuous operation of an adequate agro-meteorological station, energy balance measurements above eight different surfaces (bare and cropped soil), and soil moisture measure-

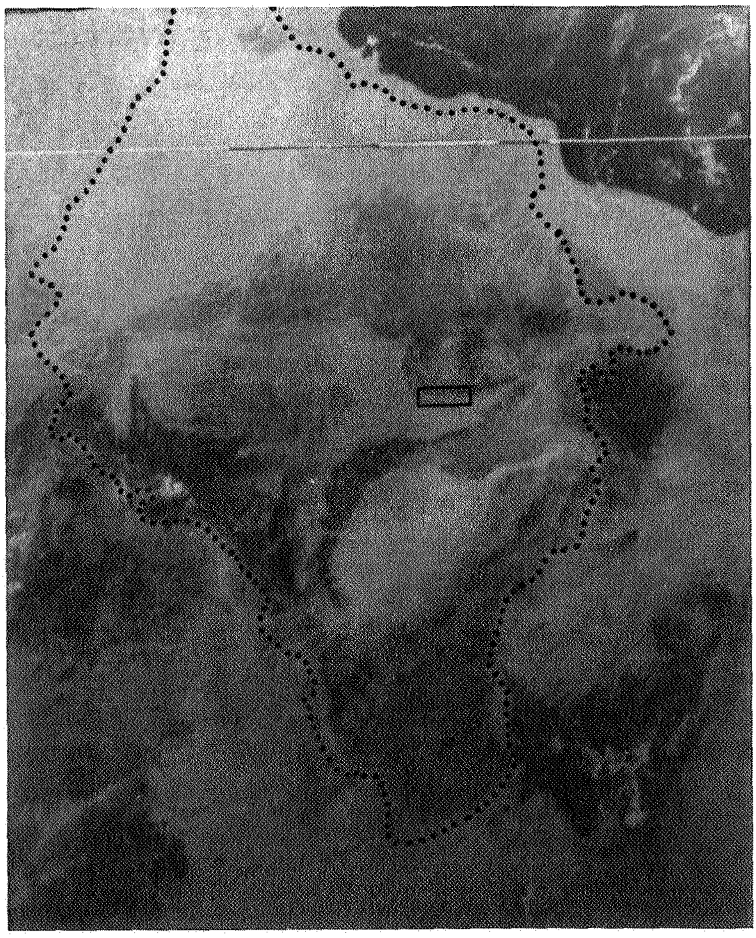

Figure 1 Visible satellite image of the western Libya aquifer (dotted line) received on 19 September 1978 through the Defense Meteorological Satellite Program (DMSP). The area in Figure 6 is also indicated

ments by neutron probe at some thirty places. Laboratory experiments included the determination of soil moisture retention and hydraulic conductivity curves. Satellite imagery was also applied, both in the visible and thermal infrared spectral region, at different resolutions. A number of images gathered by the LANDSAT, HCMM and METEOSAT scanning radiometers was used. Emphasis was given to the quantitative analysis of thermal infrared data.

Photographic imagery proved an excellent mapping tool at the scale of the study. In Figure 1 a Defense Meteorological Satellite Program (DMSP) visible image of the Libyan desert is presented. Boundaries of the aquifer system are clearly visible, especially the Amguid fault system in the top left of the image.

2 Physical background

To give a qualitative impression of the theory to be presented in sections 3 and 4, a few remarks will be made on some physical facts relevant to the problems discussed in section 1. Relationships of physical facts with effects that can be remotely sensed are briefly discussed.

Some of such physical facts are:

a) a shallow water table,

b) a moist top layer,

c) a very efficient heat transport in the top soil layer,

d) the availability of energy for evaporation.

Items a) and c) are connected with the thermal behaviour of the soil surface, while items b) en d) relate to both soil thermal behaviour and surface reflectivity.

2.1 Thermal behaviour

ad a) In terms of thermal properties the depth of a water table is defined as the depth below which soil thermal diffusivity is equal to that of saturated soil. The thermal behaviour of the soil surface depends on groundwater table depth. The deeper the groundwater level, the lower the frequency of the temperature wave that reaches it and in reverse affects the behaviour of surface temperature.

ad b) The behaviour of the surface temperature at different frequencies of the impinging temperature wave allows for estimation of the moisture content of the top soil when it has been wetted by rain or runoff. Soil thermal diffusivity is relatively high in a moist top soil, then decreases in the drier layers underneath, to increase again closely to the water table.

ad c) Soil thermal conductivity enhanced by moisture or air flow is close to that found in soils with relatively high moisture contents. On the contrary the heat capacity of the same soil layer is typical of soils with low moisture content. Such contrasting evidence is typical of heat flux enhanced by soil air flow.

ad d) When evaporation takes place inside the soil, the soil heat flux is larger than when the evaporation front is situated at the soil surface.

To translate the thermal behaviour described under a) to d) into quantitative parameters, a forcing fucntion is needed. It is well known that weather variability has many harmonics. Surface temperature undergoes natural oscillations, which can provide an opportunity to study the structure of the upper few meters of the earth skin.

2.2 Surface reflectivity

ad b) Wet soils have a relatively low surface reflectivity, which increases linearly with decreasing moisture content.

ad d) Surface reflectivity provides two types of information with respect to the actual evaporation rate. Surfaces with low reflectivity have a higher evaporation rate, everything else being equal. A threshold for surface reflectivity can be established to decide whether evaporation takes place at the soil surface or below it. In section 4 a method to estimate actual evaporation from remotely sensed surface temperature and reflectivity will be presented.

3 Detection of shallow groundwater tables

The thermal admittance y (= $\lambda\gamma$) of a semi-infinite medium is defined by:

$$G(z,t) = \lambda\gamma\ T(z,t)\ ;\ |\gamma| = \left(\frac{2\pi\rho_s c_s}{P\ \lambda}\right)^{\frac{1}{2}} \qquad (1)$$

where

G = soil heat flux

z = vertical distance

λ = soil thermal conductivity

T = soil temperature

$\rho_s c_s$ = soil heat capacity

P = the period of the temperature wave

The parameter $\lambda\gamma$ can be written as:

$$\lambda\gamma = \exp\ i\ \pi/4\ .\ (2\ \pi\ \lambda\ \rho\ c/P)^{\frac{1}{2}} \qquad (2)$$

where

$\lambda\gamma = y$

Eq. (1) is obtained when seeking a traveling wave solution to the heat conduction equation, under the boundary conditions:

$$T(0,t) = A\ \exp\ i(\omega t-\phi) \qquad (3a)$$

and

$$\lim_{z\to\infty} G(z,t) = 0 \qquad (3b)$$

Details of the derivation are given by Carslaw and Jager (1959; page 69). For the three-layer system depicted in Figure 2, the ratio of the apparent thermal admittance at the soil surface y_s to the thermal admittance of the first layer y_1 is (Menenti, 1983) :

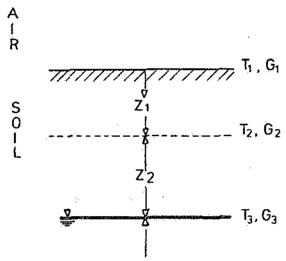

Figure 2 Sketch of a three layer soil. Soil temperature and heat flux are defined at the upper boundary of each layer. The upper boundary of the deepest layer at $Z_g = Z_1 + Z_2$ corresponds with the shallow groundwater table depth

$$\frac{y_s}{y_1} = \frac{\text{Re } N + i \text{ Im } N}{\text{Re De} + i \text{ Im De}} \tag{4}$$

where the expressions for each term are:

$$\text{Re } N = ((b_1+b_3)e^{B} + (b_1-b_3)e^{-B})\cos B + ((b_2+b_4)e^{D} + (b_2-b_4)e^{-D})\cos D \tag{5}$$

$$\text{Im } N = ((b_1+b_3)e^{B} - (b_1-b_3)e^{-B})\sin B + ((b_2+b_4)e^{D} - (b_2-b_4)e^{-D})\sin D \tag{6}$$

$$\text{Re De} = ((b_1+b_3)e^{B} - (b_1-b_3)e^{-B})\cos B + ((b_2+b_4)e^{D} - (b_2-b_4)e^{-D})\cos D \tag{7}$$

$$\text{Im De} = ((b_1+b_3)e^{B} + (b_1-b_3)e^{-B})\sin B + ((b_2+b_4)e^{D} + (b_2-b_4)e^{-D})\sin D \tag{8}$$

The following symbols are used to simplify the expressions:

$$b_1 = c_2 + c_3 \qquad b_2 = c_2 - c_3 \qquad b_3 = 1 + c_1 \qquad b_4 = 1 - c_1 \tag{9}$$

$$c_1 = |y_2|/|y_1| \qquad c_2 = |y_3|/|y_1| \qquad c_3 = |y_3|/|y_2| \tag{10}$$

$$B = |\gamma_1 z_1 + \gamma_2 z_2| \qquad D = |\gamma_1 z_1 - \gamma_2 z_2| \tag{11}$$

It should be recalled that according to eq. (2) the thermal admittances y_i depend on the period P of the impinging temperature wave and are complex variables.

Taking the thermal admittance y_3 in Figure 2 as applying to a saturated soil, the depth (Z_1+Z_2) is equal to the groundwater table depth Z_g. From eq. (4) an expression can be derived for the modulus k representing $|y_s/y_1|$:

$$k = \frac{[(\mathrm{Re}\ N)^2 + (\mathrm{Im}\ N)^2]^{\frac{1}{2}}}{[(\mathrm{Re}\ De)^2 + (\mathrm{Im}\ De)^2]^{\frac{1}{2}}} \tag{12}$$

which can be calculated as a function of Z_2 for different values of Z_1. Eq. (12) has been calculated for a number of combinations of Z_1 and Z_2 with both Z_1 and Z_2 varying in the range 0.01 m to 7 m, and a number of temperature waves with periods ranging from 1 to 550 days. The thermal properties of the three layers of the desert sand are given in Table 1, where 1, 2 and 3 relate to the upper layer going downwards to the water saturated region. The values of $|y_i|$ of Table 1 are plotted in Figure 3. The ratio k can be plotted as a function of P for a few different combinations of Z_1 and Z_2. Figure 4 relates to $Z_1 = 0.5$ m with $Z_2 = 0.2$, 0.5, 1, 2, 5 and 7 m. When large period lengths have been obtained, the separate curves proved to have a good accordance with the specific thermal admittance graphs (the general shape of which was shown in Figure 3). The curves in Figure 4 therefore provide a good description of the layering of the soil.

Table 1. Thermal admittance $|y|$ of a desert sand for different moisture conditions. Thermal conductivity (λ) and heat capacity (ρc) also are given

	$\|y_i\|$ $(W.m^{-2}.K^{-1})$	ρc $(J.m^{-3}.K^{-1})$	λ $(W.m^{-1}.K^{-1})$
1, top layer	11.1	$0.96 \cdot 10^6$	1.4
2, middle layer	3.7	$0.96 \cdot 10^6$	0.2
3, saturated subsoil	19.3	$3 \cdot 10^6$	1.7

Thus plots as in Figure 4 can be applied to estimate Z_1 and Z_2 as appearing in eq. (11), and therefore in eqs. (4) and (12), from experimentally

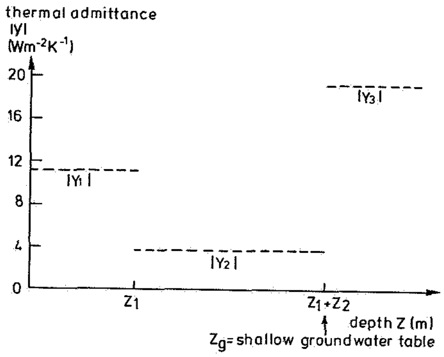

Figure 3 Modulus of thermal admittances applying to the system depicted in Figure 2. See also Table 1

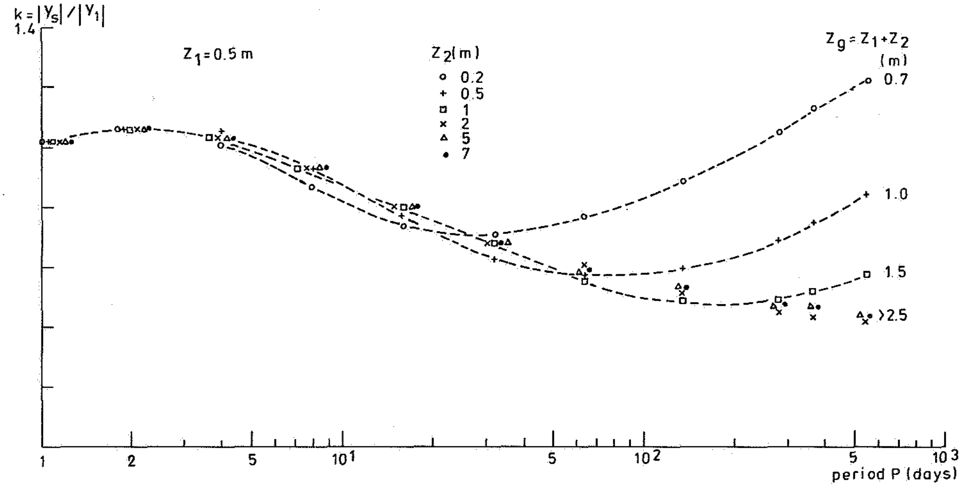

Figure 4 Modulus of the ratio k of apparent surface thermal admittance $|y_s|$ and thermal admittance of the top soil layer $|y_1|$ vs. the period P of temperature waves. Thickness Z_1 of top layer is 0.5 m; the thickness of the second layer varies from 0.2 m to 7 m. Each curve relates to a constant groundwater table depth. $Z_g = Z_1 + Z_2$

obtained values of k. Values of k are plotted vs. the period P. Then a curve of the family generated by eq. (12) is to be found that fits this experimentally obtained curve. Eventually the unknown Z_1 and y_i variables are obtained as the parameters of the curve best approximating the measured points.

Problems involved in the determination of k will be discussed in detail in Menenti (1983). Here it suffices to point out that only ratios of amplitudes of temperature and heat flux at the surface are needed. Furthermore certain categories of errors on these amplitude ratios will not cause major problems. A constant error percentage will only change the absolute value of k, but not its shape which still will vary with the groundwater table depth as it is expected. A normally distributed error over not too large period intervals will place the measured points above and below the true curve, but the problem can be overcome in the fitting procedure. It should also be noted that a calibration of the experimentally obtained curves can be achieved by comparing some of them with the actual soil layering. This latter task is by far easier than calibrating an estimated distribution of surface temperature or soil heat flux. Finally it must be noted that a long term drawdown of a water table will give curves showing a gradual connection of the curve pertaining to the initial groundwater depth and the curve for the groundwater depth finally reached.

Although the procedure was applied to estimate the groundwater table depth $Z_g = Z_1+Z_2$, the thermal behaviour of any layered structure can be studied at shallow depths according to the same approach. For a more than three layered system, graphs similar to the ones of Figure 4 can be drawn.

4 Estimation of regional evaporation

To relate the actual evaporation rate to more readily measurable variables, a combination of the energy balance equation, a transport equation for sensible heat in air and additionally known relationships can be defined as an explicit function LE(α_s, R_s, T_a, T_s, r_a, G_E), where

α_s	= surface reflectivity (0.4-1.1 μm)	(-)
R_s	= shortwave incoming radiation (0.4-2.5 μm)	$(W.m^{-2})$
T_a	= air temperature	(K)
T_s	= surface temperature	(K)
r_a	= aerodynamic resistance	$(s.m^{-1})$
G_E	= soil heat flux at the evaporation front	$(W.m^{-2})$

From satellite data only surface temperature and reflectivity can directly be obtained and G_E can be inferred from them. Before devising means to obtain the other variables it is important to establish the relative weight of each variable. This can be accomplished by calculating partial differentials of LE(D_{x_i} LE δ x_i) for a pre-specified set of average conditions and ranges of variability (δx_i) for each variable x_i. Values as in Table 2 were applied. The results are presented in Table 3. It appears that the most significant contributions are due to T_s, G_E and α_s in this order. To decide about the feasibility of a calculation procedure based on remotely sensed data the difficulties involved in performing the required ground measurements are to be known.

Table 2 Numerical values for average conditions in the Libyan desert of the parameters appearing in the function LE

Parameter	α_s	R_s	T_a	T_s	r_a	G_E
Value	0.4	250	300	305	200	100
Unit	-	$W.m^{-2}$	K	K	$s.m^{-1}$	$W.m^{-2}$

Table 3 Numerical values of partial differentials D_{x_i} LE for average conditions in the Libyan desert. The assumed range for each independent variable is also shown (δx_i)

	D_{α_s} (LE)	D_{R_s} (LE)	D_{T_a} (LE)	D_{T_s} (LE)	D_{r_a} (LE)	D_{G_E} (LE)
$W.m^{-2}$	25.	0.	-22	110.	-15	-50
δx_i	0.1	0.	2	10	100	50
Unit	-		K	K	$s.m^{-1}$	$W.m^{-2}$

It should be noted that the figures in Table 3 do not imply that the variables R_s, T_a, r_a are unrelevant or should not be measured if this easily can be done. The range of variability indicates that fewer measuring sites are needed for R_s, T_a, r_a than for T_s, α_s and G_E. Values as given in Tables 2 and 3 relate to the experiments performed in the Libyan desert. Under those conditions for the Wadi Ash Shati Basin (see Figure 6) of 2.10^3 km^2 surface area, one agrometeorological station with an additional measuring point for T_a suffices to make an efficient and feasible remote sensing approach to study actual evaporation. With increasing density of the required ground measurements, this approach becomes less effective and in the end comparable with direct use of ground data only. It has been shown (Menenti, 1980) by comparing measured LE-values with a first order Taylor's expansion of LE that the latter is a satisfactory approximation when ground reference measurements at few sites are available.

The kind of approximation achieved by the above presented procedure, can be explained as follows.

An estimator f_1 of LE should be defined with which a six-dimensional (α, R_s, T_a, T_s, r_a, G_E) space is mapped onto a set of LE-values:

$$\{\alpha_s, R_s, T_a, T_s, r_a, G_E\} \xrightarrow{f_1} \{LE\} \tag{13}$$

Since only α_s, T_s can be determined from remote observations, i.e. over the entire area, it appears that the problem of defining an estimator f_2:

$$\{\alpha_s, T_s\} \xrightarrow{f_2} \{LE\} \tag{14}$$

is ill-posed, unless the 6-strings corresponding to the same 2-string can be specified. This difficulty becomes even more clear in Fig. 5, where a surface e.g. a function $LE = LE(\alpha_s, T_s)$ has been depicted. Any variation in α_s and T_s causes the point P* to move to P", e.g. LE". The points P*, P" and the arc P*-P" lie, by definition on the surface $LE = LE(\alpha_s, T_s)$. In a real case the same couple $(\alpha_s", T_s") \equiv P"$ may relate to different sets of the remaining variables in eq. (13). Thus the actual evaporation

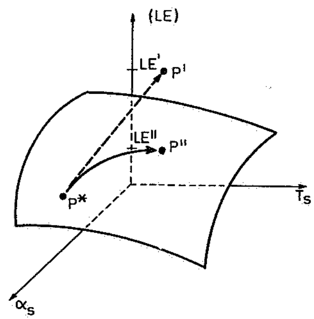

Figure 5 Sketch showing possible errors on estimates of latent heat flux due to approximated formulas of lower than six-dimensionality

rate at P" can be different: LE' instead of LE" say. Hence the variables which cannot be measured areally must be assigned to a particular set of (α_s, T_s) couples on the basis of a-priori knowledge.
A linear estimator f_2' can be obtained by casting in matrix form the condition for four points to lie on the same plane:

$$\begin{vmatrix} \alpha_s - \alpha_s^1 & T_s - T_s^1 & LE - LE_1 \\ \alpha_s^1 - \alpha_s^2 & T_s^1 - T_s^2 & LE_1 - LE_2 \\ \alpha_s^3 - \alpha_s^2 & T_s^3 - T_s^2 & LE_3 - LE_2 \end{vmatrix} = 0 \qquad (15)$$

where the coordinates of points $P_1 \equiv (\alpha_s^1, T_s^1, LE_1)$, $P_2 \equiv (\alpha_s^2, T_s^2, LE_2)$ and $P_3 \equiv (\alpha_s^3, T_s^3, LE_3)$ are specified and $P_4 \equiv (\alpha_s, T_s, LE)$. When numerical values are substituted in eq. (15), one finds for LE:

$$LE = a_1\alpha_s + a_2T_s + a_3 \qquad (W.m^{-2}) \qquad (16)$$

where a_i are dimensional constants. The coordinates of the points P_i can be calculated from a reference value LE_o and the derivatives $D_{\alpha_s}(LE)$ and $D_{T_s}(LE)$. It can be expected that points defined in this way are not completely representative of the actual shape of the LE-function. Thus experiments must be made and one or more measured 3-strings, or points, included in eq. (15). A further improvement in the estimate of LE can be achieved by applying corrections to account for the effect of those variables which cannot be observed areally. Finite increments can be calculated by integration of partial differentials D_{x_i} LE δx_i. To calculate these corrections ground based measurements again are needed. A more detailed description of the procedure can be found in Menenti (1980).
According to the eq. (15) an estimator of the kind specified by eq. (14) was established applying to the Wadi Ash Shati basin in Fezzan Libya:

$$LE = -8.52\alpha_s - 0.125T_s + 43.7 \qquad (mm.d^{-1}) \qquad (17)$$

The approach involving the determination of f_2' is essentially different from the inversion procedures described in literature. There an estimator f^{-1} is defined to calculate from variables defined at a given point $(\bar{x},\bar{y})$ the soil and latent heat fluxes defined at the same point $(\bar{x},\bar{y})$:

$$\{\alpha_s(x,y),\ T_s(x,y),\ \ldots\} \xrightarrow{f^{-1}} F(x,y) \tag{18}$$

Now f_2' on the contrary bears only an implicit dependence on space:

$$\{x,y\} \xrightarrow{f_3} \{\alpha_s(x,y),\ T_s(x,y)\} \xrightarrow{f_2'} \{LE\} \tag{18}$$

The dependence on space, f_3, is specified by the sensor mapping α_s, T_s, while f_2' relates state variables to each other.

By means of eq. (17) a map of actual evaporation was calculated from HCMM (Heat Capacity Mapping Mission) data. A result is presented in Figure 6. Daytime visible and thermal infrared data relate to 18-9-1978 and night-time thermal infrared data to 16-9-1978. Experiments performed at overpass time and during the entire year 1978 provided the required ground reference data.

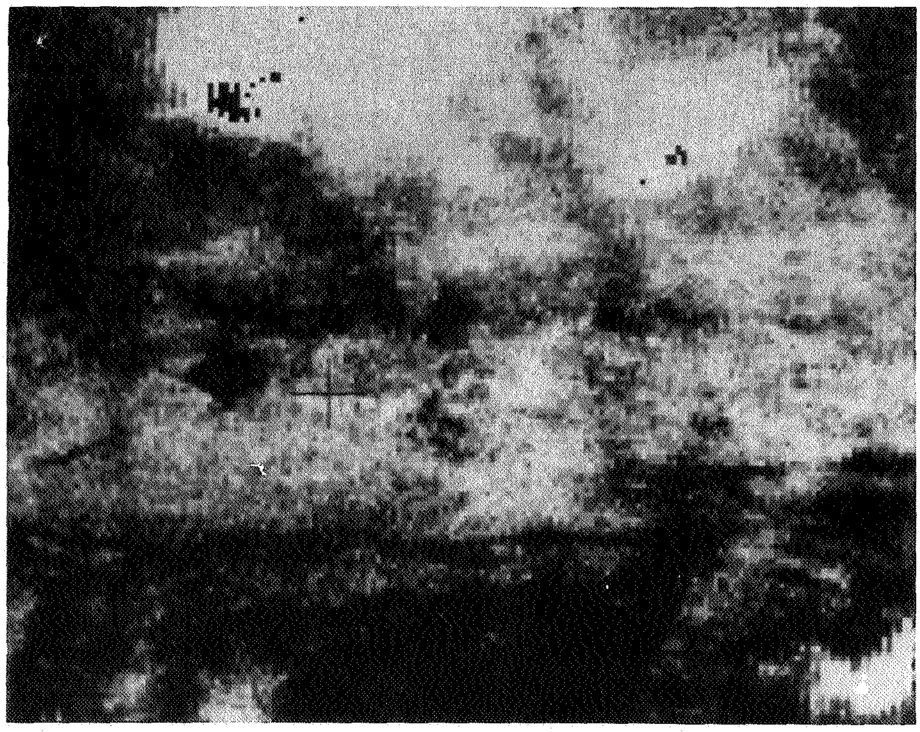

Figure 6 Evaporation map of Wadi Ash Shati calculated from HCMM visible and thermal infrared data. At the point indicated by the cross the evaporation rate is 2.3 $mm.d^{-1}$. The darkest pixels indicate the lowest evaporation rate

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References

Carslaw, H.S. and J.C. Jaeger 1959. Conduction of heat in solids. 2nd ed. Clarendon Press, Oxford, 510 pp.

Menenti, M. 1980. Defining relationships between surface characteristics and actual evaporation rate. TELLUS Newslettersn. 15, JRC Istra, 21 pp.

Menenti, M. 1983. Determination of evaporation in deserts applying remote sensing techniques. PhD thesis Agricultural University, Wageningen, the Netherlands (in preparation).

APPLICATION OF REMOTE SENSING IN GROUNDWATER RESOURCES EVALUATION

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Abstract

The Sironcha tehsil in the Chandrapur district of Maharashtra State, India is a densely forested, tribal land. It has remained unexplored due to lack of all-weather roads and communication facility. When the Government of Maharashtra decided to open this backward region to industrialization, it became imperative to evaluate its water resources. This region is enclosed by three perennial rivers but the interior parts have no surface water resource. Hence, stress had to be given on evaluating groundwater resources.

This paper describes how remote sensing techniques were applied in conjunction with conventional land based methods to evaluate groundwater resources of a 6 000 sq.km area in this region.

1 Introduction

This study area, 6 000 sq.km, is located between latitudes $18^{\circ}41'$ N - $19^{\circ}45'$ N and between longitudes $79^{\circ}45'$ E - $80^{\circ}42'$ E. This region is inhabited mostly by tribals and had not been explored for its resources though resources were known to be present. Very dense teak wood forests cover about 70% of the surface in this region. Lack of all weather roads cuts off this region from the rest of the country for about 3 months during rainy seasons every year. In the light of these constraints, this region was ideal for application of remote sensing techniques in determining its groundwater resources.

2 Methodology

Since this region under study was hither to unexplored, a method was evolved which would yield knowledge of the vital components of the water mass balance equation. Thus following mass balance equation was set up in terms of only the most significant factors.

$$\text{Precipitation} = \text{Runoff} + \text{infiltration to subsurface water} + \text{evapotranspiration} \tag{1}$$

Following steps were involved in the actual determination of the components mentioned in equation (1).

2.1 Preparation of Base Map

For the purpose of preparation of base map, following inputs were utilised

1. topographical maps
2. aerial photographs - panchromatic
3. Landsat MSS (Multi Spectral Scanner) imagery.

The area was first outlined on topographical maps and boundaries were identified. This map was updated with information superposed from aerial photographs which were of a later date. Base Map of southern portion of study area is shown in Figure 1.

A drainage map was also prepared from base map by incorporating details from a Landsat M.S.S. imagery which was of latest date.

2.2 Estimation of Total Precipitation

The study area is, fortunately, covered by meteorological observatories. The annual and seasonal isohyetal maps were produced by averaging data for over thirty years. The isohyetal map was superposed on the drainage map. By planimetry, total precipitation was determined for each of three drainage basins in the study area.

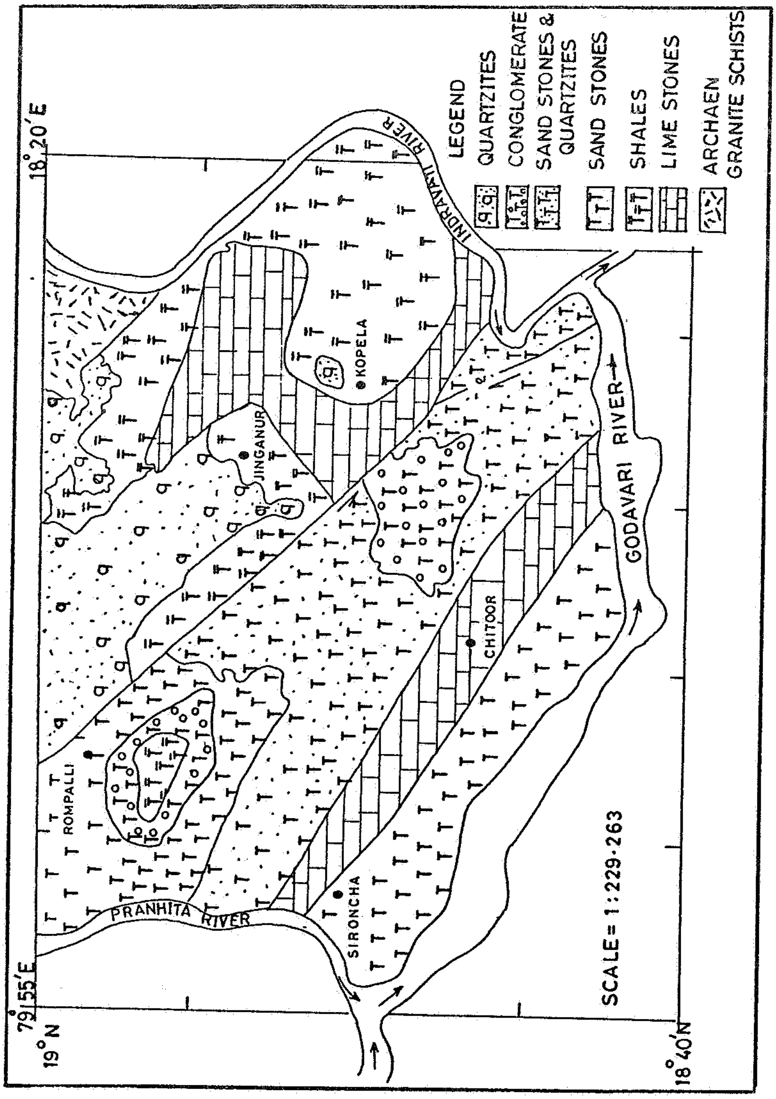

Figure 1 Base map of Southern portion of study area Chandrapur district, Maharashtra, India

2.3 Estimation of Infiltration

A pedological map showing various soil units in the study area was prepared from aerial photographs. The soil units were classified on the basis of the parent material from which they were formed and their occurrence with respect to physiographic units. During the field visit, checks were made to correct the map.

On each of the distinct soil units,infiltration tests were carried out using a Single Ring Infiltrometer with a diameter of 457.2 mm (18'').

By planimetry, areas in each basin covered by various soil units were calculated. Assigning field measurements values of infiltration over each distinct soil unit; basin-wise loss of water due to infiltration was calculated.

2.4. Estimation of Evapotranspiration

Using Penman's Equation, which is based on the most complete theoretical approach which relates evapotranspiration to quantum of radiative energy incident on surface, evapotranspiration losses for the entire area were calculated.

Evaporation pan tests were made at 2 different sites in the study area to determine mean monthly extra terrestrial radiation in mm of water evaporated per day.

Area of reflecting surface was determined from aerial photographs.

2.5 Calculation

By reconstituting equation 1, runoff was obtained as:

$$\text{Runoff} = \text{Precipitation} - (\text{evapotranspiration} + \text{infiltration}) \tag{2}$$

Runoff was also calculated using empirical equation given by Msrs. Inghs and D'Souza, which is applicable for plateau region to the east of Ghats in Maharashtra. The equation is

$$R = \frac{P - 7}{100} \times P \qquad (3)$$

were R = Annual runoff in inches

P = Annual precipitation in inches.

For one of the major streams in the study area, viz. Godavari River, annual mean runoff data were available. Figures of runoff calculated using equations (2) and (3) compared favourably with the actual data. Departure of calculated data from actual data was found to be $\pm$ 3.5%.

3 Estimation of groundwater Resources

Figures of infiltration for the study area had indicated the amount of water that was augmenting the subsurface storage of water. To determine spatial and temporal variation of groundwater resource in the study area, over 100 dug wells were monitored during pre-monsoon and post-monsoon periods spread over two years.

A groundwater table contour map was generated and areas surplus and deficient in groundwater availability were marked. Groundwater yield capabilities of the entire region were indicated in another map which now is being used for guiding groundwater exploration in this region. Pre-monsoon groundwater table contour map for southern portion of study area is shown in Figure 2 and that of post-monsoon period is shown in Figure 3.

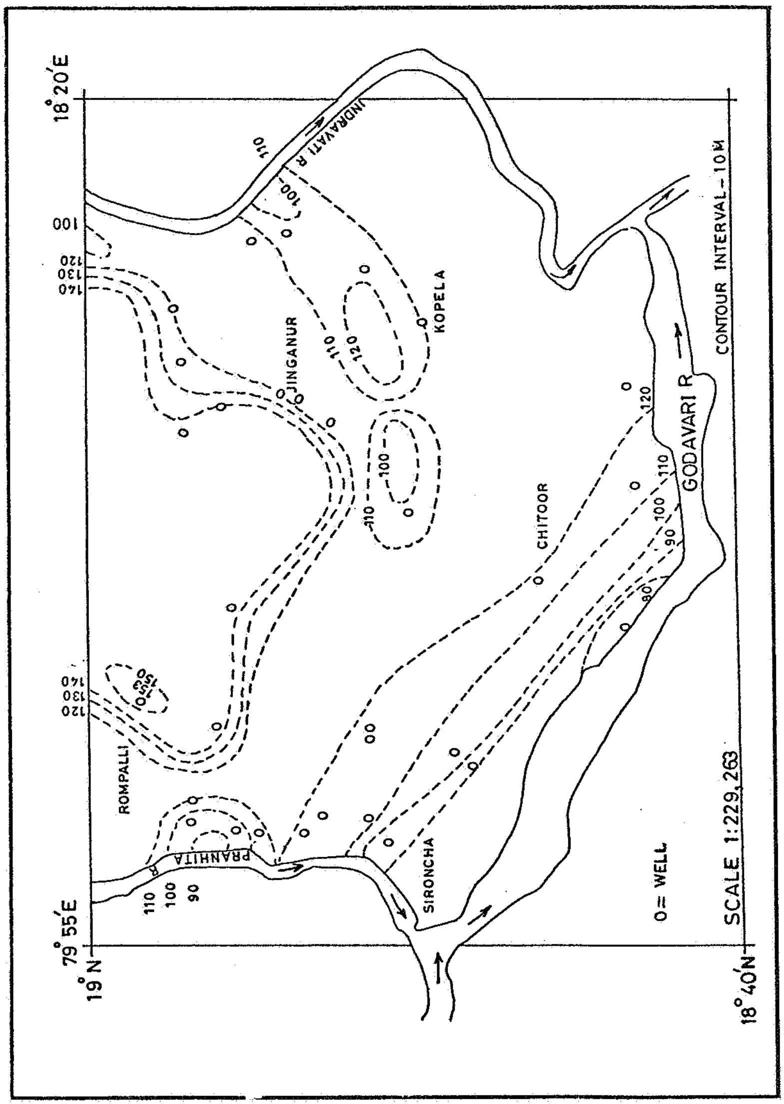

Figure 2 Groundwater contour map postmonsoon-period Nov. , 1977

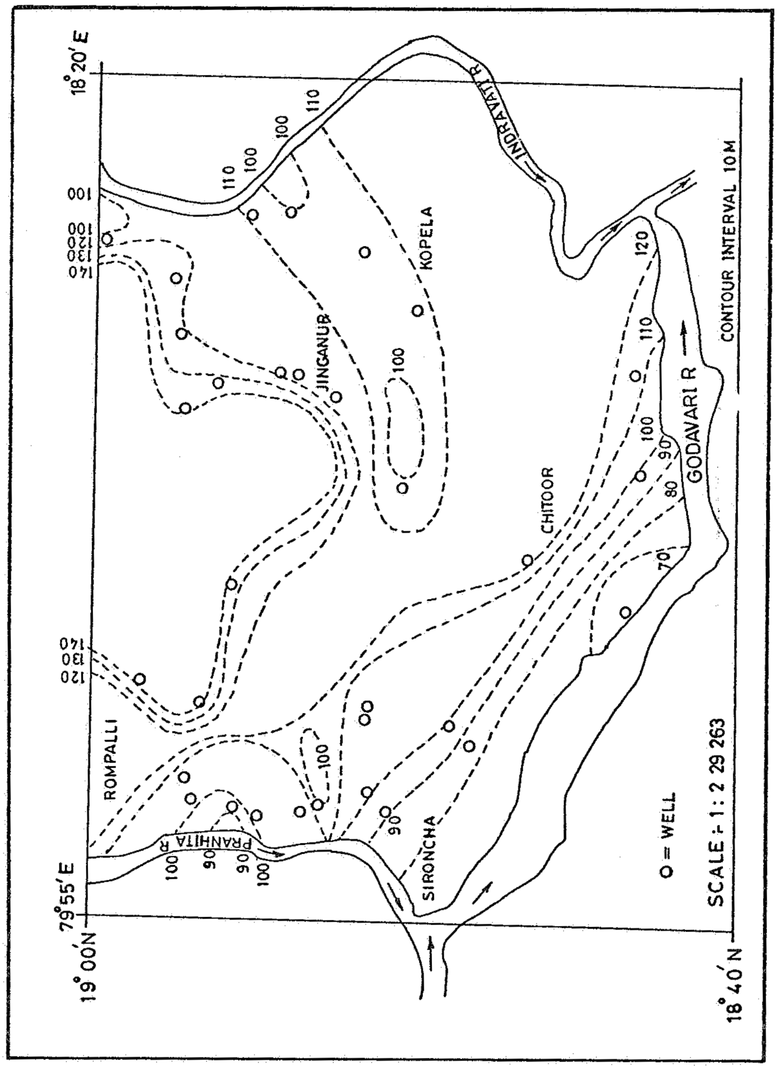

Figure 3 Groundwater contour map premonsoon period May, 1977

Mass Balance of the entire study area is shown in Table 1.

Table 1 Results of Investigation

Area	5987	Sq.km.
Mean Precipitation	1475	mm
Total Precipitation	883116	ha.m
Evapotranspiration	294010	ha.m
Infiltration	132658	ha.m
Runoff	456449	ha.m

4. Conclusion

Groundwater resources of this study area were evaluated using mass balance equation. This approach has proved very useful in assessing resources of this densely vegetated, unexplored region. Remotely sensed data enabled in completing this study in a very short duration of about 12 months whith only two scientists.

Acknowledgements

The Authors wish to thank the Head, CSRE, IIT, Bombay for guidance and encouragement they received throughout this project. The Pedology division of CSRE provided the Soil map.

ESTIMATING THE NATURAL GROUNDWATER STORAGE OF INTERMONTANE DEPRESSIONS USING REMOTE SENSING TECHNIQUES

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Abstract

A method for estimating the natural groundwater resources of intermontane depressions, based on the use of remote sensing data and knowledge of the general geological and physiographic conditions of the area under investigation is discussed. This method allows the investigators to obtain a preliminary estimate of the thickness of the sedimentary cover of intermontane depressions and their natural groundwater storage.

1 Introduction

Natural groundwater resources are an important component that is taken into account when solving problems, associated with the evaluation of groundwater safe yield, movement, circulation rates, and discharge. The natural groundwater resources of a groundwater basin, area, or aquifer includes all types of groundwater flowing or filling porous media. Under artesian conditions, they also include "elastic" resources formed as a result of water release under pressure from an artesian basin (Kudelin, 1960).

The general formula for estimating natural groundwater storage is

$$W_s = F \cdot H_s \cdot n_e \tag{1}$$

where W_s = groundwater storage
F = aquifer area
H_s = saturated thickness
n_e = effective porosity

Formula (1) shows that the estimation of the natural resources of an aquifer, water-bearing system, or groundwater basin is a complicated task, since it requires a large amount of factual data for determining such an important hydrogeological parameter as the thickness of water-bearing rocks over the area of their occurrence. This task becomes particularly complicated under specific conditions of intermontane depressions with a sharp lithological and permeability non-uniformity of rocks in horizontal and vertical directions. Therefore, a new method allowing the investigators to make a preliminary estimate of the natural groundwater storage of these areas, not resorting to exploration drilling or geophysical investigations, which are used in traditional methods for estimating natural groundwater resources, is proposed.

2 Method for determining natural groundwater storage of intermontane depressions

The method to be discussed is based on using satellite pictures and knowledge of the physiographic and geological conditions of the area under investigation. As to physiographic conditions, we should bear it in mind that a river in an intermontane depression with a closed hydrographic cycle is not only a climatic product, but also a result of the tectonic regime of the area: the rise and subsidence of the basement of the depression resulted in the fluctuation of the base level of erosion and therefore affected the formation of the river valley (Kuznetsov, 1968).

The investigations carried out by Obruchev (1945), Murzaev (1952), and Kuznetsov (1968) convincingly proved that the main features of the topography of Central Asia had formed in the Mesozoic area, though the hypsometric levels of elevations were lower than those of recent ones. At that period, there was a river and lake network separate areas of which were located where they are at the present time that is indicated by the fact that valleys of recent rivers may be confined to fairly thick conglomerates within ancient alluvial fans. In the cenozoic era, orogenic processes became more active that resulted in the separation of the Paleozoic fold country, already broken into blocks, into horsts (mountain ranges) and grabens (intermontane depressions) at various elevations. This topography constantly developed in the course of the struggle between erosion eliminating uplifted areas and vertical movements raising them. The topography, created by the latest uplifts at the end of the Neogene period or the beginning of the Quaternary period, has been preserved until the present time due to later lower uplifts.

In addition to topography, climate (air temperature, the amount and regime of precipitation in particular) makes a significant contribution. Sinitsyn (1959), Kuznetsov (1968), and Murzaev (1952) have established that the aridization of the climate of Central Asia began in the Upper Jurassic epoch and continued until recently. The paleoclimatic, paleohydrological, and palynological investigations, made within Central Asia, show that rivers being a product of the climate were no more water abundant in the geological past as compared to the recent epoch, and therefore the climate of the region is comparable to that of the present time.

The functions of mountains and valleys differentiated due to the increase in aridity: mountainous areas became river runoff formation areas, while valleys became river runoff dissipation areas. The reconstruction of the river network, however, is more frequently associated with mountain building processes. An example is the Gobi-Altaĭ earthquake of the 4th of December, 1957, that resulted in the rise of a part of the Gobi Altaĭ ranges up to 2-7 m, the rock displacements were responsible for situations where a new river was formed in one of the intermontane valleys and existing valleys with streams were buried (Solonenko et al., 1960).

The external look of rivers is also associated with slow uplift and subsidence of separate blocks: in subsiding areas they constantly wind, change their plane outlines; valleys are poorly expressed, in uplift areas a river may flow in one downcut valley. The step-like character of longitudinal river valleys, the transformation of drainage lakes into drainless ones, deep gorges, and narrow flushed valleys connecting different depressions are indicative of these block movements. For instance, the separation of the large basin of the existing drainless lake Kukunor from the Hwang Ho River system is thought to be due to the action of tectonic and erosional factors rather than due to the change in the water abundance of rivers. This shows that the recent erosion rate in the majority of regions of Central Asia appreciably lags behind the rate of tectonic movements. Thus, tectonic movements are leading not only in the formation of the structural look of a region, but also in changing the outlines of the drainage network. The structural look of vast areas with an extended network of rivers and lakes is distinctly seen in satellite pictures.

The block structure of the sedimentary cover indicates that the tectonic zones limiting the blocks are likely to be relatively stable, retaining their position for all the period of the formation of the area. The change in the structural look of the area occurring in periods of tectonic activity results in the redistribution of blocks of emerging uplifts and subsided formations, in the change of their orientation, etc., but the boundaries of the blocks remain unchanged or move slightly.

Proceeding from the above, analysing a satellite picture in detail and taking into account natural factors, geological and structural features in particular, one may distinguish artesian basins of different orders, determine their boundaries, and make measurements of the areas of the basins and individual natural elements within them. The modern high-resolution optics allows the investigators to determine, accurately, using stereoscopic pictures, the relative elevation of some topographic features as compared to other features. The obtaining of quantitative indices of intermontane basins from satellite pictures is not so difficult since the open vast areas of Central Asia are practically free from interferring factors, such as vegetation, agricultural activities and the like.

Proceeding from the above, we may assume the existence of the proportional relationship between the width 'b' and depth 'h' of the downcutting of a recent river valley and the width 'B' and depth 'H' of a distinctly shaped tectonic accumulation drainless depression (The depth of river downcutting is also determined as the difference between the weighted average height of the catchment area h_c and the average elevation of the longitudinal profile of the river channel h_r: $h = h_c - h_r$):

$$\frac{h}{b} = \frac{H}{B} \qquad \text{or} \qquad H = \frac{(B \cdot h)}{b} \qquad (2)$$

In fact, the 'H' value characterizes the thickness of the loose sedimentary cover of depressions, while the 'b' and 'h' parameters are easily taken from large-scale stereoscopic photographs of the land surface, and the 'B' parameter is readily taken from medium- and small-scale satellite pictures (Dzhamalov et al., 1977; Obyedkov, 1982). The 'h', 'b', 'B' values are averaged for an intermontane valley.

We should know the volume of rocks filling the depression in order to estimate natural groundwater resources. In the case of the simple structure of the crystalline basement (the single block basement), the rock volume is

$$V = F_d \cdot H = F_d \cdot \frac{B \cdot h}{b} \tag{3}$$

where V = volume of rocks filling the intermontane depression in cubic metres

F_d = depression area in square metres

H = depth of the depression or thickness of its sedimentary cover in metres

However, the crystalline basement of a depression is often a system of small blocks at different elevations. Then, in formula (2) the parameter $H = H_{av}$ and it expresses the generalized characteristic of the depression depth. Knowing the volume of rocks filling the intermontane depression, we may determine natural groundwater resources W_n^* which, according to formulas (1) and (2), are estimated as

$$W_n^* = F_d \cdot \alpha \cdot (H_{av} - h_a) \cdot n_e \tag{4}$$

or

$$W_n^* = F_d \cdot \alpha \cdot \left(\frac{B \cdot h}{b} - h_a\right) \cdot n_e \tag{5}$$

where α = coefficient taking into account the relative amount of water-bearing rocks in the profile

h_a = thickness of the zone of aeration (in metres) determined on the basis of general hydrogeological characteristics of the area.

The rest of the symbols are defined in formulas (1), (2), (3). The values of the porosity of rocks composing the sedimentary cover are taken from reference publications (e.g. Maksimovich, 1948).

3 Results of estimation of natural groundwater resources

The method for estimating natural groundwater resources in intermontane depressions was tested in some artesian basins of Central Asia for which sufficient factual data were available. The thickness of the sedimentary cover of the basin was estimated using gravimetric survey data, which allowed determining the character and depth of the crystalline basement of the depression, and the thickness of water-bearing rocks was evaluated using data obtained from deep oil exploration drilling, water well boring (to depths of 250-400 m), and geophysical measurements of numerous sites and profiles. The rest of the parameters, required for estimating natural groundwater resources, were evaluated from experimental field and laboratory data.

The data on the natural groundwater resources of intermontane depressions of one of the regions of Mongolia, estimated using geophysical and exploration information as well as satellite pictures at regional and local levels of generalization are given in Tables 1 and 2.

The analysis of the parameters and values of natural groundwater resources obtained using satellite pictures and exploration data, shows that sedimentary cover thickness values estimated from formula (2) are 10-15% smaller than actual ones. Natural groundwater resources values, determined from formulas (4) and (5), are smaller than actual ones by the same percent.

4 Conclusion

The discussed method allows the investigators to determine approximately the thickness of the sedimentary cover of intermontane depressions and roughly estimate their natural groundwater storage. Using this method, it is possible, however, to obtain the integrated values of rock thickness and natural groundwater resources for all the hydrogeological structures,these values for individual aquifers or water-bearing systems or

Table 1 Natural groundwater resources estimated using exploration drilling and geophysical data

Basins and main rivers	Basin area (10^9 m^2)	Average depression depth (m)	Aeration zone thickness (m)	Relative amount of water-bearing rocks in profile (α)	Water-bearing rocks thickness (m)	Effective porosity n_e (%)	Natural groundwater resources W_n (10^9 m^3)
Bontsagan Nur Baidrik River	8.5	620	11	0.75	465	20	617
Orog Nur Tuin River	4.2	850	8	0.70	595	18	322
Barun Bayan Ulan Tatsain River	3.2	780	7	0.75	585	19	334
Arguin Gol Arguin River	2.5	320	4	0.80	255	20	119
Ongiin Gol Ongiin River	17.0	640	9	0.65	420	22	1255

Tabel 2 Natural groundwater resources estimated on the basis of satellite pictures and publications

Basins and main rivers	Average thickness of zone of aeration	Relative amount of water-bearing rocks in profile α	Effective porosity n(%)	Main indices of					Natural groundwater resources $W_n^*(10^9 m^3)$
				River valleys		Artesian Basins			
				Width $b(10^3 m)$	Down-cutting depth h(m)	Width $B(10^3 m)$	Basin depth or sedimentary cover thickness H(m)	Water-bearing rocks thickness (m)	
Bontsagan Nur Baidrik River	10	0.70	0.15	7.5	30-33	130	572	390	490
Orog Nur Tuin River	10	0.70	0.15	4.5	50-52	67	774	530	335
Barun Bayan Ulan Tatsain River	10	0.70	0.15	3.0	20-25	85	730	510	245
Arguin Gol Arguin River	5	0.75	0.20	2.0	45-48	12	288	210	105
Ongiin Gol Ongiin River	10	0.70	0.15	3.0	10-20	82	570	390	995

for separate areas within the artesian basin cannot be obtained. The method does not take into account the "elastic" artesian groundwater resources, though their contribution to total natural groundwater resources is insignificant as calculations show. To sum up, the proposed method for regional estimation of natural groundwater resources has a number of advantages, if the area under investigation is poorly studied as far as its hydrogeological conditions are concerned, since it does not require expensive exploration and experimental investigations.

References

Dzhamalov, R.G., I.S. Zektser and Yu.L. Obyedkov. 1977. Deciphering of aerial and satellite pictures in studying groundwater discharge. Razvedka i okhrana nedr, No. 1 (in Russian).

Kudelin, B.I. 1960. Principles of regional estimation of natural groundwater resources. Moscow, MGU Publ. (in Russian).

Kuznetsov, I.I. 1968. Waters of Central Asia. Moscow, Nauka (in Russian).

Murzaev, E.M. 1952. Mongolian People's Republic. Physiographic description. Moscow, Geografgiz (in Russian).

Obruchev, V.A. 1947. Depressions of Central Asia and their scientific treasures waiting for study. Izvestia AN SSSR, Seriya geologicheskaya, No. 5 (in Russian).

Obyedkov, Yu.L. 1982. Methods for estimating the total groundwater resources and the rate of groundwater circulation of an intermontane artesian basin in Mongolia. Vodnye Resursy, No. 3 (in Russian).

Sinitsyn, V.M. 1959. Central Asia. Moscow, Geografgiz (in Russian).

Solonenkov, V.P., A.A. Treskov and N.A. Florensov. 1960. Catastrofic Gobi Altai earthquake of 4 December 1957. Moscow, Gosgeoltekhizdat (in Russian).

THE IN-SITU MEASUREMENT OF
HYDRAULIC CONDUCTIVITY OF WEATHERED
OR POORLY-CONSOLIDATED MATERIALS
AT SHALLOW DEPTHS

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Abstract

Knowledge of the hydraulic conductivity of shallow weathered or poorly consolidated materials is important to the understanding of infiltration and movement of pollutants. Difficulties in sampling such material necessitate the use of in-situ tests. Techniques have therefore been developed which enable reproducible measurements of hydraulic conductivity to be made in-situ on loose materials using installations which can be emplaced between 1.5 and 20 metres below ground level. The installations comprise a range of modified piezometers which can be bought ready made or easily constructed. Surface test equipment employing injection, and where possible pumping, tests at constant or variable head is described and the results of such tests at two sites are discussed.

1 Introduction

To understand groundwater flow in rocks it is necessary to know how the hydraulic conductivity of the system varies both laterally and with depth. Knowledge of the hydraulic conductivity of materials can be important in understanding infiltration, movement of pollutants and subsoil behaviour during irrigation. Unfortunately much shallow material is insufficiently consolidated for undisturbed samples to be removed for laboratory testing and tests must be performed in-situ. Also, the limited saturated thickness of such materials often prevents their testing by conventional pumping tests.

Unsaturated material can be tested by water injection, though precise analysis of the results is difficult and various formulae have been used (Stephens and Neuman, 1982). However we consider that the variations between formulae are not significant when compared with the errors caused by difficulties in obtaining a clean, unsmeared undamaged test zone. We have therefore tried to develop a simple, cheap technique whereby a repeatable value of saturated hydraulic conductivity can be obtained for a small volume of shallow material above or below the water table. By using such a technique it was hoped that enough values of hydraulic conductivity could be obtained to enable the spatial variation of this property to be determined for a given material.

2 Equipment

2.1 Downhole installation

The technique involves emplacing an installation of known geometry associated with surface equipment with which to perform constant or variable head injection tests or, in the saturated zone, a pumping test. Two downhole installations have been used (Figure 1).

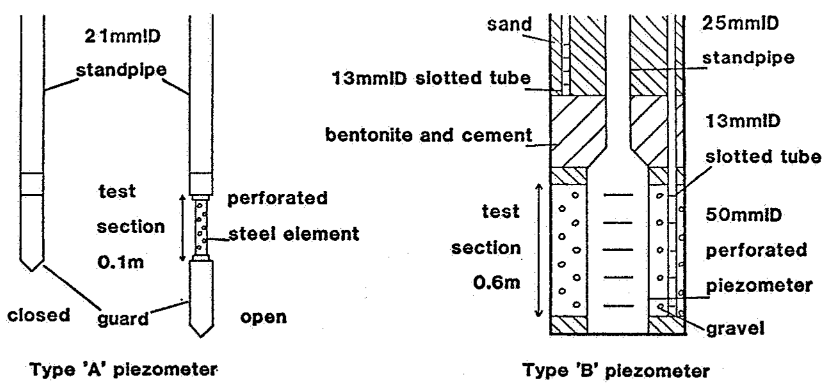

Figure 1 Downhole installations

The type 'A' piezometer consists essentially of a standard Cambridge drive-in piezometer (Parry, 1971), manufactured by Soil Instruments Ltd. and modified by removal of the porous element and the drilling of additional holes to increase the length of the perforated section to 0.1 m. The perforated section, made from 16 mm OD plated mild steel is protected during installation by a 27 mm OD conical-ended steel guard which is retained as a sliding fit by a rubber O-ring.

The piezometer is emplaced either by being driven from the surface, or inserted into a slightly undersized hole made by a hand auger. After emplacement the guard is knocked off by a steel mandrel which is inserted through the standpipe into the piezometer, thus exposing the test section. Weathered material at depths down to 3.5 metres has so far been tested using these piezometers. The installations are cheap to buy and emplace, and can be withdrawn and re-used (although the guard is lost on withdrawal).

The type 'B' piezometer (Figure 1) consists of a 50 mm ID perforated plastic or metal pipe, blanked off at one end and reduced at the other to accept 25 mm ID water pipe. Its emplacement procedure is more complicated than that for the type 'A' piezometer. Initially a 100 mm hole is drilled slightly deeper than the intended test zone depth. A power auger has proved satisfactory in weathered sandstone and non-flinty Chalk to depths of 15 m. Below this depth, or where flints are troublezome, a truck mounted percussion rig has been used. The piezometer and standpipe are then emplaced in the hole along with 13 mm ID PVC access tube which is slotted over the length of the test section. The access tube allows the head in the test zone to be monitored directly. Both the standpipe and the access tube extend about 0.5 metres above ground level to facilitate connection to the test apparatus.

The bottom of the hole is backfilled with sand to define the bottom of the test zone, followed by gravel, and finally more sand to define the top of the test zone. These materials are emplaced via a 25 mm ID temporary plastic pipe so that damage to the test zone is minimised. Cement/bentonite pellets are used to seal the top of the test zone, and a second plastic access tube perforated over the bottom 0.5 m is then emplaced to detect leakage through the plug. The hole is then back-filled to the surface with sand.

The test zone length is taken to be that of the gravel pack, and the test zone radius to be that of the hole. During a test the head in the test zone is measured with an electric water level dipper, and a similar instrument is used to detect any leakage through the grout plug during injection tests. The installation measures hydraulic conductivity over a larger volume than the type 'A' piezometer, is more proof against leakage, can be emplaced deeper, and can be used for a greater variety of tests. However it costs more to emplace, particularly if a truck-mounted rig has to be used.

2.2 Surface equipment

2.2.1 Injection testing

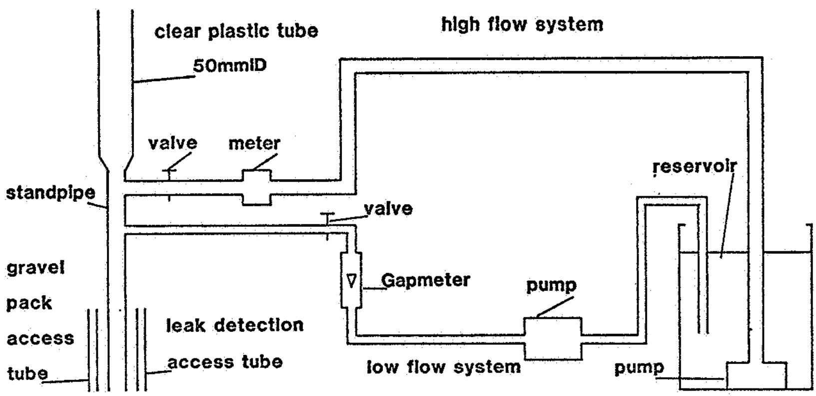

Figure 2. Surface equipment for injection test

For an injection test two surface systems are available, providing either low or high injection rates (Figure 2). For the low flow system water is pumped from a reservoir by a small centrifugal pump through a Gapmeter™ flowmeter via a regulating tap into the standpipe. By interchanging tubes and floats the Gapmeter is able to accurately measure flows up to 5 l/min, above which rate the high flow system is used.

In the high flow system water is pumped by a submersible garden fountain pump in the reservoir through a helical flowmeter and regulating valve into the piezometer standpipe. The flowmeter is accurate between 2 l/min and 110 l/min, and the submersible pump will deliver up to 60 l/min.

During an injection test, when the injected head is higher than the top of the test zone access tube a 2 m long 25 mm ID clear plastic tube is attached to the top of the access tube, and the head is observed directly. The 50 mm ID clear plastic tube attached to the top of the standpipe serves three main functions. It provides a means of escape for air displaced during the early part of the test when the pipework is filling with water; it is an effective surge tank which damps small oscillations in the discharge from the fountain pump (a common event caused by variation in electrical generator output); and it provides the means of head measurement during injection tests on type 'A' piezometers where the head in the test zone cannot be measured directly. These values of head are corrected for head losses through pipework and the piezometer tip. A subsidiary function of the 50 mm tube is that it provides a reservoir during a falling head test - the head during the test being measured in the test zone access zone.

All the equipment used for injection testing is readily available and is relatively cheap. However, two modifications can reduce the cost still further, if required. Firstly, the electrically-driven pumps (and therefore the generator) can be eliminated if a high constant-head reservoir tank is used (Pearson and Money, 1977). Secondly, the injection rate can be determined from the rate of fall of water level in a tank supplying the constant-head tank, thus removing the need for flowmeters.

2.2.2 *Pumping tests*

When the test zone is beneath the water table a small-scale pumping test may be performed. Figure 3 shows the equipment used. A downhole submersible pump cannot be used because of the small diameter of the standpipe, and therefore a system employing a vacuum to provide suction from the surface is used. Essentially the system uses a vacuum pump to evacuate a cylinder. Water is drawn into the evacuated cylinder from the piezometer via a plastic pipe placed inside the standpipe. The rate of discharge is controlled by a regulating valve on the discharge pipe and the discharge is measured by observing the rate at which the cylinder fills. Two cylinders are used so that flow is not interrupted - when a cylinder is full, flow is switched to the second cylinder, while the first is emptied and evacuated.

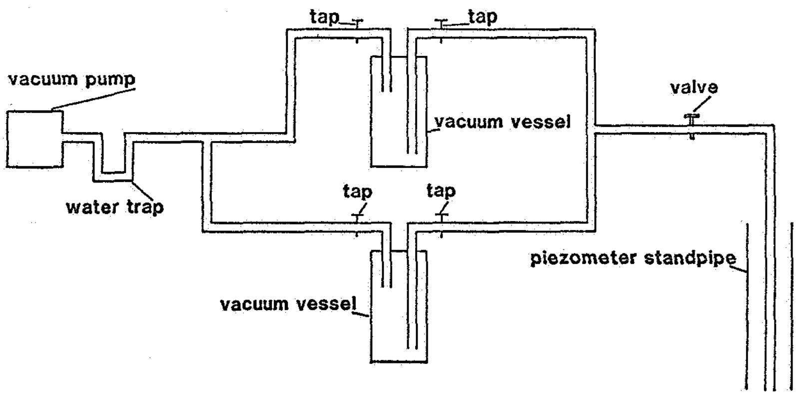

Figure 3 Surface equipment for pumping test

As with the injection tests the head (or in this case the drawdown) is measured by observing the level of water in the access tube in the gravel pack. This implies that constant drawdown tests cannot be performed using the type 'A' piezometer because there is no access tube. However rising-head tests can be employed by removing a 'slug' of water from the standpipe and monitoring the recovery of the water level within the standpipe.

3 Test sites

Tests have been carried out on the Chalk at two sites in north-west Norfolk, England. The sites are in the Terebratulina lata zone which dips gently eastward at less than $\frac{1}{2}^{\circ}$ and is represented by beds of rather hard platy chalk with frequent scattered and tabular flint bands

(Peake and Hancock, 1961). Periglacial weathering during the Pleistocene has extensively affected the near-surface chalk which is highly fissured and covered by 1 to 2.5 metres of drift and soil.

At the first site seven type 'A' piezometers were emplaced at depths ranging between 1.5 and 3.5 metres, and ten type 'B' piezometers were installed at depths ranging between 2 and 17.5 metres. The water table at this site is always below the deepest piezometer.

At the second site the water table varies between 2 and 6 metres below ground level and 11 piezometers were emplaced, most of which lay in the zone of water table fluctuation, the total variation in piezometer depth being 1 to 6.5 metres. One type 'A', eight type 'B' and two shorter versions of the type 'B' were used.

4 Testing

Only injection tests could be performed at the first site. These were in the form of stepped constant-head injection tests with three steps. Falling-head tests were also performed for comparison. Each piezometer was tested at least twice; once during the winter, when the soil moisture deficit was lowest, and once during the summer, when the deficit was highest. Most of the piezometers were tested more frequently, over a period of two years.

At the second site the piezometers were tested by injection when the test zone was unsaturated and when it was saturated, and by pumping when the test zone was saturated.

5 Theory

Assuming that the material around the test zone is homogeneous, isotropic and saturated and that the injected water is flowing into the material under conditions which satisfy Darcy's law, then the hydraulic conductivity, K, of the material under conditions of constant rate injection is given (Hvorslev, 1951, Figure 12) by:

$$K = Q \ln [\ell/2r + (1 + \{\ell/2r\}^2)^{\frac{1}{2}}]/2\pi\ell H \qquad (1)$$

where

Q = injection rate

H = injection head in the test zone

ℓ = length of test zone

r = radius of test zone

Equation (1) can be reduced to:

$$K = \frac{QS}{H} \qquad (2)$$

where S is a constant for a given installation and is termed the shape factor.

Where the test zone is below the water table H is measured relative to the water table. In the case of injection tests in unsaturated material we assume that the material under test is saturated, and H is measured from the centre of the test zone. Support for this assumption comes from the results from the second site, where values of hydraulic conductivity derived from injection tests in unsaturated material, and from the same material when saturated, are substantially the same (Allen, 1981). Pumping tests are treated in the same way as injection tests, with H being taken as drawdown instead of the injected head.

For falling head tests equation (2) is modified (Allen and Price, 1980) to:

$$K = SA \ln (H_o/H_t)/t \qquad (3)$$

where

A = cross-sectional area of standpipe

t = time since start of test

H_o = head at start of test

H_t = head at time t

Where rising head tests are performed, H_o and H_t refer to drawdown.

6 Results

As stated above, it is considered that values of hydraulic conductivity derived from injection tests in unsaturated material represent values of saturated conductivity.

At the first test site, where the fullest investigation has taken place, values of hydraulic conductivity generally decrease with depth, ranging from about 15 m/d at the top of the Chalk to 0.1 m/d at about 15 metres depth. These values were obtained in winter. As the soil moisture deficit built up through the spring and summer the values of hydraulic conductivity tended to fall to a minimum value in late summer (when the soil moisture deficit was highest) which was, in general, about 50% of the winter value. It is considered that this is because of air entrapment in the tested material in summer (Price et al., 1979).

The above values of hydraulic conductivity are about an order of magnitude higher than those obtained from similar material by irrigation experiments (Wellings and Cooper, 1982). However these discrepancies are not unexpected because the irrigation values were obtained from material which still retained a tension of a few centimetres of water. This tension would be sufficient to prevent flow in the larger fissures which, in the material tested, would provide most of the saturated permeability.

At the second site, where pumping tests were possible, we found no significant difference between values obtained by pumping and by injection, indicating that the injection technique does not cause fracturing of the tested material.

Falling head tests from both test sites provided similar results to constant head injection tests and have been used to extend the lower limit of the injection technique.

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References

Allen, D.J. 1981. Hydraulic conductivity tests on weathered Chalk in the zone of water table fluctuations. IGS unpublished report WD/ST/81/18.

Allen, D.J. and Price, M. 1980. Hydraulic conductivity tests in the unsaturated zone of the Chalk: Further development and work in north-west Norfolk. IGS unpublished report WD/ST/80/15.

Hvorslev, M.J. 1951. Time lag and soil permeability in ground water observations. Bulletin 36, US Waterway Experimental Sta. Vicksburg, Miss.

Parry, R.H.G. 1971. A simple drive piezometer. Geotechnique, 21 (2), 163-167.

Peake, N.B. and Hancock, J.M. 1961. The Upper Cretaceous of Norfolk. Trans. Norfolk and Norwich Nat. Soc. 19 (6), 293-339.

Pearson, R. and Money, M.S. 1977. Improvements in the lugeon or packer permeability test. Quart. Jour. Eng. Geol. 10. 221-239.

Price, M, Morris, B.L. and Allen, D.J. 1979. Hydraulic conductivity tests in the unsaturated zone of the Chalk - development and initial results from two sites in north-west Norfolk. IGS unpublished report WD/ST/79/14.

Stephens, D.B. and Neuman, S.P. 1982. Vadose zone permeability tests: summary. J. Hyd. Div. Proc. Am. Soc. Civ. Eng., 108 (HY5), 623-639.

Wellings, S.R. and Cooper, J.D. 1982. The variability of recharge of the English Chalk aquifer. Conference proceedings: European Geophysical Soc. and European Seismological Commission, Leeds.

L'UTILISATION DE CERTAINS MÉTHODES GÉOPHYSIQUES À L'INVESTIGATION DES AQUIFÈRES DANS LES FORMATIONS CARBONATÉES

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Résumé

Les forages exécutées à travers des formations aquifères carbonatées, à l'aide des installations de forage hydraulique à circulation inverse, n'offrent pas d'informations complètes sur le collecteur par lequel l'eau circule. L'utilisation des combinaisons de méthodes géophysiques d'investigation adéquates pour l'étude des formations carbonatées nous a permis la détermination assez exacte de ses traits physiques, de la lithologie et la structure géologique, de la minéralisation du fluide; ces données-là sont utiles pour l'étude de la dynamique complexe des eaux souterraines qu'on rencontre dans une zone de côte de la mer Noire, à structure géologique compliquée, où la couche aquifère est intensément exploitée.

Abstract

Boreholes sunk in carbonized water bearing formations by means of hydraulic drilling rigs with reverse circulation do not yield complete information on the conduit through which the water is calculating.
The application of combinations of adequate geophysical investigation methods for the study of carbonized rocks has allowed us to determine rather accurately the physical features, the lithology and the geological structure and the mineralization of the liquid. Such data are useful for the study of the complex dynamics of the groundwater encountered in

coastal zone of the Black Sea which has an intricate geological structure, and where the aquifer is being intensively exploited.

1 Introduction

L'utilisation en Roumanie du forage hydraulique à circulation inverse pour l'investigation et l'exploitation des eaux souterraines a engendré l'élaboration des programmes d'investigation géophysique complexe des trous de forage âfin d'obtenir les données nécessaires au choix de plus efficientes solutions d'équipement des forages à filtres, pompes etc. en vue d'exploitation. La réussite de ces programmes d'investigation géophysique dépend du choix judicieux des méthodes aussi bien que de la corrélation des résultats à fin d'obtenir correctement les caractéristiques physiques des formations aquifères (porosité, perméabilité, épaisseur, lithologie, aspects tectoniques etc.).

Le programme d'investigations exécutées sur un forage emplacé dans des formations carbonatées a inclut:

- des déterminations pétrographiques et microfaunistiques en vue de reconstituer la colonne lithologique, les limites et les rapports stratigraphiques et tectoniques entre les formations;
- l'investigation géophysique complexe à l'aide de l'installation PGAC;
- le teste de la couche aquifère :
  - le prélèvement des échantillons d'eaux pour des analyses isotopiques et physico-chimiques;
  - le mesurage de la pression et de la température;
  - des pompages expérimentaux.

2 Cadre naturel de l'emplacement du forage

Le forage d'investigation a été emplacé dans une zone située à l'est de la Dobrogea méridionale, limitrophe au littoral roumain de la mer Noire (Figure 1).

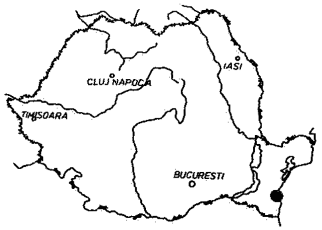


Figure 1 Emplacement du forage

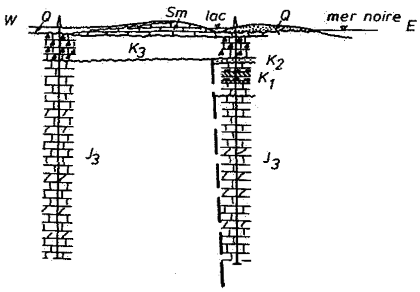


Figure 2 Section géologique dans la zone du forage

La Dobrogea méridionale représente une unité structurale connue dans la littérature géplogique comme la "Plate-forme sud-dobrogéenne", Mutihac et Ionesi (1973). Cette unité-là présente des traits caractéristiques à la sêdimentation et à l'évolution tectonique des zones de plate-forme qui contiennent des formation carbonatées très épaisses qui surmontent un soubassement cristallin.

L'objet de nos investigations est représenté par la couche aquifère des formations jurassiques, intensément exploités. On a emplacé le forage entre la zone exploitée et la rive de la mer, à fin de l'utiliser comme forage d'observation pour l'etude de la limite eau douce - eau salée (Figure 2).

3 Programme d'investigation géophysique

On le sait que l'utilisation du forage hydraulique à circulation inverse n'offre pas d'informations exactes sur la lithologie des formations traversées, sur la limite entre elles et surtout sur leurs caractéristiques physiques.

L'équipement des forages d'exploitation pase des problèmes concernant l'efficiênce et le prix de revient.

L'efficience d'un tel investissement doit être maximum, surtout dans les conditions présentes, ce qui implique :
- l'investissement initial minimum;
- frais d'exploitation réduits;
- extraction du débit maximum;
- fonctionnement de longue durée du puits.

L'accomplissement de ces conditions-là dépend surtout d'une bonne connaissance des caractéristique physiques du collecteur, en vue de choisir la solution d'équipement la plus efficiente.

Parmi ces caractéristiques, le trait le plus important est représenté par la détermination de la porosité efficace, qui influence directement la capacité d'emmagasinage au de cession.

En vue de préciser ces caractéristiques, on a élaboré un "programme optimum" d'investigation, en utilisant de différentes combinaisons de méthodes géophysiques propres aux formations carbonatées. On a pris pour exemple l'expérience de l'investigation géophysique concernant les hydrocarbures ODPT (1972).

Notre programme d'investigation géophysique a inclut:
- des méthodes électriques;
- des méthodes radioactives;
- d'autres méthodes.

Les analyses pétrographiques des fragments de roche ont mis en évidence, dans l'intervalle 120-420 m, des formations carbonatées constituées de deux composants minéralogiques ($CaCO_3$ et $MgCO_3$), dépourvues d'argiles, à porosité secondaire (fissures, cavernes) bien développée.

On a choisi les méthodes géophysiques d'investigation en tenant compte des conditions géologiques et hydrogéologiques rencontrées. On va présenter brièvement les caractéristiques de ces méthodes et les paramètres déterminés.

3.1 Méthodes électriques

Pour une détermination correcte de la résistivité apparente on a utilisé le carottage électrique inductif en corrélation avec le latérologue. On a obtenu à la fois la courbe du potentiel naturel (P.S.). Les diagraphies nous ont offert un premier indice sur le fluide de la formation carbonatée et sur la présence des zones à porosité accentuée.

3.2 Méthodes radiométriques

On utilise ces méthodes pour détermines des éléments qui caractérisent les collecteurs. On acquient ainse des informations sur la corrélation des formations, la détermination de la lithologie, la détermination des propriétés physiques (porosité initiale et secondaire, densité etc.).

Parmi les méthodes radiométriques frèquemment utilisées pour l'investigation du trou de forage, on a employé les méthodes gama naturel (γ), gama-gama (γ-γ - carottage de densité) et neutron-neutron (n - n).

3.2.1 Carottage gama naturel

On l'utilise pour mesurer la radioactivité naturelle des formations traversées. La courbe obtenue sert pour la comparaison avec les autres courbes de carottage radioactif.

3.2.2 *Carottage gama-gama (compensé)*

On l'utilise pour mesurer le paramètre de densité des formations, par l'intermède d'une émission de radiations gama provenues d'une source radioactive. Ce type de carottage représente l'une des méthodes principales de détermination des valeurs de la porosité de la formation investiguée.

3.2.3 *Carottage neutron-neutron*

On utilise une source émettrice de neutrons rapides, tandis que les diagraphies enregistrées représentent l'effet de relantissement des neutrons à la suite de la collision avec les atoms de hydrogène des roches.

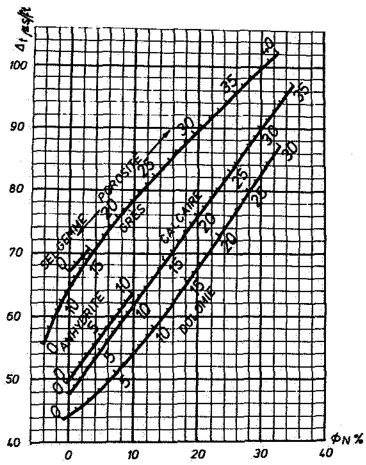
La méthode du carottage neutron-neutron met en valeur les faits suivants:
- la détermination de la porosité des formations qui manquent d'argile, surtout dans des puits non-tubés, forrés par des fluides doux;
- l'identification de la lithologie associée à d'autres carottages de porosité (carottage gama-gama).

Cette méthode se caractérise par une haute précision, car la diagraphie n'est pas influencée par la radioactivité naturelle on par celle gama émise par la course de neutrons, fait qui engendre un paramètre de porosité bien réel comme dimension.

3.3 D'autres méthodes

On a utilisé aussi le carottage acoustique de vitesse compensé, la pendagemétrie et la diagraphie du chlore.

3.3.1 Carottage acoustique de vitesse compensé



Cette diagraphie utilise les propriétés elastiques des formations de propagation des ondes ultra acoustiques pour déterminer le paramètre de porosité.

Ce type de carottage s'associe au carottage neutron-neutron (Figure 3) et offre des donées sur la lithologie et la porosité secondaire ODPT (1973).

Figure 3 Détermination de la porosité et la lithologie des carottages acoustique et de neutrons

3.3.2 Pendagemétrie

La détermination du sens et du pendage des couches d'argiles et anhydrites qui surmontent la formation carbonatée est tout à fait importante pour notre étude, car celles-ci protégent ou non la couche aquifère douce contre l'effet inattendu des eaux salée à travers le temps.

Ainsi on a effectué la pendagemétrie sur l'intervalle 94-112 m. A la suite de l'utilisation de cette méthode on a déterminé exactement la pendage et le sens des couches traversées aussi bien que la déviation du trou de forage.

3.3.3 *Diagraphie du chlore*

On a éffectué la diagraphie du chlore en vue de mettre en évidence une intrusion d'eau salée marine on des zones locales de pénétration des eaux salée dans les conditions d'une exploitation intensive de la couche aquifère La méthode consiste en l'activation du chlore (dans l'acception de sa présence dans l'eau comme chlorures) comme source de neutrons et l'enregistrement de son activité.

4 Interprétation des résultats de l'investigation géophysique

Les diagraphies géophysiques ainsi obtenues contiennent des informations pétrographiques et des données sur les propriétés physiques du collecteur et du fluide qu'y circule.

Selon la colonne lithologique reconstituée suivant les fragments de roche et les données fournies par l'investigation géophysique, on note les formations suivantes:

0 - 13,0 m	sable gris à fragments de coquilles (Quaternaire);
13 - 15,0 m	blocs de calcaire lumachellique, blanc-jaunâtre dans une matrice argileuse jaunâtre (Sarmatien);
15 - 56,8 m	craie blanche et rose (Crétacé supérieur - Sénonien);
56,8 - 61,0 m	grés calcaire verdâatre, glauconitique (Crétacé moyen-Albien);
61 - 121,0 m	alternance de calcaire d'eau douce, anhydrites rose-jaunâatres et argiles et marnes rouges et vertes (Crétacé inférieur - Purbekien - Wealdien);
121 - 420,0 m	alternance de calcaires jaunâtres-blanchâtres, calcaires dolomitiques et dolomies grisâtres, dures à structure saccharoide, fissurées, karstiques (Jurassique supérieur)

L'analyse des diagraphies du carottage électrique inductif en corrélation avec le latérologue et la courbe du potentiel naturel démontre l'existence des formations surtout carbonatées, très fissurées et karstifiées, surmontées dans l'intervalle 85-121 m par un paquet d'argiles et marnes.

Les valeurs élevées de la résistivité indiquent que par ces fissures et cavernes-là circule de l'eau à teneur diminuée en sels, fait confirmé aussi par les analyses physico-chimiques effectuées sur des échantillons d'eau prélevés à l'aide de l'installation PGAC.

La diagraphie gama naturel, par les valeurs diminuées de la radioactivité naturelle 10-15 u API, met en évidence pour l'intervalle 121-420 m, des formations dépourvues de minéraux argileux, ce qui confirme en outre la présence des formations carbonatées.
La carottage de densité gama-gama a mis en valeur des formations à porosité intergranulaire, à valeur très réduites, ce qui leur confère le caractère impermèable. La porosité secondaire de celles-ci créée par le système de fissures et creux, mène à des valeurs élevées de la porosité totale, déterminée par les carottages neutron-neutron et acoustique de vitesse compensé.

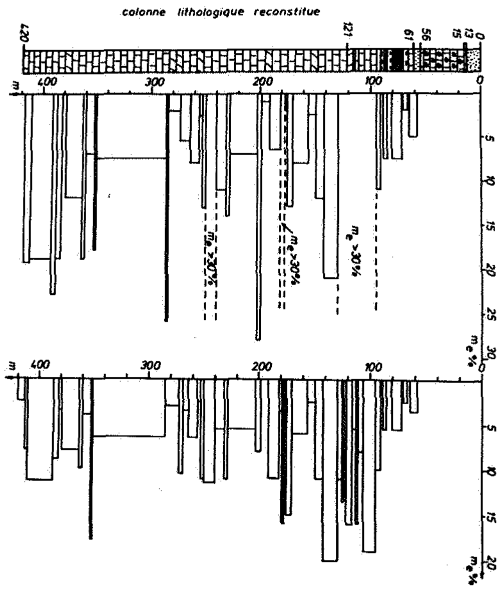


Figure 4 Variation de la porosité efficace par rapport à la profondeur
a) selon le carottage acoustique de vitesse compensé
b) selon le carottage neutron-neutron

La porosité secondaire de ces formations carbonatées est dûe aux exondations répétées durant le Jurassique à la suite des mouvements tectoniques qui ont affecté la zone. Dans la Figure 4 on présente la colonne lithologique et la distribution verticale de la porosité efficace. En considérant les valeurs trés réduites de la porosité primaire, l'interprétation des carottages de densité, neutron-neutron et acoustique, on a déterminé la valeur de la porosité efficace, en utilisant la relation ODPT (1977):

$$\phi_{se} = \phi_{N-D} - \phi_{S}$$

ou : ϕ_{se} - porosité secondaire

ϕ_{N-D} - porosité déterminée par la diagraphie neutron-neutron et gama-gama

ϕ_{S} - correction de porosité

En considérant aussi les carottages neutron-neutron et acoustique de vitesse compensé, on a déterminé assez corectement la lithologie et on a établi que les intervalles à porosité totale diminuée correspondent aux roches dolomitiques, tandis que ceux à porosité totale élevée correspondent aux calcaires.

On a effectué la pendagemétrie à fin d'éclaircir certains aspects de la dynamique des eaux souterraines douces du système aquifère investigué aussi bien que le rôle d'écran joué par la formation argilo-marneuse à calcaires et anhydrites du toit du complexe carbonatique, Băncilă (1969) L'interprétation de ces données-là indique que les dépôts présentent un pendage de 5-36^{o} vers la N à la base et vers l'E à la partie supérieure et des variations du sens entre W-E et N-S, ce qui montre soit une discordance angulaire soit un dérangement tectonique. Il faut remarquer quand même que l'investigation géophysique a mis précisément en évidence le rôle d'écran des argiles et des marnes du toit, en assurant la protection du gisement d'eau douce contre l'invasion des eaux salée marines durant l'exploitation.

A fin de détecter les zones par lesquelles l'eau salée de la mer pourrait y pénétrer, dans les conditions de l'exploitation intense de la couche aquifère, on a effectué la diagraphie du chlore en stimulent l'activité de l'ion Cl^{-} par une source de neutrons. Par la superposition de la courbe de l'activité de l'ion Cl^{-} et la courbe gama naturel, on a démontré qu'il est impossible que les eaux salée de la mer pénétrent jusqu'à la profondeur de 420 m dans les conditions des débits exploités, Simionaş (1974).

Conclusions

La corroboration des données obtenues de l'investigation géophysique avec les analyses pétrographiques et micropaléontologiques des fragments de roches, avec les données sur la composition lithologique et la tectonique de la zone (de la littérature spécialisée) et avec les données obtenues des forages situés dans des zones avoisinantes, a conduit à :

- la détermination exacte de la colonne lithostratigraphique, de la limite entre les formations et leur épaisseur réelle, aussi bien que des éléments de tectonique;
- la détermination des valeurs du sens et du pendage des dépôts argilo-marneux du toit de la formation carbonatée aussi bien que leur rôle dans la protection du gisement d'eau douce;
- la détermination de la porosité efficace du complexe carbonaté, élément principal de la dynamique des eaux souterraines et la détermination exacte des zones intensément fissurées;
- le choix de la plus économique et la plus efficace solution d'equipement du trou de forage en vue de tester la couche aquifére;
- l'élaboration des "programmes optimums d'investigation géophysique" en tenant compte des besoins pour tous les types de formations à porosité interstitielle ou des fissures.

Bibliographie

Băncilă, I. 1974. Asupra prezenţei unei formaţiuni gipsifere Purbekian-Wealdiene în regiunea Feteşti-Constanţa. St. cercet. Geol. Geof. Geograf., Ser. Geol., 18, 1, Edit. Acad. RSR Bucureşti, pp. 115-125

Mutihac, V. et Ionesi, L. 1973. Geologia României. Ed. Teh. Bucureşti pp. 88-105

ODPT 1972. Indrumător pentru perfecţionarea pregătirii profesionale a cadrelor superioare în probleme de geofizică de sondă. ODPT, Bucureşti, 329 pp.

Simionaş, St. et Simionaş, C. I. 1974. Asupra limitei apă dulce - apă sărată în zona Constanţa-Mamaia. Studii de hidrogeologie, vol. XIV I. M. H. Bucureşti, pp. 75-90

DETERMINATION OF HYDROGEOLOGICAL PARAMETERS AND AQUEOUS-PHYSICAL CHARACTERISTICS OF ROCKS ON THE RESULTS OF GEOPHYSICAL OBSERVATIONS

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Abstract

For a substantiated mapping of hydrogeological parameters and indices of rocks it is necessary to have a continuous information of their space variations. It leads to large expence, even with the use of traditional methods of studies (drilling and sampling of wells, pumping-test works). An application of geophysical methods for investigation is considered to be more perspective.

An effective complex of geophysical methods, being an integral part of hydrogeological and engineering-geologic studies performed for the purpose of land reclamation and in groundwater exploration, has been developed and widely used in the U.S.S.R.

A complex of geophysical mehods enables an estimation of the following phenomena to be made (on the basis of empirical correlations):

a) permeability coefficient or hydraulic conductivity of the productive aquifers;
b) permeability coefficient of sandy-argillaceous rocks in the zone of aeration;
c) water yield of sandy-gravel deposits under the conditions of unconfined aquifers;
d) groundwater mineralization and the degree of rock salinization in the zone of aeration;

e) depth of the first from the earth's surface unconfined aquifer; groundwater flow rate;
f) granulometric composition of sandy-argillaceous rocks, as well as a number of engineering-geological properties.

Error limits of the quantitative evaluation of the parameters vary from 10 to 30%.

1 Introduction

For a substantiated mapping of hydrogeological parameters and engineering-geological properties of rocks it is necessary to have a continuous information on their variations in space. It leads to large expences and a long time of their production even with the use of traditional methods for studies (drilling and sampling of wells, pumping-tests works).

An application of mobile and less labour-consuming geophysical methods of investigation is considered to be more perspective.

An effective complex of field geophysical methods, being an integral part of geological and hydrogeological studies, when carrying out hydrogeological and engineering-geological surveys on the 1:50 000 scale for the purpose of land reclamation and fresh groundwater exploration, has been developed and widely used in the U.S.S.R.

When developing the procedure we have taken into consideration that the optimal results of geophysical works can be only those which are directly taken into account and used in preparing and compilling the final maps and sections, which contain a quantitative information on the rock properties. In this connection, the main peculiarity of the procedure developed is a quantitative geological and hydrogeological interpretation of the geophysical data with the aim of estimating the values of hydrogeological and engineering-geological parameters and aqueous-physical properties of rocks.

A quantitative geological and hydrogeological interpretation of the geophysical data is based on compilling and applying the empirical correlations between the geophysical parameters on the one hand, and between the hydrogeological parameters and the rock properties on the other hand. The correlations are found on the basis of comparing the interpretation results of parametric soundings around the wells and other mine openings with the data on their hydrogeological and engineering-geological testing.

The character of the relationships and the procedure of their compiling can be significantly different depending upon the geological and hydrogeological conditions of the area under survey or the type of a groundwater development area.

However, under various geological and hydrogeological conditions of the area a complex of geophysical methods allows one to assess:

- the permeability coefficient or hydraulic conductivity of productive or recharge aquifers composed by loosely fragmental, sandy-argillaceous, fractured, and fissured-karst rocks;
- the permeability coefficient of sandy-argillaceous rocks in the zone of aeration;
- the water yield of sandy-gravel deposits under the conditions of unconfined aquifers;
- the groundwater mineralization and the degree of rock salinization in the zone of the aeration;
- the granulometric composition of sandy-argillaceous rocks (content of separate fractions, plasticity index, specific surface) as well as a number of engineering-geological properties (moisture, porosity).

A solution of the above problems is made by applying the resistance method and the method of induced polarization of electrical prospecting, and the refraction method of seismic prospecting.

While applying the above methods the main parameters are:

- specific electrical resistance of rocks (ρ);
- polarization of rocks (η);
- relative values of polarization or conductivity of induced polarization ($\eta^{*} = \frac{\eta}{\rho}$);
- velocities of longitidinal and transversal refracted waves (V_p and V_s).

Under natural conditions the values of the above parameters are dependent upon a change in a sufficiently wide spectrum of hydrogeological and engineering-geological properties and indices of rocks.

At present, a sufficient amount of data has been accumulated, which enabled the most stable correlations to be found out between the geophysical and geological-hydrogeological parameters and indices of rocks and their complexes.

2 Main correlations in solving the geological and hydrogeological problems

2.1 Determination of the rock permeability of aquifers

This determination is made by using the method of induced potentials under the conditions of sandy-gravel deposits and, in addition, the refraction method, when an aquifer is represented by fissured corbonate rocks.

The value of a specific electrical resistance of loose sandy-gravel deposits tends to decrease with decreasing the permeability of rocks. Accordingly, the correlations $\rho = F(Kf)$ can be made. However, it is necessary to note that in a general case the prescribed interrelation is unstable above all, due to a determining effect on the value, ρ of a dissolved solids content of groundwater. Finding the correlations between the relative value of polarization and the permeability coefficient of rocks is more effective. For the aquifers represented by sandy-gravel deposits a general regression equation is $I/Kf = -a + b\eta^{*}$, where a value of the factor "a" amounts to 0.06 and a value of the factor "b" varies depending on the groundwater mineralization from 0.126 (with the mineralization 0.5 g/l) to 0.044 (with the mineralization 4.0-5.0 g/l).

The values of the correlation coefficients of particular regression equations range from 0.92 to 0.97.

A high stability of the interrelation $\eta^{*} = F\ (Kf)$ is marked for various stratigraphic and genetic complexes of rocks.

When the aquifer thickness is slightly changeable within the work area, it is possible to find the correlation of the form $\eta^{*} = F\ (Km)$, where "Km" is the permeability coefficient of an aquifer.

Mapping of the permeability of fractured collectors is carried out using the correlations of the form $\rho = F\ (Kf)$, $\eta^{*} = F\ (Kf)$, V_p and $V_s = F\ (Kf)$.

A necessity for finding several correlations is dictated, above all, by the fact that an increase of the water contained in fissured rocks can be related both to the increase in their degree of jointing and the decrease in the content of a clay crack filler.

In the first case, when Kf increases, the value of a specific electrical resistance decreases and a relative polarization increases.

For carbonate deposits the regression equations are obtained as follows:

$$\ln Kf = 17.6 - 3.06 \ln \rho \text{ , correlation coefficient 0.81.}$$

$$\ln Kf = 2.5 + 0.3 \ln \eta^{*}\text{, correlation coefficient 0.94.}$$

In the second case, when Kf increases, the value " ρ "increases and a relative polarization decreases. The regression equations for the same area are as follows:

$$\ln Kf = 1.15 + 0.5 \ln \rho \text{ , correlation coefficient 0.84.}$$

$$\ln Kf = 2.3 - 3.1 \ln \eta^{*}\text{, correlation coefficient 0.95.}$$

The correlation of a boundary velocity of longitudinal and transversal elastic waves refracted on the top of carbonate rocks to the permeability coefficient in both cases is single-valued and characterized by the equations:

$$\ln Kf = 19.1 - 2.0 V_p\text{, correlation coefficient 0.85.}$$

$$\ln Kf = 1.8 - 0.6 V_s\text{, correlation coefficient 0.80.}$$

The permeability of fractured collectors can be unequivocally estimated using the above equations.

Finding the above correlations is made by comparing the results of the data interpretation on the parametric soundings carried out near the groups of wells, in which the pumping-test works have been carried out. An error of estimating the values "Kf" ranges from 10% to 30% according to the results if the geophysical works.

2.2 Determination of the permeability coefficient of the rocks in the zone of aeration

This determination can be done using the interrelations $\eta^{*} = F$ (Kf). In this case, it is necessary to take into account a significant dependence of polarization and a specific electrical resistance of rocks on their moisture content and a dissolved solids content of interstitial water. In this connection, the correlation graphs $\eta^* = F$ (Kf) are plotted separately for the aquifers differing in the values of moisture and the degree of rock salinization.

The following regression equations obtained for the areas of Zavolzhje can be given as an example:

$\ln Kf = -1.82 - 2.85 \ln \eta^*$ (the degree of rock salinization is less than 0.1%, moisture is more than 15%).

$\ln Kf = 0.94 - 3.1 \ln \eta^*$ (the degree of rock salinization is 1.0-2%, moisture is more than 15%).

2.3 Estimation of the value of the gravitational water yield of rocks

This estimation is based on a close dependence of the parameters of the method for induced polarization upon the specific surface and porosity of sandy-argillaceous rocks.

For the areas of the Nechernozemnai zone of the R.S.F.S.R. the equation of the correlation graph obtained is as follows:

$\ln \mu = -4.6 + 1.21 \ln \eta^*$, correlation coefficient 0.94.

2.4 Estimation of the groundwater mineralization (M) and the degree of rock salinization (C) in the zone of aeration

This estimation is made by the value of a specific electrical resistance. The relationship $\rho = F$ (M) at a double logarithmic scale is a straight line for the gravel-pebble deposits. An increase in the degree of the rock dispersivity results in changing the angular coefficient of the straight line in the field of small mineralizations. The dependence graphs ρ on M are characterized by the presence of two straight line segments, one of which (in the field of large mineralization) is the same for all the types of rocks. The example of the regression equations of the graphs obtained may be:

$\ln M = 3.89 - 0.90 \ln \rho$ (gravel-pebble deposits);

$\ln M = 3.89 - 0.90 \ln \rho$ (sands, water mineralization is more than 2.0 g/1);

$\ln M = 21.5 - 5 \ln \rho$ (sands, water mineralization is less than 2.0 g/1).

An error of determining the groundwater mineralization with the use of the above equations will not exeed 10%. The degree of the rock salinization has a determining influence on the value of a specific electrical resistance of rocks in the zone of aeration. The effect of the moisture and a lithological composition of the rocks on the value, ρ , is a changeable factor. In this connection, the relationships $\rho = F\ (C)$ are plotted separately for the rocks of a different lithological composition and individual intervals of their moisture. The following equations of the graphs of the interrelation under consideration are given as an example:

$\ln C = 1.36 - 0.8 \ln \rho$ (loams, moisture is less than 10%);

$\ln C = 1.25 - 0.96 \ln \rho$ (loams, moisture is more than 10%).

The correlation coefficient of the first equation is 0.72. The correlation coefficient of the second equation is 0.89. An error of determining the degree of rock salinization does not exceed 15-30% with allowance for all the interfering factors.

2.5 Determination of the sandy-argillaceous rock composition

A rock is usually related to one or another lithological difference on the basis of its granulometric analysis and studying the plastiscity indices and a specific surface.

Table 1 The basic regression equations

Name of correlation	Regression equation	Amount of data	Correlation coefficient
Specific electrical resistance (ρ) and specific surface (S)	$\ln S=8.29-052\ln \rho$	34	0.82
Relative polarization (η^*) and specific surface (S)	$\ln S=5.01+1.67\ln \eta^*$	34	0.97
Particle content in diameter of less than 0.005 mm (γ_1)and η^*	$\ln\gamma_1=2.5+0.68\ln \eta^*$	120	0.78
Particle content in diameter of less than 0.01 mm (γ_2) and	$\ln\gamma_2=3.1+0.6\ln \eta^*$	120	0.89
Plasticity index(Π) and η^*	$\ln\Pi =3.39+0.36\ln \eta^*$	120	0.91

In this connection, finding particular correlations is needed to characterize a lithological composition of a rock on the results of geophysical investigations.
The basic regression equations obtained from comparing the results of the curve interpretation of parametric soundings to individual rock indices are given in table 1.

3 Porosity of rocks

Porosity of rocks has an important influence on the values of a specific electrical resistance and polarization of rocks. However, in most cases, a certain correlation between the electrical parameters of rocks and their porosity can be obtained only for coherent soils. Thus, the correlation between the relative polarization (η^*) and porosity (Π) has been attained for morainic sandy loams and loams. The correlation equations are $\ln \Pi = 4.61 + 0.72 \ln \eta^*$, the amount of initial data is 44, the correlation coefficient is 0.86. A range of changing the rock porosity is 25-53%.
Such basic empirical correlations are used for the geological and hydrogeological interpretation of the geophysical results. However, despite the fact that in each particular case we can use a double correlation, a multifactor analysis is expedient to apply with the aim of increasing a reliability of estimation of the hydrogeological parameters and rock indices.
Thus, the equation has been obtained to estimate the groundwater mineralization:

$$\ln (M + 1.7) = 3.76 - 0.24 \rho - 0.013 \eta^* + 0.004 \rho^2,$$

the amount of data is 136, the correlation coefficient is 0.91. The correlation equation for assessing the rock clayness has a form as:

$$\ln \Gamma_1 = 21.95 - 14.1 \eta + 1.38 \eta^2 + 0.78 \ln \rho + 33.1$$
$$\ln (\eta + 1.93) + 0.69 \ln (\eta^* + 1.85)$$

The amount of data is 201, the correlation coefficient amounts to 0.81. The permeability of sandy-gravel water-bearing deposits can be estimated by the equation:

$$\ln (Kf + 1.98) = 4.09 + 0.12 \eta^* - 0.78 - 1.83 (\eta^* + 1.5) +$$
$$2.03 \ln (\eta + 1.8) - 0.38 \ln \rho$$

The amount of initial data is 40, the correlation coefficient amounts to 0.79.

Combining the traditional and geophysical methods for studies allows the geological and geophysical sections along the profiles and maps of lithological-stratigraphical rock complexes, maps of the groundwater mineralization, maps of the degree of rock salinization in the zone of aeration, maps of hydraulic conductivity of particular aquifers and their complexes, maps of the rock permeability in the zone of aeration to be constructed using the results of the combined works.

The maps showing individual indices are contructed in the gradations meeting the requirements for the survey and prospecting works for water.

In conclusion, one of the important points of the procedure under consideration should be emphasized.

In case of the geological and hydrogeological interpretation of geophysical data based on the use of the correlations. special demands are made on wells applied to parametric measurements, non-observance of which can lead to the negative results.

The parametric wells by its purpose can be divided into mapping hydrogeological and engineering-geological wells and wells for studying hydrodynamic parameters.

The first group wells enable the initial data to be obtained to estimate the indices of a granulometric composition of porosity, moisture, and a specific surface.

The requirements for their sampling are as follows:

a) rock samples designed for a granulomatric analysis and an analysis of aqueous extract must be taken by a furrow method; sampling intervals are chosen for uniform (visually) beds and corrected by the results of a quàntitative interpretation of parametric curves;
b) sampling for moisture is carried out over the thickness of the aeration zone with a necessary decrease of sampling intervals in its lower part;
c) monolithes are taken from each of lithological horizons to determine the engineering-geological properties of rocks.

The requirements for wells or more exactly for groups of wells and also pits, on which the results of pumping-test studies are the basis of making the correlations of geophysical parameters and the permeability coefficient of rocks, are presented as follows:

a) studies in situ on the permeability coefficient of aquifers and rocks in the zone of aeration must provide for obtaining the value Kf(Km) with a high degree of reliability, they being carried ouy by a single procedure over the whole area of works to obtain the compared results;
b) under conditions of a layered section the filter intervals established for a total sampling of an aquifer complex must correspond to the depth interval of a homogeneous aquifer distinguished on the curves of induced polarization;
c) in the presence of a persistent aquifer clearly distinguished on the curves of induced polarization, a construction of the central well must be as perfect as possible.

The well logging is needed.

A production application of the procedure developed has showed its high efficiency in the course of hydrogeological and engineering-geological prospecting-survey and exploration in many regions of the U.S.S.R.

IDENTIFICATION OF GROUNDWATER FLOW AND AQUIFER HETEROGENEITY BY GEOTHERMOMETRY

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Abstract

In the last decade much work has been done to numerically simulate subsurface transport of contaminants and heat connected with groundwater flow. Several simulators have been put forward. They may be judged by their completeness in taking account of the physics and chemistry pertaining to porous media flow domains, and by their numerical reliability. Prediction of transport, however, cannot be improved with even the most elaborated simulator if the physical and chemical parameters of the flow domain are insufficiently known. With regard to subsurface transport there is a definite need to identify variations of hydraulic conductivity within the flow domain. These variations may be identified by measurement of temperature profiles in observation wells, if groundwater passing these wells is labeled thermally. Such situations arise at wells where water is injected at temperatures deviating from ambient underground temperatures, at waste disposal sites in which percolating water has become heated and at heat wells. Several measured temperature profiles clearly indicate variation of hydraulic conductivities along the vertical. These observations give rise to the question whether the incorporation of non-microscopic hydrodynamic or thermal dispersion in numerical simulators truly accounts for transport phenomena in subsurface porous media.

1 Introduction

One of the most outstanding problems to scientists and engineers dealing with the subsurface environment is to efficiently identify and predict man-induced changes in its physical and chemical state. The most important changes pertain to such variables as groundwater piezometric head, concentration of dissolved constituents in groundwater, subsurface temperature and stress.

Changes are brought about by exploitation of the Earth's natural resources and by underground injection. The latter activity aims not only at disposal of waste products, but also at temporary storage of fresh water, gas, compressed air or heat. Furthermore there is an increasing flux of contaminants from located and distributed sources at the ground surface into the underground. Most contaminants are dissolved in water, recharging groundwater flow systems; some contaminants are immiscible with water and occur as a distinct phase.

Subsurface groundwater flow systems as parts of the hydrologic cycle may have a pronounced effect upon the Earth's subsurface temperature field (Van Dalfsen, 1981). Compared to the hypothetical case of no groundwater flow, which means only conduction of the Earth's internally generated heat to the ground surface, subsurface temperatures are lower in areas with downward groundwater fluxes (recharge areas) and higher in areas with upward groundwater fluxes (drainage areas). The influence of heat transport by flowing groundwater upon the subsurface temperature field is clearly demonstrated by the relatively low temperatures at 100 m depth below the Veluwe and Utrechtse Heuvelrug recharge areas in The Netherlands (Figure 1). From these data it appears that subsurface temperature in a groundwater flow domain may be regarded as a variable depending upon flow and thermal history of the groundwater. Conversely, deviation in measured subsurface temperatures may be indicative with respect to spatial variation in groundwater flow. In this paper it is exemplified that subsurface temperature deviations may be used to identify local variations in groundwater flow and hydraulic conductivity.

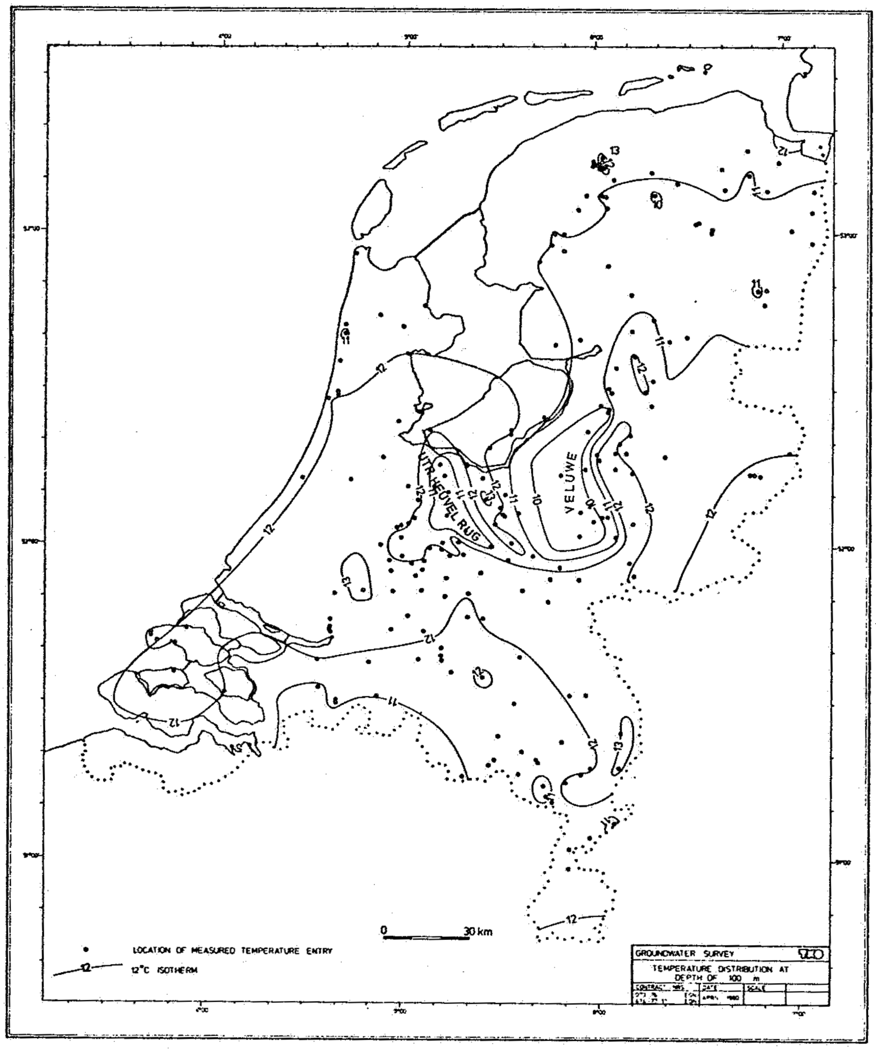

Figure 1 Isotherms at 100 m depth in The Netherlands

2 Injection at temperatures different from ambient subsurface temperatures

Figure 2 shows two temperature profiles measured by lowering a platinum resistance temperature probe in an observation well (PVC tube of 2,5 cm diameter with a 1 m screen at its lower end). This observation well is located 5 m aside an injection well screened between 108 m and 123 m depth. Both wells are involved in a pilot test of the Amsterdam Water Supply to store fresh surface water in a (semi)-confined aquifer between 94 m and 141 m depth with salt groundwater.
The dotted temperature profile represents undisturbed temperatures, measured a few days before the start of injection. Below 35 m depth it shows an increase of temperature with depth, which fits quite well into the large scale subsurface temperature field delineated by Van Dalfsen (1981). The temperature profile to the left, with linear interpolation between measured temperatures at depth increment of 1 m, was obtained after about 40 h injection of cold (4°C) water at a volumetric flow rate of 20 m^3/h.
Within the aquifer the latter temperature profile identifies three levels at which the temperature decrease attained a (relative) maximum These levels are at 109 m, 113 m and at 117 m depth. Obviously these depths mark the presence of three distinct flow paths, through which the temperature front proceeds faster than at the levels immediately above and below. As the velocity of the temperature front is proportional to that of the injected water, we have also identified three flow-paths with relatively high hydraulic conductivities.
It appears that in case of injection at temperatures different from ambient subsurface temperatures, hydraulic layering can be identified quite easily by recording temperature profiles in nearby observation wells.

3 Heated percolate from a waste disposal site in a groundwater flow system

In the last decades large volumes of waste materials have been accumulated in waste disposal sites. If there is a hydraulic contact between

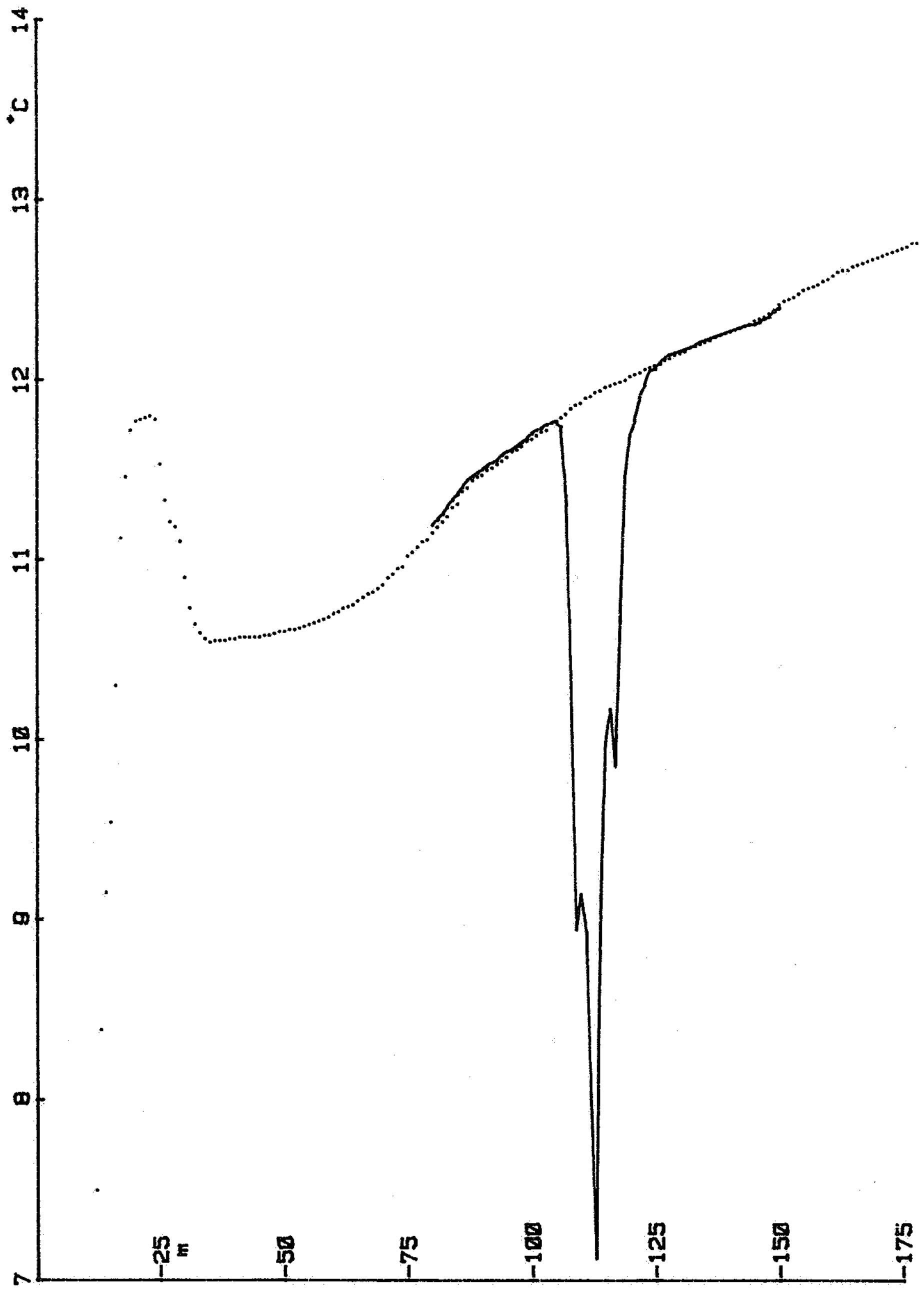

Figure 2 Subsurface temperature profiles before and after start of cold water injection through a nearby well

a waste disposal site and its underground, contaminated percolate may enter a groundwater flow system. In many cases this groundwater flow system is (partly) due to an elevated water table in the waste disposal site, compared to that in its surroundings. Apart from any change in water table elevation, the groundwater flow field may be affected by density differences between natural groundwater and heavily contaminated percolate out of the waste disposal site. In such a case an unstable groundwater stratification gives rise to vorticity in the subsurface flow domain.

An important phenomenon in the context of this paper is the heat, which is generated inside a waste disposal site through (bio)chemical decomposition of waste materials. Figure 3 shows three temperature profiles, measured in spring 1980 for the National Institute for Water Supply of The Netherlands at the waste disposal site Noordwijk. The profiles are designated by the codes P1, P2 and P3, which have also been assigned to the locations of the observation wells in which they were measured. P1 is a normal profile, whereas P2 and P3 show deviating high temperatures, which attain their maximum deviation at depths of 13 m and 31 m respectively.

These deviations are ultimately due to heat generation inside the waste disposal site. At P2 the maximum temperature deviation at 13 m depth occurs about 2 m above the bottom of the site. The variation of the temperature deviation along the vertical through the site depends on a large number of factors including heat loss to the atmosphere, percolation of infiltrated precipitation and spatial and temporal variation in heat generation. Anyway, percolate leaving the site to enter the existing groundwater flow system has become labeled thermally. In view of this thermal labeling, the temperature maximum at 31 m depth at P 3 identifies a flowpath for heated percolate passing this observation well. Furthermore this is the path which has become more heated than any other path passing P3. Therefore one may interpret this path as a relatively high hydraulic conductivity path or (and) as a path originating from a relatively high temperature part inside the waste disposal site.

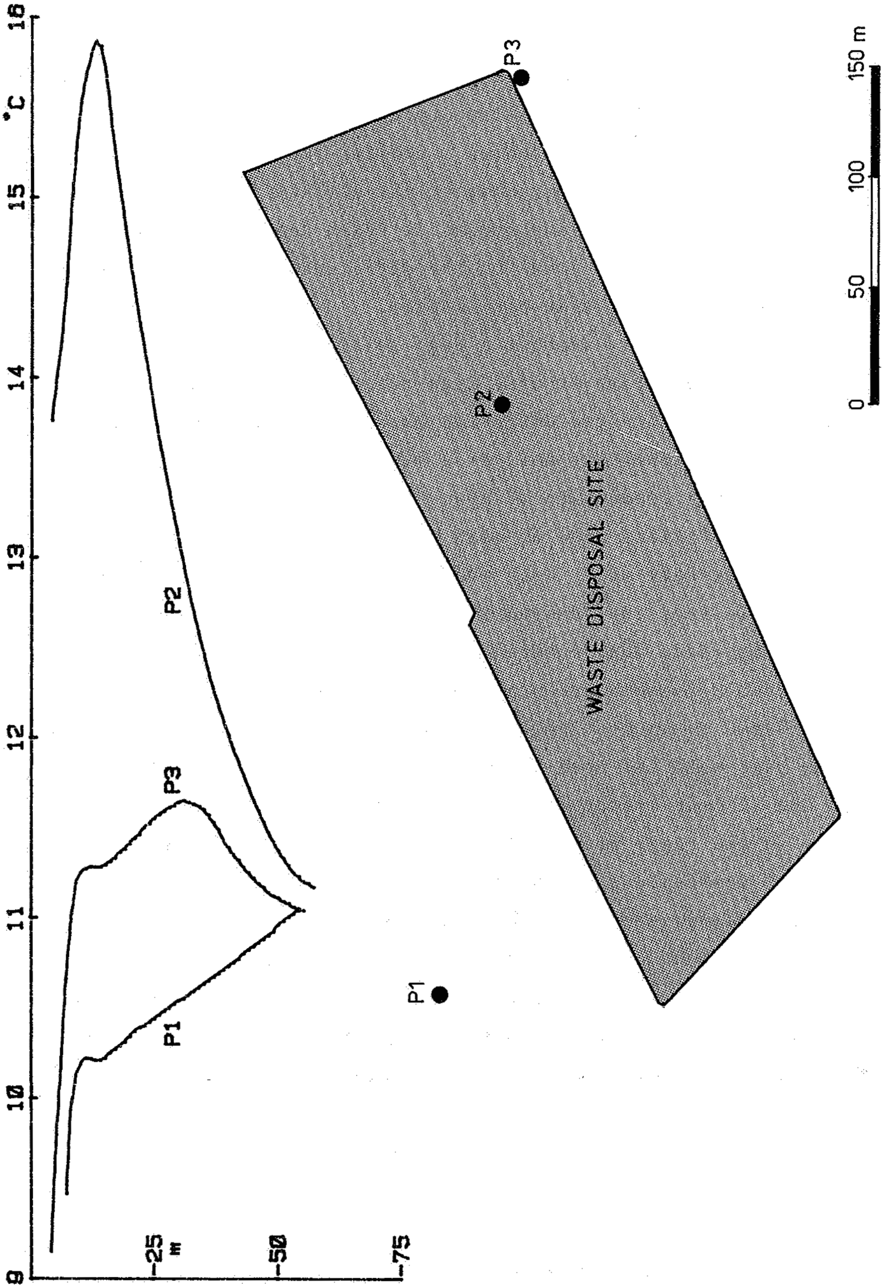

Figure 3 Subsurface temperature profiles at a waste disposal site

4 Heat well in unconsolidated sediments

At a number of places in The Netherlands heat wells are in active operation to recover heat from shallow depths. A heat well is a borehole with a tubing system, through which a cooling fluid circulates. With respect to its subsurface surroundings the cooling fluid acts as a heat sink, receiving heat and transporting it to an heat pump. The heat flow across the tube is proportional to the difference between the temperatures on inner and outer side of the tube.
An important question is whether a heat well can sustain a sufficient heat flow for a long time or not. With respect to the heat transfer to an heat well, there are two different cases: a situation with groundwater flow passing the heat well or a situation without. In the latter case there is only conduction of heat towards the heat well, which implicates an inefficient mode of heat transfer. In case of groundwater flow passing the heat well, heat transfer towards the well is sustained much better. In fact the combination of heat well and groundwater flow acts as a subsurface heat exchanger.
Figure 4 shows a temperature profile measured in a PVC tube in a borehole, completed to an heat well. The temperature profile was measured after the heat well had been in active operation for more than three months. Due to heat extraction, temperatures are below normal which is somewhere in the range between 10°C and 11°C. More important seems the variation of temperature versus depth. This variation is tentatively ascribed to groundwater flow, whose horizontal velocity varies with depth along the heat well. In this view the high temperature peaks identify flow paths with relatively high hydraulic conductivities.
Admittedly, the above interpretation of the temperature profile is not the only one possible. The temperature variations may also be ascribed to variation in distance between PVC tube and cooling fluid or to variation of thermal conductivity versus depth.

5 Dispersion based transport simulators

Several numerical codes exist, developed to numerically simulate sub-

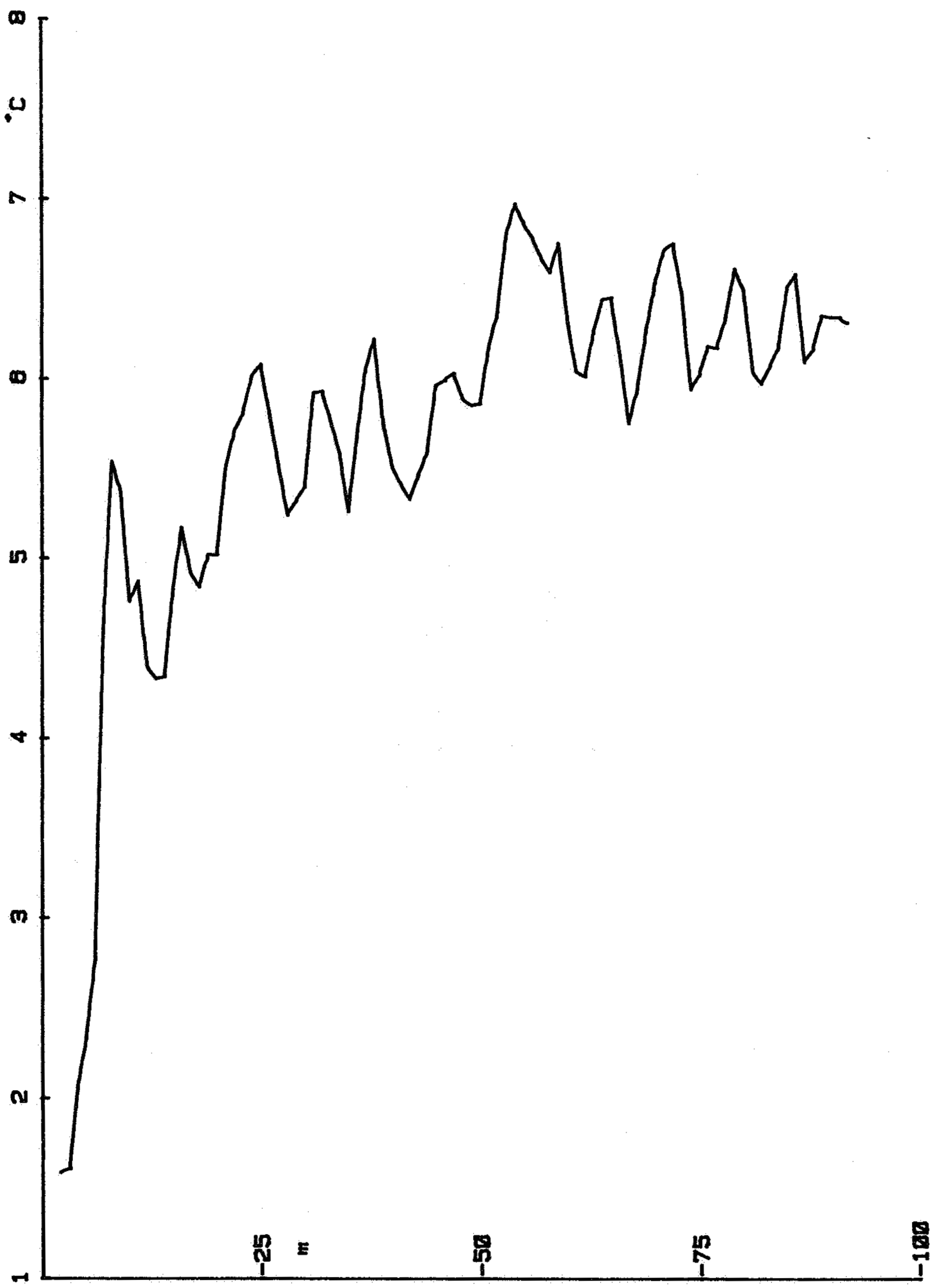

Figure 4 Subsurface temperature profiles along a heat well

surface transport of heat or constituents dissolved in groundwater. Among these, the more popular simulators are based upon the hydraulic approach (Bear, 1979). In this approach all values of both hydraulic conductivity and (point) storativity are lumped into the parameters transmissivity and aquifer storativity, respectively. Formally this is achieved by integrating hydraulic conductivity and (point) storativity along the vertical through the aquifer. The resulting groundwater flow equation is independent of the depth coordinate, and therefore also its solution in terms of groundwater velocity or front motion. Then in transport problems account is made for the undetermined variation in groundwater velocities by adding a dispersion coefficient to the thermal or solute diffusivity. This procedure has been proven to be useful with transport experiments on preparated samples on the laboratory scale.

Application of dispersion coefficients to simulate transport in aquifers is problematic. As a consequence of aquifer hydraulic heterogeneity, in this paper exemplified by the thermometrical method of investigation, part of the heat or solute is transported much more rapidly than the other part along relatively high hydraulic conductivity flow paths. Along these paths the groundwater velocity is much higher than the aquifer averaged velocity according to the hydraulic approach. To account for this variation with a dispersion based model, one has to introduce irrealistically large dispersion coefficients. Their irreality becomes manifest in the appearance of heat or solute upstream of the direction of groundwater flow.

References

Bear, J. 1979. Hydraulics of groundwater. McGraw-Hill Inc., New York, 567 pp.

Van Dalfsen, W. 1981. The shallow subsurface temperature field in The Netherlands. Groundwater Survey TNO, Delft, 105 pp.

ENVIRONMENTAL ISOTOPES, RECHARGE AND DISCHARGE AREAS AND PALAEOWATER OCCURENCE IN MADRID AQUIFER SYSTEM

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Abstract - Resumé

The Madrid aquifer system is a large ($6000 km^2$) tectonic basin filled with thick (up to almost 4km) Tertiary continental detrital sediments. An environmental isotope study of oxygen 18 (99 data), deuterium (26), tritium (18), carbon 13 (14) and carbon 14 (21) is presented, including a reassessment of literature data. The range of O-18 variation is relatively small (-7.0 to -9.2‰). Groundwaters in the recharge areas are enriched in heavy isotopes, as compared to discharge areas, indicating confinement and suggesting the presence of 'paleowaters'. This hypothesis is in agreement with the correlation found between oxygen 18 and carbon 14 contents. However no significant variation in deuterium excess is observed between ancient (discharge) and recent, tritium bearing (recharge) waters suggesting similar average atmospherica circulations during flow transit time.

Le bassin, vaste ($6000 km^2$), tectonique de Madrid est comblé par un puissant (près de 4000m) complexe détritique, tertiaire, continental, aquifère. On présente une étude des teneurs en isotopes du milieu: oxygène 18 (99 mesures), deutérium (26), tritium (18), carbone 13 (14) et carbone 14 (21) qui inclut des résultats inédits et réexamine les données de la littérature. Les teneurs en oxygène 18 varient peu mais de façon significative entre -7,0 et -9,2‰. Les eaux souterraines des zones de recharge sont systématiquement enrichies en isotopes lourds par rapport à celles des zones de décharge. Ceci confirme le confinement des circu-

lations et suggère la présence d'eaux anciennes dans la recharge. Cette hypothèse est en accord avec la corrélation qui est mise en évidence entre teneurs en oxygène 18 et en carbone 14. Toutefois, on ne relève aucune variation significative des excès en deutérium entre eaux anciennes de décharge et eaux récentes (contenant du tritium) de recharge ce qui supposerait que les circulations atmosphériques sont restées similaires sur la plage de temps correspondant au transit des eaux.

1 Introduction

Environmental isotopes provide a well-suited technique for groundwater studies. Methodology and most significant results (obtained during the past two decades were recently reviewed (Fontes, 1980; IAEA, 1981). Groundwaters from the sedimentary basin of Madrid were investigated for several years, combining isotopic studies (mainly oxygen 18 and carbon 14 analyses) with other hydrogeological methods (Sastre, 1978; Herráez et al., 1979; Lopez Vera et al., 1981; Herráez and Llamas, 1982). Conclusions from isotopic studies appeared in agreement with hypotheses on flow systems drawn from water potential distribution and classical hydrochemistry.

Carbon 14 contents of the total dissolved carbon appeared much lower in the discharge zones than in the areas considered as recharge zones. Because the ^{18}O contents of waters in the discharge zones were somewhat lower than in the recharge zones, Sastre (1978) advanced the hypothesis of a paleoclimatic effect. Lower ^{18}O contents were attributed to a colder climate during recharge episodes. This interpretation was criticised by Lopez Vera et al., (1981) who maintained that the observed differences (≃1‰) were not statistically significant. From almost the same set of data and a new evaluation of previous results Herráez and Llamas (1982) reached the conclusion that there is a small but significant shift in ^{18}O contents between discharge and recharge zones. Furthermore, these authors observed a direct trend between ^{18}O and ^{14}C contents.

For the present paper, a much greater number of ^{18}O data (99) is available. Deuterium contents were also measured on selected samples (26).

2 Hydrogeology of the Madrid Basin

The sedimentary basin of Madrid is a large ($6000 km^2$) tectonic depression filled with detrital deposits of Tertiary age. The thickness of these continental deposits is more than 3000m in the deepest parts (Figure 1). Sediments are detrital (sands, silts and clays) in the vicinity of the surrounding mountains. Evaporites and limestones occur toward the centre of the basin. Groundwaters located in detrital sediments were extensively investigated in the recent years (Llamas, 1976; Llamas and Martinez Alfaro, 1981).

Annual precipitation is about 500mm with almost no summer rains. Annual mean temperature is about 15°C (9°C in winter and 21°C in summer). Annual mean recharge over the whole basin is estimated at 60 mm a^{-1}. Groundwater circulations occur mainly through local flows from recharge areas (interfluves) to the nearest river beds. All the rivers (under natural regime) are in draining situation. The occurence of intermediate and regional flows and of stagnation zones is highly probable (Figure 1b). The aquifer system is made up of large levels of sands interbedded in clays and silts. Digital models were elaborated assuming a single, phreatic, heterogeneous and anisotropic aquifer. The horizontal permeability is low (0.1 to 0.3 m d^{-1}) and the vertical permeability is lower by two orders of magnitude. Therefore, one would expect lone residence times, greater than several millenia even in local flows of average length (Figure 1b).

3 Sampling and Isotopic Analyses

Sampling deals with small springs (21), shallow wells (5) and boreholes deeper than 50 m (73) (Figure 1a). Table 1 lists the origin of available results whilst detailed lists of data will be published elsewhere. Oxygen 18 and deuterium contents are expressed in terms of δ (or ‰) versus SMOW. Carbon 13 contents are also given in ‰ and referred to PDB standard. Carbon 14 activities are expressed in % of modern carbon (pmc). Tritium activities are given in Tritium Units (T.U.). Carbon 14 contents of total dissolved carbon (collected according to the IAEA field protocol) were not converted into radiometric 'ages'. Reasons for

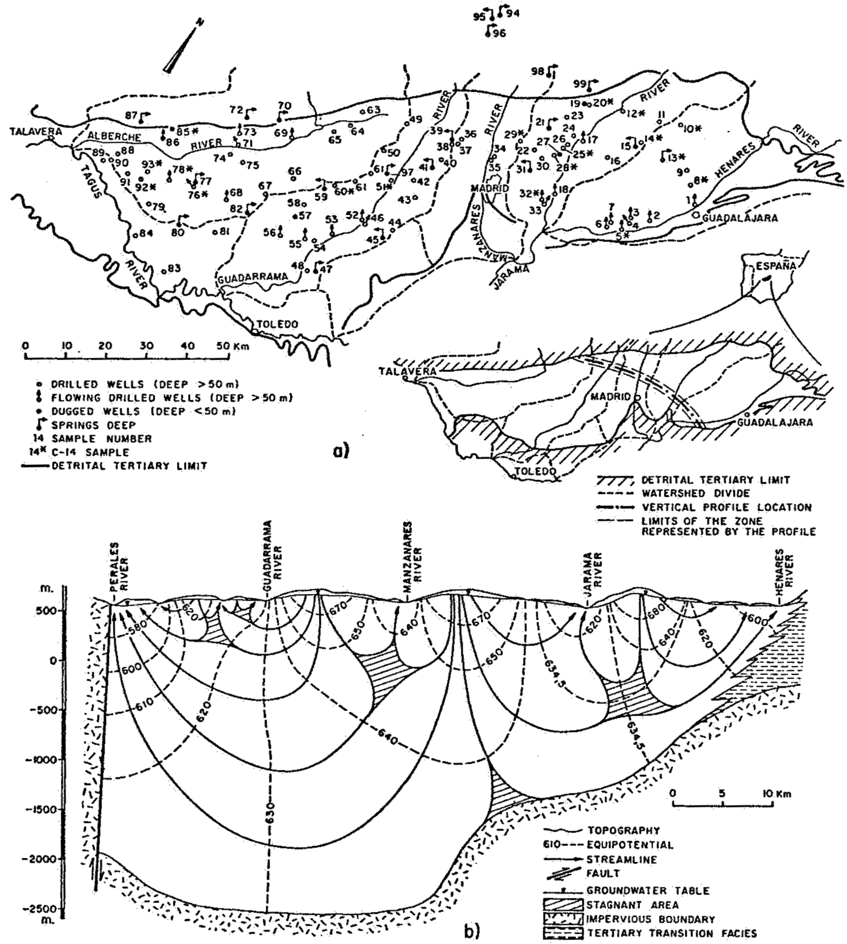


Figure 1 The sedimentary basin of Madrid

a) Location of samples in the study area

b) Groundwater flow system in a vertical profile (after Llamas et al. in Herráez and Llamas, 1982)

that are:

- the concept of 'age' is of poor significance in the case of circulating groundwaters submitted to dispersive phenomena (Fontes, 1982) and should be substituted by that of transit time distribution.
- all the samples most probably consist of mixtures of waters having a wide spectrum of transit time, due to hydrogeological conditions

(multi-layered aquifer) and to drilling and pumping conditions (several levels of withdrawal).

Table 1 Origin of available results

Reference	Laboratory	^{18}O	^{2}H	^{13}C	^{14}C	^{3}H
Sastre, 1978	Univ. Paris VI	15	-	-	-	-
Herráez et al., 1979	Univ. Tucson	13	-	2	2	-
Lopez et al., 1981	GANOP (*)	10	-	10	17	11
This work	Univ. Paris-Sud	61	26	2	2	7
Total		99	26	14	21	18

(*) Gabinete de Aplicaciones nucleares de Obras Públicas, Madrid

4 Isotopic features of the recharge

Stable isotope (^{2}H and ^{18}O) and ^{3}H contents of precipitations collected in the meteorological station from the 'Centro de Estudios Hidrográficos' (Madrid, elevation 580m) are measured from 1978 care of 'Gabinete de Aplicaciones Nucleares de Obras Públicas' (GANOP). This record gives average weight values of $\delta^{18}O = -4.6$ and $\delta^{2}H = -36.6‰$ for the period 1978-1981. Taking into consideration the season of recharge (November to March) the average values for the fraction of precipitation which infiltrates 'effective rain', becomes $\delta^{18}O = -7.3$ and $\delta^{2}H = -41.6‰$. These values were compared with stable isotope contents of spring and shallow wells (Figure 2). Springs with low flow are assumed to represent local recharge (Gonfiantini et al., 1976). This is supported by the existence of a gradient between the altitude and the ^{18}O content of spring waters. The value of -0.23‰ per 100m appears consistent with data available from the literature (Fontes, 1980, 1981). The correlation includes the representative point of the effective rain.
Assuming that the ^{18}O content of the recharge is thus evaluated through that of shallow springs, one computes the values for the recharge on the interfluve of each single subbasin. Thus, in the Henares-Jarama interfluve (800m altitude) the $\delta^{18}O$ value would be -7.8; in the Jarama-Manzanares (750m) -7.7; in the Manzanares-Guadarrama (650m) -7.5; and in the Guadarrama-Alberche (550m) -7.3.

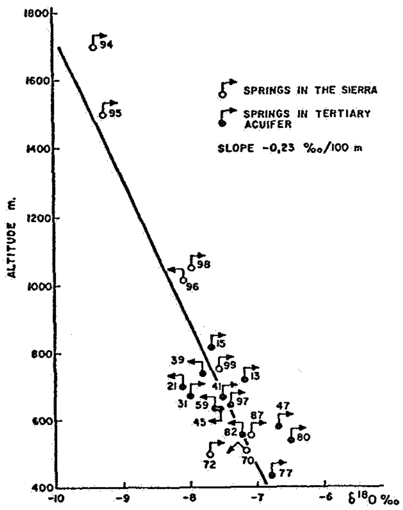


Figure 2 $\delta^{18}O$ in springs versus altitude. (All the Tertiary springs are located in recharge areas. The flow is lower than 0.1 $l.s^{-1}$)

5 Isotope contents of groundwaters

5.1 Discussion on ^{18}O contents

Over the entire basin, waters most depleted in heavy isotopes appear in discharge zones i.e. valley bottoms (Figure 3). Table 2 and Figure 4 give the distribution of ^{18}O average contents in each subbasin according to the hydrogeological conditions (recharge zone, discharge zone and transition zone).

From the results (Table 2 and Figure 4) one can point out the following:

a) Even if ^{18}O is rather homogeneous, there is a clear difference between recharge and discharge waters (the latter being depleted by about 1.1‰) despite the fact that discharge zones are, of course, located at a lower altitude.

b) The standard deviation on the average stable isotope content is reduced when recharge, transition and discharge zone, are considered separately (σ = 0.24‰ for recharge, 0.31‰ for transition and 0.25‰

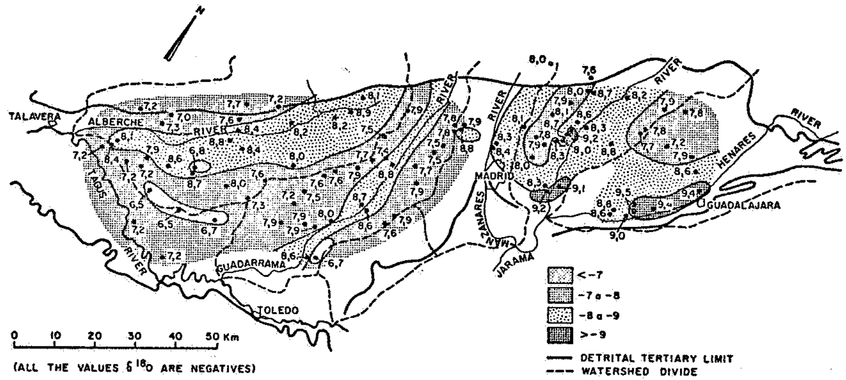


Figure 3 $\delta^{18}O$ distribution map

INTERVAL	NUM. SAMP.	FRECUENCY (F)	F. IN EACH ZONE		
-6,5 A -7,0	6	6,45	R=5,37	T=1,07	D-0
-7,1 A -7,5	15	16,12	R=12,9	T=3,22	D-0
-7,6 A -8,0	33	35,48	R=25,8	T=7,52	D=2,15
-8,1 A -8,5	15	16,12	R=2,15	T=1,07	D=12,9
-8,6 A -9,0	17	18,27	R=1,07	T=3,22	D=13,9
-9,1 A -9,5	7	7,52	R=0	T=0	D=7,52

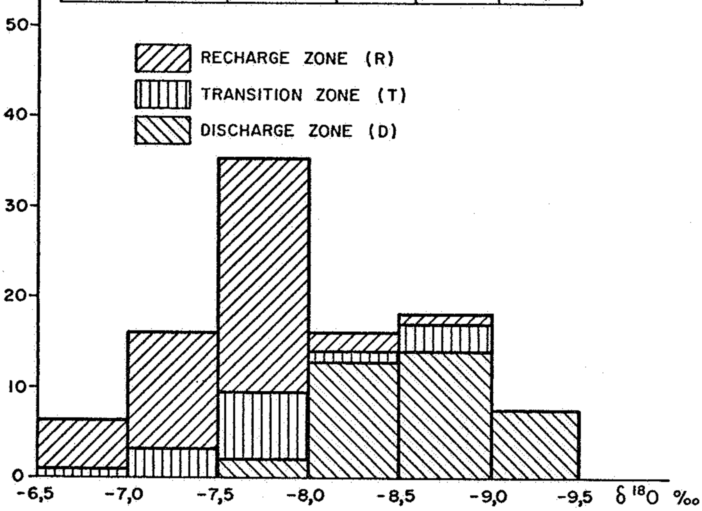


Figure 4 Histogram of $\delta^{18}O$ values (Distribution of $\delta^{18}O$ average)

Table 2   Statistical values in each subbasin according to the hydrogeological conditions

Zone \ Subbasin	Henares		Jarama		Manzanares		Guadarrama		Alberche		Total	
	N	X	N	X	N	X	N	X	N	X	N	X
Recharge	3	-7.85	9	-7.91	2	-7.95	11	-7.54	19	-7.32	44	-7.52
Transition	2	-8.60	4	-8.45	0	-	3	-7.90	6	-7.48	14	-7.84
Discharge	8	-9.17	7	-8.77	2	-8.35	5	-8.54	12	-8.40	34	-8.63

N = number of samples   X = average

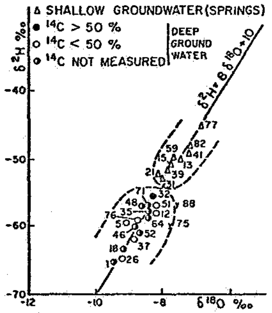
for discharge zones respectively and 0.53‰ for all 93 samples). This suggests that two distinct masses of water are involved in the same flow process.

c) Groundwater discharge is clearly more depleted in the eastern part of the system (Henares subbasin). This can hardly be attributed to the slight topographical rise (cooler condensation = altitude effect) since the local recharge does not show the same effect. Three hypotheses may be invoked to account for this depletion: (i) a supply of water infiltrated at high altitude (sierra), (ii) a local concentration of groundwaters infiltrated under cooler climatic conditions ('paleowaters', Fontes, 1981) and (iii) a recharge process preferentially due to heavy and exceptional rains having thus a lower heavy isotope content (amount effect) under climatic conditions similar to present ones. Because of hydrogeological constraints (the crystalline Sierra is largely impervious) hypothesis (i) may be discarded. Interpretations (ii) and (iii) are not exclusive but (ii) is in agreement with ^{14}C data.

5.2 Relationship between oxygen 18 and deuterium

All the 26 analysed samples (9 in the recharge zones, 17 in the discharge zones) lie along a correlation similar to the classical 'world meteoric line' (Craig, 1961) which is representative of present day oceanic precipitation (Figure 5). The deuterium excess $d = \delta^2H - 8\delta^{18}O$

close to 10 for both recharge and discharge types would indicate that atmospheric patterns and air moisture origin remained similar during the time elapsed in groundwater flow.



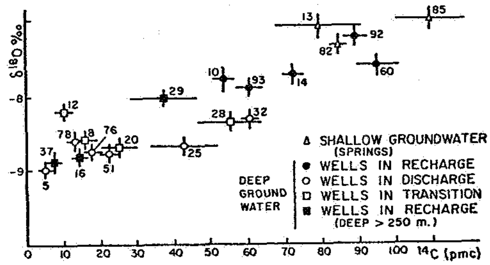


Figure 5 $\delta^{18}O$- $\delta^{2}H$ plot. Figure 6 $\delta^{18}O$ as a function of water ages

5.3 Relationship between oxygen 18 and carbon 14

Figure 6 shows a rather good correlation between ^{18}O of waters and ^{14}C contents of the total dissolved carbon. Carbon 13 contents suggest that no (or small) isotopic exchange occured between the dissolved carbon and some old solid carbonate from the matrix. This is in agreement with the lack of carbonate in the recharge zones. Therefore, any decrease in ^{14}C activities may be due to (i) age effect, (ii) mixing with very old (^{14}C 'dead') waters. Because of the rough linear trend observed between stable and radioactive environmental isotopes, we think that a mixing phenomena could provide a valid explanation for the evolution of the system.

Groundwaters from Madrid basin would be essentially the product of variable proportions of mixing between recent waters (present day recharge) with an ancient component. This old water infiltrated under cooler but similar climatic conditions as present day ones and is now totally free of ^{14}C by radioactive decay i.e. probably 'older' than 20 thousand years.

Acknowledgements

We should like to thank A.Filly, of the University of Paris-Sud, for her isotopic measurements and E.Baonza and A.Plata for precipitation data.

References

Craig, H. (1961). Isotopic variations in meteoric waters. Science 133, pp 1702-3.

Fontes, J.Ch. (1980). Environmental Isotopes in Groundwater Hydrology. In : Handbook of environmental isotope geochemistry . P. Fritz and J.Ch. Fontes ed. Vol. 1 chap. 3. The Terrestrial Environment, A. 1980, pp. 75-140.

Fontes, J.Ch. 1981. Palaeowaters. Stable Isotope Hydrology. Monograph. Chap. 12. IAEA, Vienna 1981, pp. 273-302.

Gonfiantini, R., Gallo, G., Payne, B.R., Taylor, C.B. 1976. Environmental Isotopes and Hydrochemistry in Groundwaters of Grand Canaria. In: Interpretation of Environmental Isotope and Hydrochemical Data in Groundwater Hydrology . Proceed. Adv. Group Meeting, Vienna, 27-31 January 1975, IAEA, Vienna, pp. 159-170.

Herráez, I., Llamas, M.R. 1982. Avance de las variaciones en O-18 en las aguas subterráneas del acuífero terciario detrítico de la Fosa del Tajo. Actas de la III Semana de Hidrogeología. Lisbon. (preprint, pp. 35).

López Vera, F., Lerman,J.C., Muller, A.B. 1981. The Madrid basin aquifer: Preliminary isotopic reconnaissance. Journal of Hydrology, 54, pp.151-166.

Llamas, M.R. 1976. La utilización de aguas subterráneas en Madrid. De los 'mayrat' musulmanes a los modelos digitales. Estudios Geológicos 32, pp. 121-139.

Llamas, M.R., Martinez Alfaro, P.E. 1981. Aplication of different computer models to the study of solute transport in a vertical profile of Madrid aquifer. The Science of the Total Environment, 21, pp. 347-352.

Sastre, A. 1978. Hidrogeología regional de la cuenca Terciaria del río Alberche. Tesis Doctoral, 3 tomos. Facultad Ciencias Geológicas Universidad Complutense. Madrid.

ELECTROMAGNETIC SENSING FOR GROUNDWATER AT SHALLOW DEPTHS IN HARD FORMATIONS

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Abstract

Limitations of conventional resistivity sounding in the study of subsurface conditions within resistive formations, call for multifrequency sounding (sensing) techniques. One such method called central frequency sounding (CFS) has theoretically been developed which involves the measurements of vertical component of magnetic field induced at the centre of a circulor ar square loop of limited and convenient extent. An analysis of response characteristics for frequency-domain and time-domain CFS in terms of resolution, detectability and equivalence confirm the applicability of the approach for shallow groundwater sensing in hard formations. Resolutions is found to be largely controlled by thickness and conductivity of individual layers. Equivalent is observed to be stronger in K-type compared to H-type earth model, the detectability of intermediate conductive layer being satisfactory.

Résumé

Les limitations de sondages conventionnels de résistivité dans l'étude des conditions substratum dans lws formations résistantes, appelent des techniques de sondage multi-fréquence. Une telle méthode "central frequency sounding" a été développée théoriquement et comprends le mesurage des components verticaux du domaine magnétique induit au centre du ruban circulaire ou carré de dimension limitée et convenable.

L'analyse de caractéristiques de réponse pour le domaine de fréquence ainsi que le domaine de temps CFS en termes de résolution, de détection et d'équivalence confirme l'application de l'approche pou "shallow groundwater sensing" dans les formations difficiles. La résolutions se trouve contrôlée en grande partie par l'épaisseur et la conductibilité de couches individuelles. On remarque que l'équivalence est plus fort dans le type-K que dans le type-H du modile terre. La détection de couches conductibles intermédiaires est bonne.

1 Introduction

Following Koenigsberger (1939) and Yoshizumi et al. (1959), Patra (1967, 1970) introduced a multifrequency sounding (sensing) method using a horizontal circular loop known as central frequency sounding abbreviated as CFS. Later studies on CFS through numerical integration by Sanyal (1975) and through Koefoed's (Koefoed et al., 1972) digital linear filter technique by Shastri (1981) have led to an easy availability of frequency- and time-domain response curves for CFS over multi-layer earth models.

2 Statement of the problem

Application of digital linear filter technique to the computation of CFS response curves over a multi-layer earth opened up avenues for presentation of data in various forms and analysis in terms of resolution of layer parameters. The problem consists of a comparison of frequency- and time-domain approaches, an analysis of resolution between curves, studies on detectability of intermediate layers and equivalence conditions for H- and K-type earth models.

3 Presentation and analysis of data

An insulated circular loop carrying an alternating current is placed on the ground (transmitter) and the vertical component of the secondary magnetic field induced at the centre is measured trough a receiver coil placed concentrically (Figure 1). Shastri (1981) has presented sets of

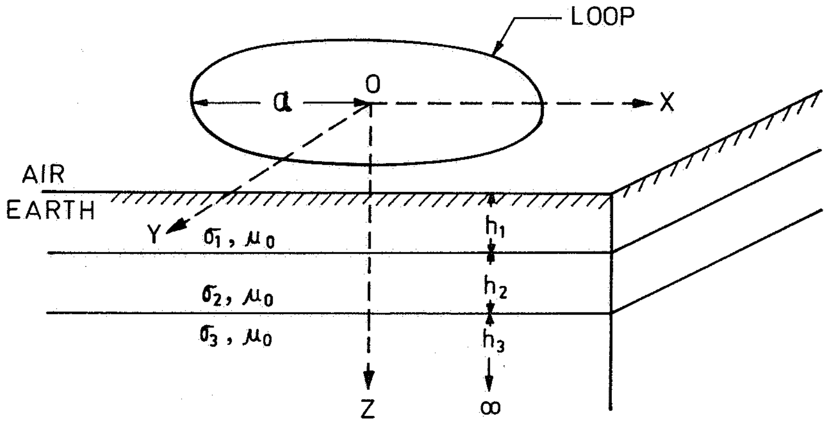

Figure 1 A loop of radius α over a three-layer earth

mukti-layer curves. One set of typical three-layer amplitude versus conductivity parameters (B) curves is repriduced in Figure 2. Curves showing variation of time-domain response are presented in Figure 3. Figure 4 and 5 correspond to the separation of amplitude responses respectively of three-layer earth (for two values of thicknessratio) from that of the homogeneous earth giving an idea of the detectability in frequency-domain. Phasor diagrams for H-type and K-type models respectively equivalence in frequency-domain are reproducted respectively in Figures 6 and 7. Figures 8 and 9 represent phasor diagram for equivalence in time-domain measurement. The results of analysis of data are as folows:

3.1 Resolution

Amplitude responses over two- and three-layer earth models provide a significant resolution of layer parameters. The separation between curves on response diagrams is prominent for different layer thicknesses meaning thereby a good resolution for two-layer earth. Layer parameters for three-layer model (Figure 2) are well resolved at high conductivity contrasts for varying loop radius. This is particularly true for highly resistive basement. Significant dissimilarity in the shape of three-layer time-domain response curves (Figure 3) is also observed. Variation in both layer conductivity and thickness is well resolved on two-layer time-domain response curves (not presented here). In general resolution of layer parameters in both the modes is satisfactory over three-layer earth models.

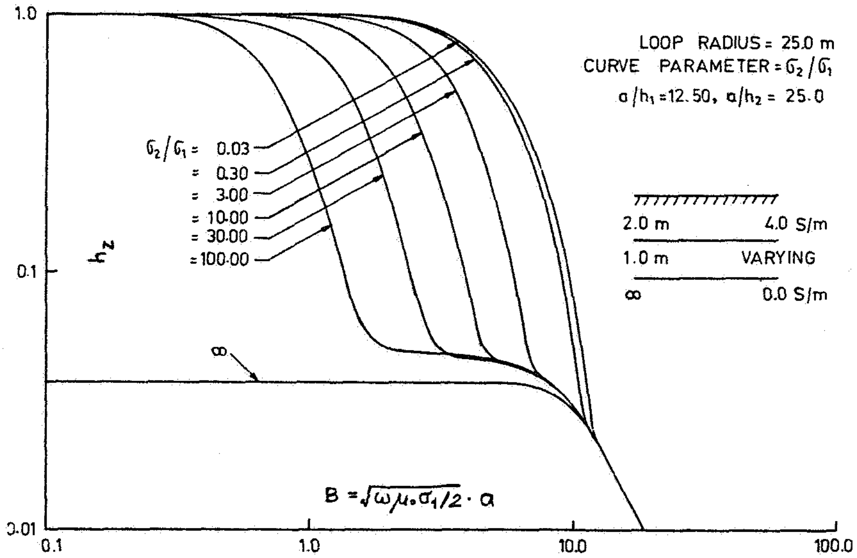

Figure 2 Amplitude response curves over a three-layer earth model

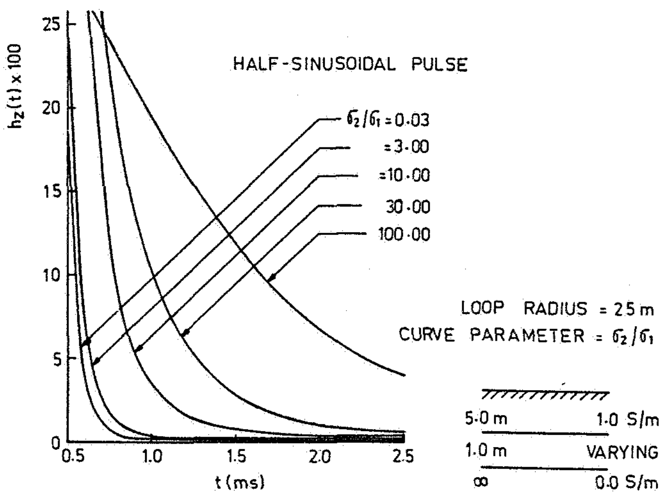

Figure 3 Time-domain response curves over a three-layer model

3.2 Detectability

Detectability of intermediate conductive layer in H-type is found to be satisfactory in both the modes particularly with larger loop radius (Figure 4). Detection of intermediate resistive layer (K-type) is improved even for larger loop radius (Figure 5). Thus, intermediate conductive layer is well detected by the shallow sensing method, CFS.

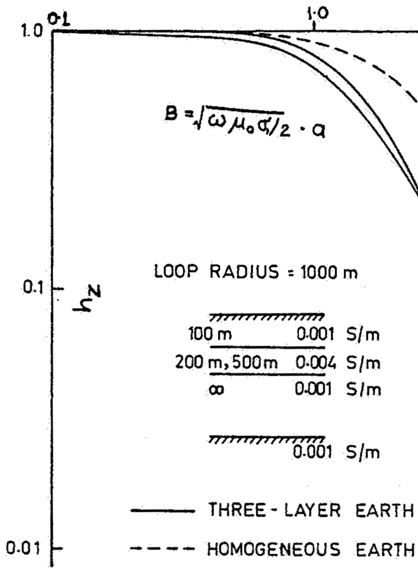

Figure 4 Detectability of an intermediate conductive layer

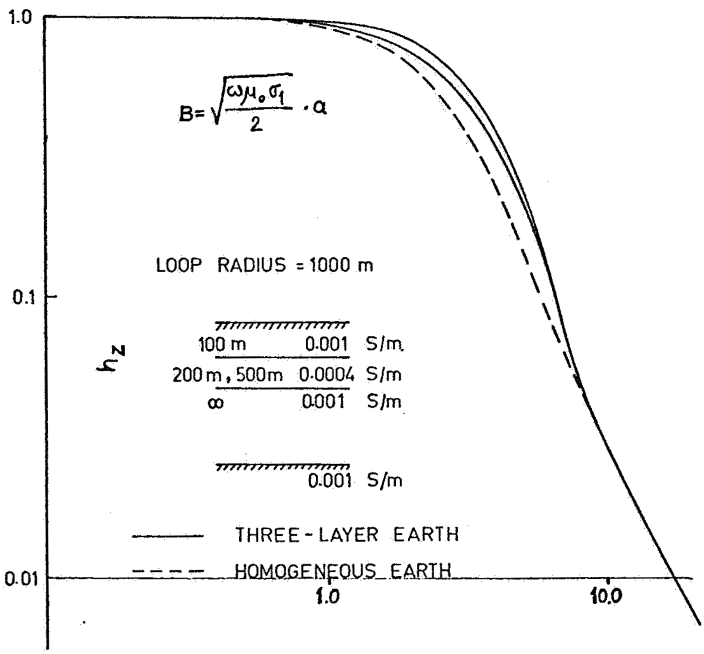

Figure 5 Detectability of an intermediate resistive layer

3.3 Equivalence

Phenomenon of equivalence is analysed in both the modes of presentation. Typical examples for H- and K-type earth models are given in Figures 6 and 7 for frequency-domain and in Figures 8 and 9 for time-domain. RMS difference of four percent or less between responses of original and equivalent models is considered for defining equivalence. In case of H-type earth models, equivalence is observed to be poor. Equivalence is found to be strong in K-type earth models implying thereby a poor resolution of layer parameters.

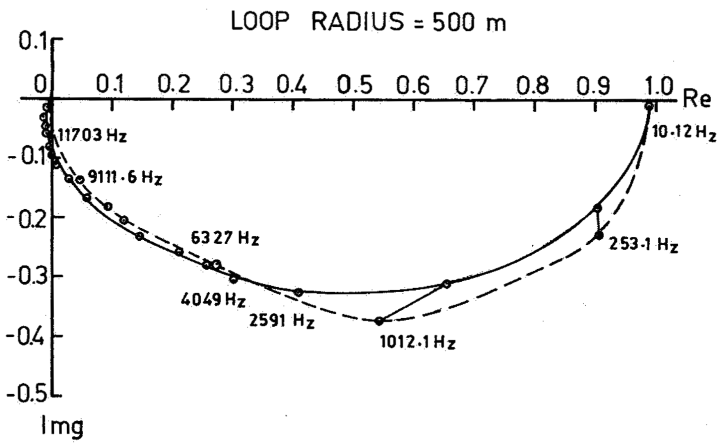

Figure 6. Frequency-domain phasor diagram for H-type model

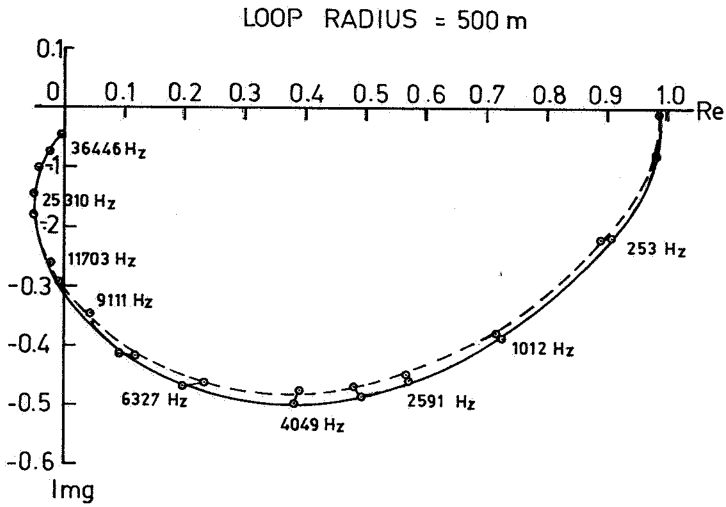

Figure 7 Frequency-domain phasor diagram for K-type model

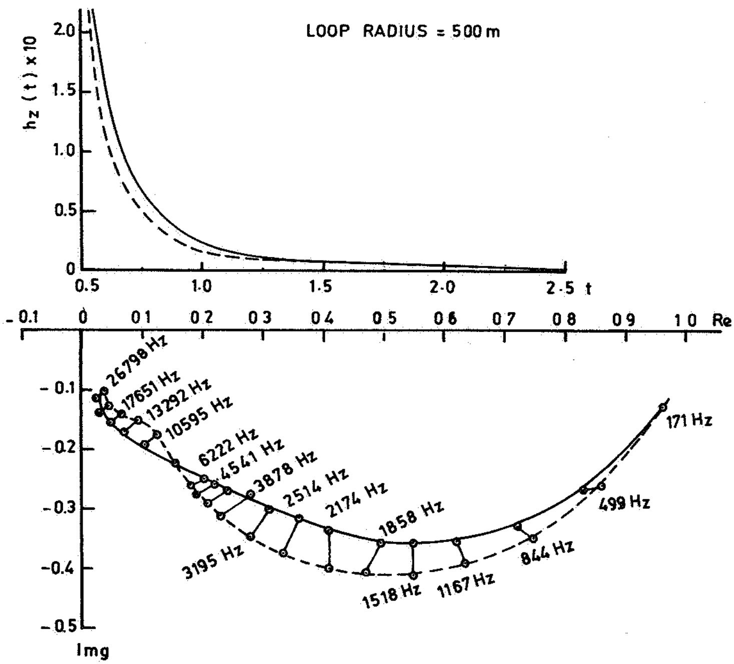

Figure 8 Time-domain phasor diagram for H-type model and its equivalent

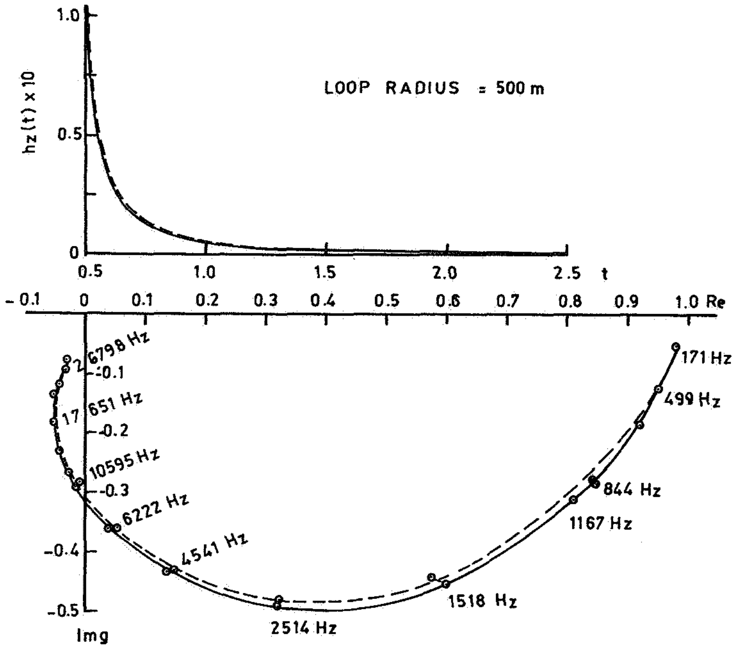

Figure 9 Time-domain phasor diagram for K-type model and its equivalent

4 Conclusion

A study of variation in amplitude of normalized magnetic field on the surface of multi-layer earth models (only three-layer cases presented here) provides an idea of relative depth of investigation and the resolution capabilities. An analysis of sets of curves puts the depth of investigation approximately at half the loop radius. Resolution of intermediate conductive layer in a three-layer sequence is significant, particularly with the increase of loop radius. Frequency-domain response resolution is poor for increasing number of layers. Time-domain sounding resolves layer parameters better in a sequence comprising more than three layers. Plots of RMS difference values (not reproduced here) of frequency- and time-domain responses and phasor diagrams between H-type and K-type original and equivalent models show that the equivalence in K-type model is stronger than that for H-type. Studies show that H-type model offers better detectability compared to K-type. For models with more than three layers, the resolution of layer parameters, in general, is poor in both the modes even with increased loop radius.

5 Concluding remarks

It is established that three-layer H-type models with weak equivalence are well resolved and detected. The results, therefore, are likely to play an important role in delineating saturation zones between top hard formation and a resistive basement. CFS, thus, can help in a rapid shallow groundwater sensing as and when a suitable instrument is developed for purpose.

References

Koenigsberger, J.G., 1939. Elektrische vertikal-sondierung von der erdoberfläche aus mit der zentral-induktions methode. Beitr. Angew. Geophys., 7: 112-161.

Patra, H.P., 1967. Some studies on geoelectric sounding in engineering and hydrogeological problems. Ph.D. thesis, Indian Institute of Technology, Kharagpur.

Patra, H.P., 1970. Central frequency sounding in shallow engineering and hydrogeologycal problems. Geophys. Prospect., 18: 236-254.

Sanyal, N., 1975. Some studies on electromagnetic depth sounding for shallow groundwater exploration problems. Ph.D. thesis, Indian Institute of Technology, Kharagpur.

Shastri, N.L., 1981. Some studies on electromagnetic depth sounding over a multi-layer earth. Ph.D. thesis, Indian Institute of Technology, Kharagpur.

Yoshizumi, E., K. Taniguchi and T. Kiyono, 1959. Vertical electrical sounding by central inducation method. Mem. Fac. Eng., Kyoto Univ., 21: 154-169.

ESTIMATION OF AQUIFER TRANSMISSIVITY FROM SURFACE GEOELECTRICAL MEASUREMENTS

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Abstract

An analytical relationship between aquifer transmissivity and a normalised transverse resistance of homogenereous and isotropic aquifers has been suggested in the paper, after modifying an earlier relation established by the present authors, between aquifer transmissivity and transverse resistance. By normalising the aquifer resistivity suitably, with the help of aquifer water resistivity, a modified relation is obtained between aquifer transmissivity and normalised transverse resistance. The relationship obtained is found to be linear as the product of aquifer hydraulic conductivity and normalised aquifer conductivity is constant for a particular basin ans seems to be valid for calculation of aquifer transmissivity in areas with varying groundwater quality. The transmissivities obtained by this approach are closer to the actual field transmissivities as evident from a field example of alluvial aquifers of Southern Banda District, Uttar Pradesh, India.

1 Introduction

Many approaches are available for estimation of aquifer parameters like transmissivity and storativity. The formulae for calculating these parameters from pumping tests are valid only if various assumptions about aquifer continuity, thickness, homogeneity, isotropy and well storage are valid. All the assumptions are, however, seldom satisfied in practice, and the results are, more often, quite approximate. At the same time the procedures are time consuming and costly if resorted to indiscriminately in a particular area. However, if we have some approximate information of transmissivity and storativity of the aquifer from some other methods, on lower cost / information ratio basis, the excessive use of pumping test methods to calculate these parameters can be minimised. In the present paper, an attempt has been made to calculate aquifer transmissivity from surface geoelectrical measurements.

The surface resistivity sounding techniques are commonly employed in ground water investigations. The results obtained after interpretation of the resistivity data give thicknesses and resistivities of various sub-surface layers. From this data, the Dar Zarrouk parameters, i.e., transverse unit resistance (R) and longitudinal unit conductance (C) (Maillet, 1947) can be calculated easily. In the recent past, Dar Zarrouk parameters have been used for development of indirect methods of resistivity interpretation (cf. Orellano, 1963, Zobdy, 1965, 1974). Some attempts have also been made recently, to establish empirical and semi-empirical relationships between aquifer transmissivity and some geoelectrical parameters obtained in resistivity measurements (Ungemach et al. 1969, Steeples, 1970, Henriet, 1976, Kelly, 1977; Heigold, 1979; Kosinski and Kelly, 1981).

Kelly (1977) tried to establish empirical relationships between aquifer resistivity and aquifer hydraulic conductivity, and transverse resistance and aquifer transmissivity in glacial outwash materials of Southern Rhode Island. He also gave semi-empirical relation between the aquifer formation factor and hydraulic conductivity. Heigold et al. (1979) obtained an inverse relationship between the aquifer resistivity and hydraulic conductivity for glacial outwash materials in Central Illinois. Kosinski and Kelly (1981) tried to establish a direct equivalence between so called "normalised transverse resistance" and aquifer transmissivity. Sri Niwas and Singhal (1981) established, for the first time, an analitycal relationship between ttansverse unit Resistance and Aquifer Transmissivity, by combining the two fundamental laws., viz. Ohm's law of Current flow and Darcy's law of ground water flow for homogeneous and isotropic porous media. They tested the applicability of the relation using published data for glacial outwash materials of Rhode Island. However, the applicability of the formula was restricted in the sense that an assumption was made that the quality of groundwater remains fairly constant in a basin with uniform geological conditions.

In the present study, a modification has been suggested in the analytical relationship originally proposed between transmissivity and transverse resistance, so as to account for variation in chemical quality of groundwater as well. This modified relationship is likely to have wider applications as aquifers having groundwater with varying chemical quality can be considered for calculation of the aquifer transmissivity. The approach has been tested using some data of Banda District Uttar Pradesh, India.

2 Formulation

If a prism of isotropic and homogeneous aquifer material having unit cross sectional area and thickness "h" is considered, the Darcy's law and Ohm's law can be combined to give the folowong relationship (Sri Niwas and Singhal, 1981).

$$T = K \sigma R \qquad (1)$$

where

T = the aquifer transmissivity

K = the hydraulic conductivity

σ = the electrical conductivity (= $1/\rho$, the resistivity of the medium)

and R = the Transverse Unit Resistance.

Equation (1) assumes that the change in aquifer resistivity is due to changes in aquifer material (excluding rock matrix), and tortuosity of the interconnected pores, while it is presumed that the gross chemical quality of the groundwater remains relatively uniform. The equation also indicates a linear relationship between aquifer hydraulic conductivity and aquifer resistivity. This linear relationship was explained by Kelly (1977) and Sri Niwas and Singhal (1981).

Equation (1) can be modified by taking into consideration a "normalized aquifer resistivity" instead of "aquifer resistivity" (Kosinski and Kelly, 1981). The normalization factor is always the ratio of actual average aquifer water resistivity (ρ_w, av) and the aquifer water resistivity (ρ_w) at a particular location. Thus we can rewrite equation (1) as

$$T = K\ \sigma' R' \tag{2}$$

where

σ' ($= \sigma . \dfrac{\rho_w}{\rho_{w\ av}}$) and R' ($= R . \dfrac{\rho_{w\ av}}{\rho_w}$) are, respectively, "normalized conductivity" and "normalized transverse resistance" of the aquifer.

Equation (2) gives an analytical relation between aquifer transmissivity and the so called "normalized transverse resistance" of the aquifer by taking into consideration the variation of quality of aquifer water at different places. In this equation, product K. σ' remains constant.

A natural corollary of equation (2) can be written as,

$$K = \alpha\ \rho' \tag{3}$$

where

α is equal to product K σ' which is always constant in a basin and ρ' is the normalized aquifer resistivity.

Equations (2) and (3) appear to be useful for computing the transmissivity and hydraulic conductivity of the aquivers in porous, homogeneous and isotropic media where the variation on quality of groundwater is of consequence in influencing the bulk resistivity of the aquifer. In such a situation equation (1) is not applicable as the product 'K σ' varies considerably at different locations. A rise in aquifer water resistivity at any place, would result in an increase in the resistivity of the aquifer at that point and the figure for transmissivity obtained by using equation (1) is likely to be higher than the actual field transmissivity. The transmissivity thus obtained can be normalized with respect to an average value of aquifer water resistivity (ρ_w, av) within the basin, by considering the fact that for boreholes with higher aquifer water resistivity, the transmissivity should be reduced by an appropriate factor and vice versa. However, use of this approach would be valid only if rock matrix, like interstitial clay etc. is not responsible for changes in overall aquifer resistivity.

3 Results and discussion

The validity of equation (2) has been tested using the data of unconfined aquifers in the alluvial materials of South Banda District, India. The actual field transmissivities of the aquifers were calculated by conducting pumping tests on five large-diameter dug wells (or dug cum bore holes) using the methods given by Papadopolous (1967), and Boulton and Streltova (1976) for early stages of response of unconfined aquifers tapped by large diameter wells. The aquifer water samples of the wells were tested and their electrical conductivities were measured which were later, reduced to temperatures of 25°C. Subsequently the vertical electrical soundings were recorded on the well sites and were interpreted using direct as well as indirect methods. Table 1 summarises the results of vertical electrical soundings, aquifer water resistivities, field hydraulic conductivities and transmissivities for five locations of the alluvial aquifers of Banda District, India. With the help of data of aquifer water resistivity, an average value of aquifer water resistivity (ρ_w, av= 12.01 ohm-m, at 25°C) may be calculated for the purpose of calculation of the normalized aquifer resistivity ($\rho' = \rho \cdot \frac{\rho_{w,av}}{\rho_w}$) and normalized transverse resistance ($R'=h\rho'$).

It is clear from Table 1, and figure 1 plotted between products $K\rho'$ (and $K\sigma$) at different sites and spatial distribution of test sites that product $K\sigma'$ is constant (1.13) in the basin, while the product $K\sigma$ without normalization of electrical conductivity, shows considerable variation. From this general value of $K\sigma'$ for the alluvial aquifers of Banda distt. values of transmissivity using equation $T=1.13\ R'$ have been computed (Col.12 Table 1). These values of computed transmissivities compare fairly well with the actual field transmissivities (Col. 11 Table 1).

Table 1: Comparison of resukts of Geoelectrical soundings and pumping resrs for alluvial aquifers of Southern parts of Banda District, Uttar Pradesh.

Col No.	Locality and Site No. / Data of VES and pumping tests.	Sitapur Site 1	Sitapur Site 2	Karwi town Site 3	Bhaunri Site 4	Bhaunri Site 5
1.	VES No.	CS5	CS15	CS19	CS17	CS17
2.	Aquifer thickness in well (m)	13.10	8.40	2.60	4.00	4.15
3.	Aquifer thickness from VES (m)	28.4(?)	12.00	26.00	4.00	4.00
4.	Aquifer water resistivity at 25°C, $_w$ (Ohm-m)	18.74	13.80	5.15	11.17	11.17
5.	Aquifer resistivity, ρ (Ohm-m)	19.00	11.40	14.40	17.80	17.80
6.	Normalized Aquifer resistivity, $\rho' = \rho \cdot \frac{\rho_{wav}}{\rho_w}$ (Ohm-m^2)	12.18	9.92	33.81	19.14	19.14
7.	Normalized transverse resistance, R' (Ohm-m^2)	159.56*	119.04	879.06	76.56	76.56
8.	Aquifer Hydraulic conductivity, K(m/day)	11.63*	15.32	39.85	18,25	19.60
9.	Normalized conductivity σ' (mho/m)	0.0821	0.1008	0.0296	0.0522	0.0522
10.	K_σ'	0.954	1.544	1.180	0.953	1.023
11.	Actual field transmissivity, T(m^2/day)	152.34	183.80	1036.20	73.00	78.40
12.	Aquifer transmissivity using Eq.T=1.13R' (m^2/day)	180.3	134.5	993.3	86.5	86.5

* These values of normalized transverse resistance and aquifer hydraulic conductivity were arrived at, by using aquifer thickness of the well as the interpretted electrical thickness of aquifer seems questionable.

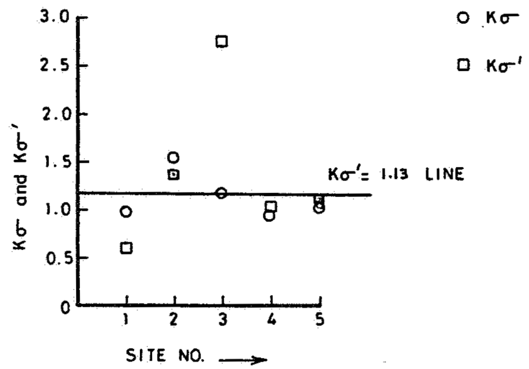

Figure 1 Plot of products Kσ and Kσ' versus spatial distribution of the sites.

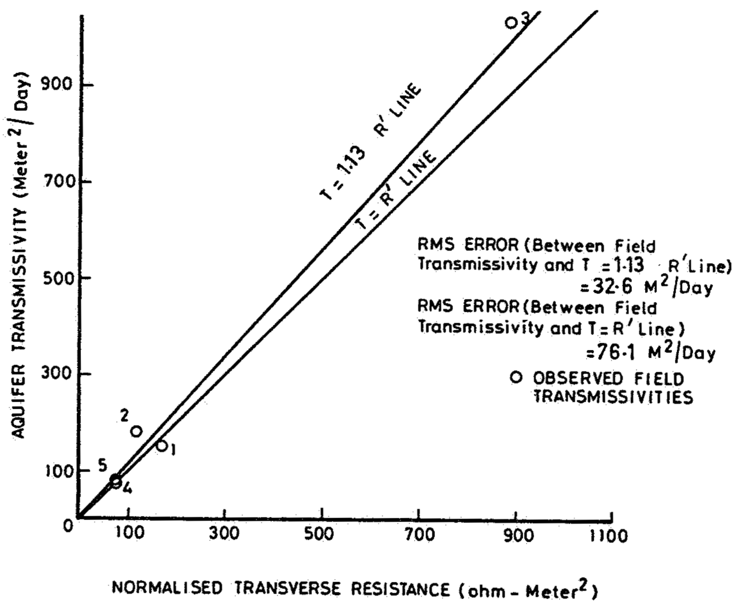

Figure 2 Test plot between Transmissivities obtained using various approaches and the Normalized transverse resistance (R').

Figure 2 shows a graph between transmissivities derived from different approaches and the normalized transverse resistance. The root mean square error ($= \sqrt{1/n \, \Sigma (T_e - T_o)^2}$) between the actual field transmissivities and those derived by equation (2) amount to 32.6 m^2/day and is found to be lower than the error (76.1 m^2/day) between actual field transmissivity and the normalized transverse resistance as suggested by Kosinski and Kelly (1981). Here T_e and T_o are the computed and observed transmissivities respectively and n is no. of data points. This suggests that if there is no information available, whatsoever, about the hydraulic conductivity at any point in the considered formula $T = R'$ may be used for having an approximate idea of the aquifer transmissivities.

It is relevant to note that electrical anisotropy of different geological formations in field conditions may be of considerable consequence in giving rise to variation in values of actual field transmissivities (arrived at from the pumping tests) as well as the normalized transverse resistance (calculated from geoelectrical data). A proper correction factor for the aquifer transmissivity and transverse resistance may be feasible to calculate particularly in case of transversely isotropic homogeneous aquifers if extent of anisotropy is known.

4 Conclusions

From the above discussion, it may be concluded that the direct relation between aquifer transmissivity and so called normalized transverse resistance of a homogeneous and isotropic aquifer in a groundwater basin with varying water quality is more valid rather than the one between aquifer transmissivity and transverse resistance. Accordingly, equation (2) seems to be avaible relation to estimate the aquifer transmissivity from so called, normalized transverse resistance, if some limited information about hydraulic conductivity and aquifer water quality at any other point is available from existing data. The hydraulic conductivity at a location may be estimated from the relation $K = \alpha \, \rho'$. These relations may not however hold good if the bulk resistivity of the aquifer is also affected by the rock matrix. The approach needs to be tested in areas with differing geological conditions.

Acknowledgements

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References

Boulton, N.S. and T.D. Streltsova, 1976. The drawdown near an abstraction well of large diameter under non steady conditions in an unconfined aquifer. Hydrol., 30: 29-46.

Henriet, J.P., 1976. Direct application of Dar Zarrouk parameters in groundwater surveys. Geophys. Prospec., 24: 344-353.

Kelly, W.E., 1977. Geoelectric sounding for estimating aquifer hydraulic conductivity. Groundwater, 15: 420-425.

Kosinski, W.K. and W.E. Kelly, 1981. Geoelectrical soundings for predicting aquifer properties. Groundwater, 19: 163-171.

Maillet, R., 1947. The fundamental equations of electrical prospecting. Geophysics., 12: 529-556.

Orellano, E., 1963. Properties and drawing of so called Dar Zarrouk curves. Geophysics., 28: 99-110.

Papadopulous, I.S., 1967. Drawdown distribution around a large diameter well. Proc. Groundwater Hydrology Symp. Am. Water. Res. Assoc. pp. 157-167.

Sri Niwas and D.C. Singhal, 1981. Estimation of aquifer transmissivity from Dar Zarrouk parameters in porous media. J. Hydrol., 50: 393-399.

Steeples, D.W., 1970. Resistivity methods in prospecting for ground water. M.Sc. thesis, Kansas State University, Manhattan, Kans.

Ungemach, P., F. Mostaghimi and A. Duprat, 1969. Essais de determination du coefficient d'emmagasinement en nappe libre et application a la nappe alluvial du Rhin. Int. Assoc. Sci. Hydrol., 14: 169-190.

Zohdy, A.A.R., 1965. The auxiliary point method of electrical sounding interpretation and its relationship to the Dar Zarrouk parameters. Geophysics., 30: 644-660.

Zohdy, A.A.R., 1974. Electrical methods. In: Application of Surface geophysics to Groundwater Investigations; A.A.R. Zohdy, G.P. Eaton and D.R. Mabey, Ch.D.U.S. Geol. Surv. Tech. Water Resour. Invest., No. 116.

INVESTIGATING FISSURED GROUNDWATER SYSTEMS USING SLUG TESTS AND SINUSOIDAL TESTS

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Abstract

A new interpretation of a conventional test, the slug test, is presented together with a new unconventional test approach. It is shown that using conventional slug test analysis leads to large errors in the determination of specific storage and smaller errors in the determination of hydraulic conductivity from slug tests in fractured crystalline rocks. A method for correcting conventional analysis is given. The sinusoidally varying pressure test is briefly described together with a preliminary trial. It is seen that homogeneous porous medium analysis is not appropriate so the fissured porous medium approach is suggested here also. Some of the possibilities of the test are outlined together with some of the practical benefits.

Resumé

Cet exposé présente une nouvelle interprétation de l'essai classique dit 'slug test" et décrit un essai innovateur à pression variable en fonction sinusoidale du temps.On met en évidence que l"analyse classique des slug tests faits dans roches cristallines fracturées amène à des erreurs importantes sur le stockage spécifique;de plus il y a des erreurs moins importantes sur la conductibilité hydraulique.On présente une méthode de correction de ces analyses classiques.L'essai à pression sinusoidale est brèvement décrit y compris une première épreuve de ce principe. L'analyse des résultats à partir d'un milieu poreux homogène s'avérant inapte, et on reprend le modèle du milieu poreux fissuré. On décrit à grands traits quelques possibilités et avantages pratiques de cette technique.

1 Introduction

During the course of carrying out hydrogeological investigations as part of the UK programme on the feasibility of geological disposal of radioactive waste, a number of techniques of wider application have been developed. In particular, whilst working in fractured granites, a new analysis of the behaviour of slug tests in fissured porous media has been developed. The basic assumption that water in fissures interacts with pore water within the rock matrix has also been used in the analysis of a novel variation of interference testing involving a sinusoidal pressure change as a source.

Whilst slug tests are a comparatively well-known method for measuring rock properties with a generally accepted analysis, the idea of using an artifically-generated, sinusoidally-varying pressure wave to measure aquifer properties is new to hydrogeology. Thus it has been necessary both to carry out practical trials of the method and to develop an adequate method of analysing the results.

This paper sets out to show that a fissured porous medium slug test analysis explains the responses obtained in a large number of slug tests in fractured granite and that the same applies to fissured aquifers. It can be seen that the results of a slug test in fissured rock are primarily a function of the time scale of the test compared to the hydraulic properties of the matrix rock. In slug testing there is only a limited ability to alter the timescale of the test by changing the diameter of the tubing in which water level fluctuations occur. On the other hand, using a periodic signal of sinusoidally varying pressure enables a whole range of timescales to be controlled by the experimenter, thus yielding results which can include varying amounts of fissure-matrix interaction. The sinusoidal test has the added advantage of being a cross-hole technique whose interpretation is envisaged as a point source with a number of observation points. This potentially enables the results to be analysed in terms of a hydraulic conductivity

ellipsoid if sufficient measurements are made.
The refinement of slug test analysis and the methodology of the sinusoidal wave test both offer obvious benefits to the assessment of pollution migration in fissured rocks.

2 Slug Tests in Fissured Rock

The analysis of slug tests in homogeneous rocks is well established and all effective variations are based on the method put forward by Cooper et al, 1967. In their analysis every test yields values for hydraulic conductivity and specific storage (the amount of water elastically stored in unit volume of rock per unit change in pressure). In actual rock the amount of possible variation in specific storage is comparatively limited, resulting either from the compressibility of the matrix (with a minimum value of about 10^{-7} m^{-1} for granite and higher for all sedimentary rocks) or from the compressibility of the water in the fissures (with a maximum value for very frequent fissures of about 10^{-8} m^{-1}). The results from a large number of slug tests in crystalline rocks at Altnabreac, Scotland (Holmes, 1981) show (Figure 1) that many tests yield specific storage values which are outside the values possible for flow in either the fissures only or the matrix only. It is also notable that a large number of results apparently involve less specific storage than even a single planar fissure per slug tested zone. Another general observation about the results is that as hydraulic conductivity increases specific storage decreases. This is exactly the opposite of what would be expected in a homogeneous porous medium.
It was against this background that a new analysis was derived (Barker and Black, in press) which explicitly includes fractures (which account for the bulk of the rock's hydraulic conductivity and very little of the storage) and porous matrix with complementary properties (i.e. low hydraulic conductivity and the bulk of the storage). The analysis is expressed in the form of four dimensionless parameters α, β, γ and T of which two (α and T) are common to the original analysis of Cooper et al, 1967. It can be shown that as the rock matrix becomes less permeable or has little storage capacity the parameter β will be very small and the analysis becomes the same as Cooper et al, 1967. In other words the

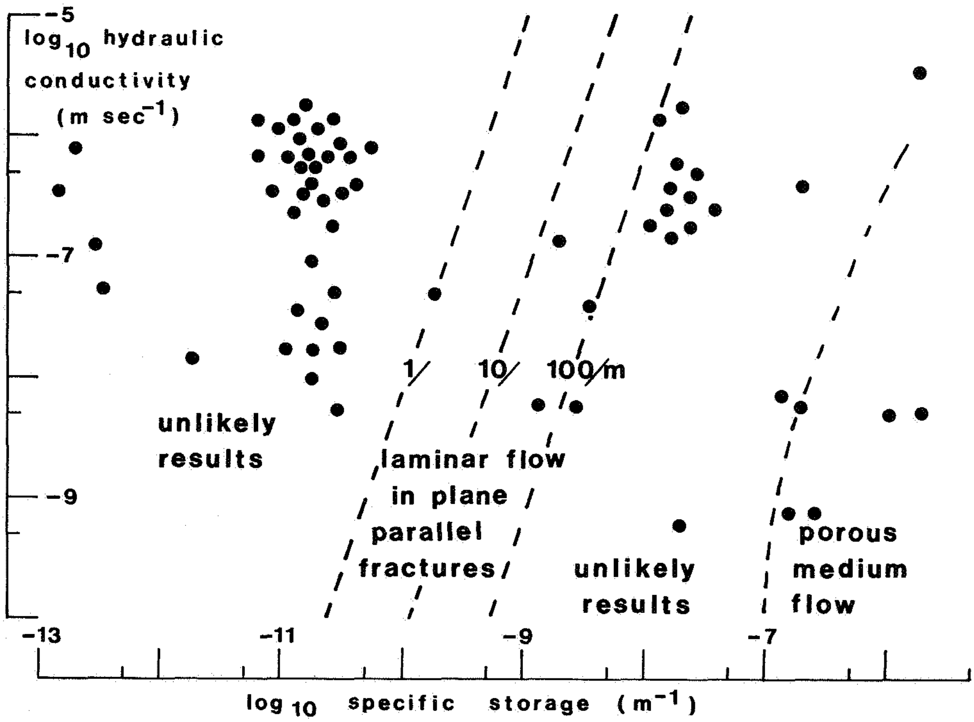

Fig. 1 Results of slug tests in three boreholes at Altnabreac derived using the analysis of Cooper et al, 1967

fractured rock behaves qualitatively as homogeneous rock with a specific storage related only to the fissures. Alternatively if the permeability of the rock matrix is large or there are very frequent fissures then the parameter γ will be small. This means that the fractured rock behaves qualitatively as would homogeneous rock with a specific storage equal to the sum of the storages of the fissures and the matrix.

Since the analysis of individual test results now involves solving for four unknowns, rather than two, the previous type curve method becomes unwieldly since there are an infinite number of possible type curves. A technique for solving this problem by treating groups of test results on the same interval has been presented (Black and Barker, 1982) but is still unwieldy and not altogether satisfactory. The ideas underlying a more practical approach have now been put forward (Barker and Black, in press) whereby the errors involved in using the pre-existing homogeneous analysis in fissured rock are evaluated. Thus the errors in hydraulic conductivity and specific storage derived using the analysis of Cooper et al, 1967 are calculated in relation to the dimensionless parameters α, β and γ. It can be shown that derived hydraulic conductivity will not be wrong by more than a factor of 3 and under most circumstances will not exceed a factor of 2. Errors in derived specific storage when compared to fissure-only storage can range from 10^{-4} to 10^{5} whilst errors compared to total storage range from no error to 10^{-6}.

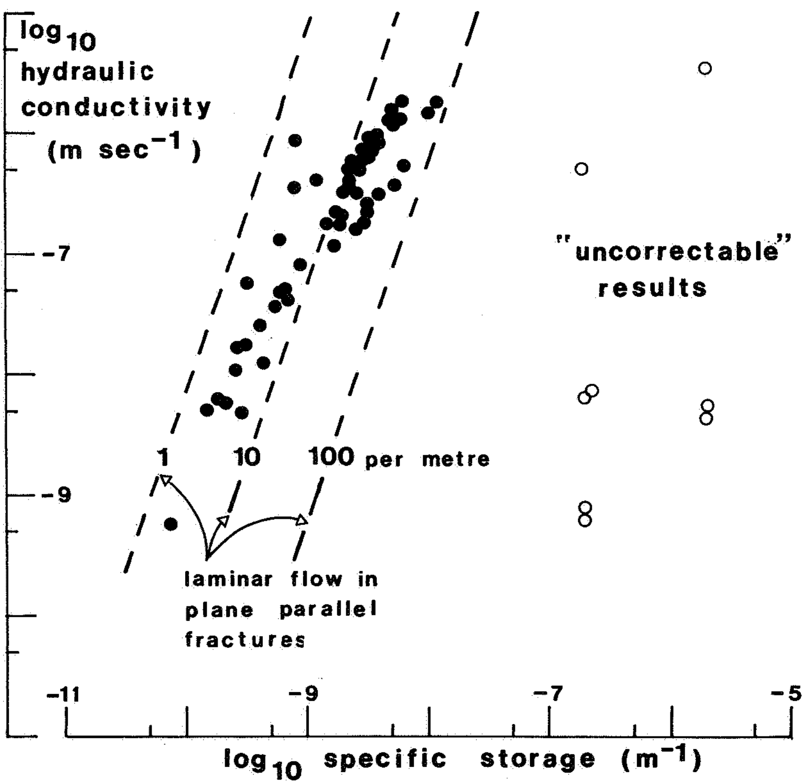

Fig. 2 The results of Figure 1 replotted after "correcting" with the fissured porous medium analysis

The results in Figure 1 have been "corrected" using this method whilst those which cannot be "corrected" have been left "uncorrected". As can be seen in Figure 2 the bulk of the test results are more than adequately explained by the fissured porous medium analysis. Where the maximum number of fissures in any particular interval are known by some independent method (such as borehole television), or the matrix properties are known, then the results can be analysed in more detail. However if tests are carried out with this sort of detail then it is obviously more sensible to use the fissured porous medium analysis directly rather than using the inappropriate homogeneous analysis and then "correcting" it afterwards.

3 Sinusoidal Pressure Tests

The sinusoidal pressure wave method has been suggested as a method for measuring directional hydraulic diffusivity (hydraulic conductivity/specific storage) by Black (1981) and Black and Kipp (1981). The method consists of generating a sinusoidal variation of pressure in a small packered-off section of a borehole and detecting the signal in a packered-off section of an adjacent borehole. By moving both the source zone and the receiver zone various orientations can be measured.

The method has been tried in two adjacent boreholes at an experimental quarry in granite (Black and Holmes, in press). Signals were measured easily for zones separated by up to 40 m of rock. The source signal with a frequency of 1.3×10^{-3} Hz was generated by alternately abstracting-from and injecting-into a 7 m long packered-off section of 140 mm diameter borehole. A sample of the output from both the source and receiver zone is shown in Figure 3. Although the pulsing of the hydraulic pump is clearly evident on the source signal the received signal is as expected with considerable peak amplitude attenuation and phase lag and none of the very short wavelength pulses discernible.

The results (Table 1) were analysed by taking the amplitude attenuation and the phase lag and deriving values of diffusivity from the homogeneous analysis presented in Black and Kipp (1981). When this was attempted only two of the seven positive results yielded completely satisfactory interpretations (Black and Holmes, in press). For this reason the analysis was extended to take into account the idea of a fissured porous medium.

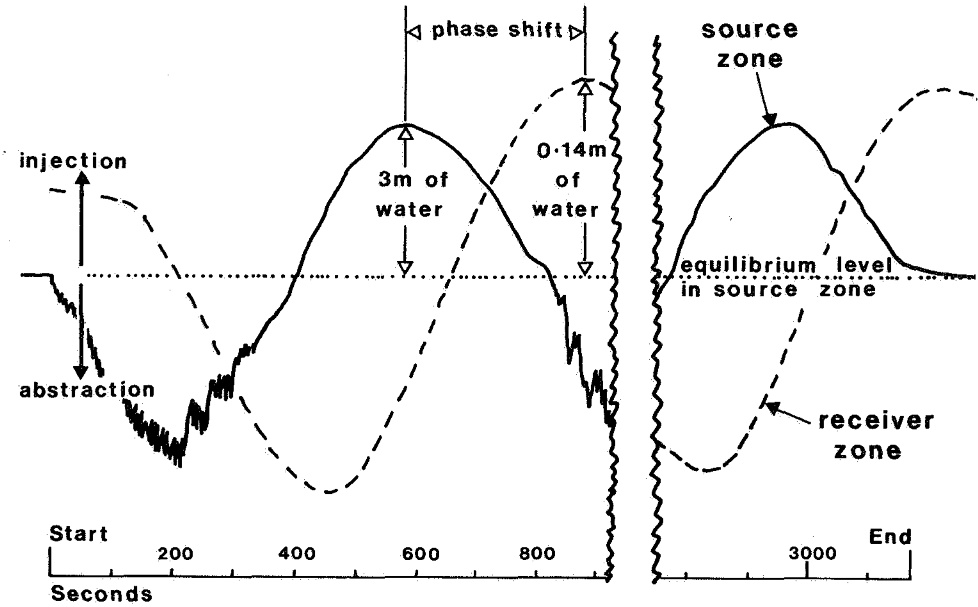

Fig. 3 Example of source zone and receiver zone pressures during a sinusoidal pressure test

The fissured porous medium analysis is similar in configuration to that for the slug test and depends on a number of dimensionless parameters which include five unknowns (fissure hydraulic conductivity, fissure specific storage, matrix hydraulic conductivity, matrix specific storage, block geometry). In the case of the results reported here, the matrix properties were known approximately, the two fissure properties were combined as fissure diffusivity and the block geometry was assumed to be in the form of slabs. The two results for each test (amplitude attenuation and phase shift were then used to derive fissure diffusivity

Table 1 Summary of results

Depth of receiver zone in Borehole 4 (m below ct)	Average: Received Amplitude*	Average: Phase shift (radians)	Direct distance from source to receiver zone (m)
15-22	1.48×10^{-3}	0.56	37.9
23-30	3.62×10^{-3}	0.41	32.2
30-37	3.80×10^{-2}	0.29	28.1
34-41	4.10×10^{-2}	0.30	26.4
41-48	8.65×10^{-3}	0.40	24.6
46-53	7.54×10^{-3}	0.39	24.5
53-60	6.88×10^{-3}	0.33	26.0

*All amplitudes result from calculating: $\frac{\text{maximum amplitude received}}{\text{maximum amplitude generated}}$

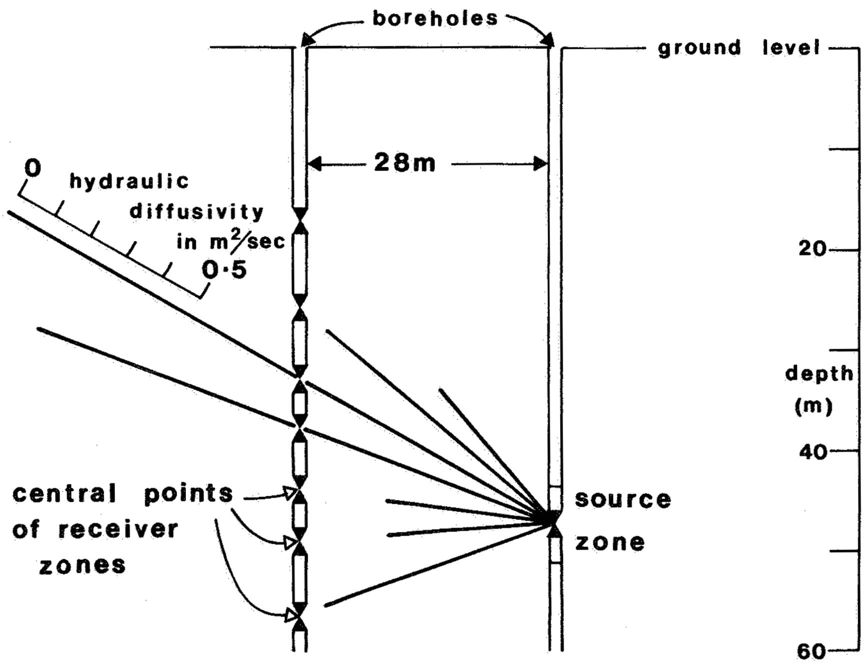

Fig. 4 Results of sinusoidal pressure tests plotted in terms of directional hydraulic diffusivity vectors

and slab thickness.
The results of the trial are shown in Figure 4 in terms of the hydraulic diffusivity measured in the direction between the source zone and each receiver zone.
The trial thus showed that a sinusoidal wave test is possible, that it yields measurable fluctuations over significant distances in fractured granite and gives directional results. It was notable that the measured fluctuations died away within one cycle of stopping the source cycling thus allowing the source or receiver zone to be moved relatively rapidly. In comparatively low hydraulic conductivity rock such as fissured granites the amount of water stored at the surface was in the order of a few tens of litres for a cycle period of 13.2 minutes. Unfortunately borehole availability did not allow either a change of source zone or the use of a range of different frequencies.

4 Conclusions

Work in fissured, low permeability rocks has focussed attention on the way in which interaction between fissures and matrix affects the results from slug tests and sinusoidal pressure tests. It can be seen that the interaction has a major effect on the specific storage results derived from slug tests and a minor effect on the hydraulic conductivity results. Calculations show that the same effects can be expected in results from fissured aquifers such as the Chalk of the U.K.
The sinusoidal pressure test is introduced as a method to yield detailed results between boreholes on such topics as directional hydraulic conductivity, matrix properties and system geometry. Additionally there are practical benefits such as the ability to carry out such tests against a background pumping regime and the need only to store a small amount of water on surface without the requirement of either a water supply or a discharge main. The best ways of analysing data from such tests are only just being developed but ambiguity of interpretation is reduced to a minimum if a number of frequencies are used. In fissured porous media it should be possible, at certain distances, to use a "cocktail" of frequencies with some penetrating the matrix and others not. Further work is concentrating on these aspects.

Acknowledgements

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References

Barker, J.A. and Black, J.H. Slug tests in fissured aquifers. Submitted to Water Resources Research.

Black, J.H. 1981. Nuclide transport models: a step towards relevant field testing. Proc. Rockstone '80 Stockholm, June 1980 3: 1325-1329.

Black, J.H. and Kipp, K.L. Jr. 1981. Determination of hydrogeological parameters using sinusoidal pressure tests: a theoretical appraisal. Water Resources Research. 17: 686-692.

Black, J.H. and Barker, J.A. 1982. Transient hydraulic tests in granite: fissured porous medium analysis and results. Materials Research. Soc. meeting (1981 Boston, Mass.), Scientific basis for nuclear waste management, V.6 : 223-230.

Black, J.H. and Holmes, D.C. Hydraulic testing in granite using the sinusoidal wave method. Rep. Inst. Geol. Sci. ENPU 82-3 in press.

Cooper, H.H. Jr., Bredoheoft, J.D. and Papodopulos, I.S. 1967. Response of a finite diameter well to an instantaneous charge of water. Water Resources Research. 3(1): 263-369.

Holmes, D.C. 1981. Hydraulic testing of deep boreholes at Altnabreac: development of the testing system and initial results. Rep. Inst. Geol. Sci. ENPU 81-4.

ELECTROMAGNETIC RESISTIVITY PROFILING FOR THE DETERMINATION OF LATERAL VARIATIONS IN LITHOLOGY OR GROUNDWATER QUALITY IN THE NETHERLANDS

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Abstract

Spatial hydrogeological variations, like lateral transitions in lithology or groundwater quality, in many cases are connected with lateral variations in subsurface electrical resistivity. These variations can be identified by either direct current (DC) or electromagnetic (EM) resistivity methods, with which the (apparent) resistivity of the subsoil is determined at the surface. In general EM resistivity profiling is faster and cheaper than DC resistivity profiling. In the present study two different EM resistivity measuring devices, a horizontal or vertical loop (HL/VL) induction instrument (EM34-3) and a very low frequency (VLF) radio wave instrument (EM16R) have been used. The results obtained with these instruments are in good agreement with those obtained by DC measurements and borehole information. Four examples of the application of both the HL/VL and VLF resistivity methods in the Netherlands will be discussed. They were applied to detect a shallow fresh groundwater lens in a saline area, to locate a conductive clay layer and to map groundwater pollution plumes around two waste disposals. The examples prove to be successful applications of EM resistivity profiling and mapping.

1 Introduction

The direct current (DC) or geo-electrical resistivity method has proved to be a successful tool for groundwater and geological investigations. Combined with geophysical borehole logging DC sounding has been frequently applied for regional groundwater reconnaissance in the Netherlands. In particular it has been used to examine the regional distribution of fresh, brackish and salt groundwater, and of sand and clay (loam and peat) in the Pleistocene and Holocene sediments in the Netherlands.
A combination of DC sounding and profiling has been used in several detailed hydrogeological surveys.
Recently, some successful examples of EM methods in hydrogeological investigations were published (Müllern and Eriksson, 1981 and Steward, 1982). In the Netherlands several experimental studies with the horizontal or vertical loop (HL/VL) resistivity (conductivity) method have been carried out (Van Dongen, 1979, Hoogeveen et al., 1980 and Overzee, 1981).

Figure 1 Location of survey areas Grijpskerke (1), Marum (2), Noordwijk (3), and Utrecht (4)

Thusfar the Very Low Frequency (VLF) radio wave resistivity method has not been used for groundwater investigations in the Netherlands. The paper shortly treats both EM resistivity methods and describes their utility to determine lateral changes in groundwater quality and lithology by means of the results obtained in four different survey areas in the Netherlands (Figure 1).

2. Electromagnetic resistivity methods

EM methods have been used in mineral exploration for half a century. The various EM methods and their theories of operation are desribed in several handbooks. EM methods use time varying, magnetic and electric fields. Under certain conditions (low induction number, magneto-telluric configurations) the apparent electric resistivity (or conductivity) of the earth can be calculated from measurable components of (transmitted or received) magnetic or electric fields.
Because of the attenuation of EM fields in conductive media EM fields have a certain (effective) penetration depth or skin depth for a homogeneous earth. The skin depth represent the exploration depth (for a homogeneous earth) of "source" limited EM resistivity methods, which use large transmitter-receiver spacings. The exploration depth of a source limited method depends on the frequency of the EM fields used and the resistivity of the earth. For EM resistivity methods where the transmitter-receiver spacings are much smaller than the skin depth for all frequencies used and resistivities of the earth (condition of low induction number) the exploration depth is "geometry" limited. It depends on the transmitter-receiver spacing and configuration.
When horizontal layering occurs within the exploration depth of the EM resistivity method, the measured resistivity no longer represents the true resistivity of the earth. It is called the apparent resistivity.
The HL/VL resistivity instrument used is the EM34-3 of Geonics, which has been described by McNeill (1980). It consists of two portable coils: one to transmit a magnetic field and another to receive this and the magnetic field excited by conductors in the earth (Figure 2). The EM34-3 is so deviced that the condition of low induction numbers

exists for both horizontal and vertical coplanar loops. Then the apparent resistivity of the earth can be derived from the ratio of the secondary and primary magnetic field at the receiver.

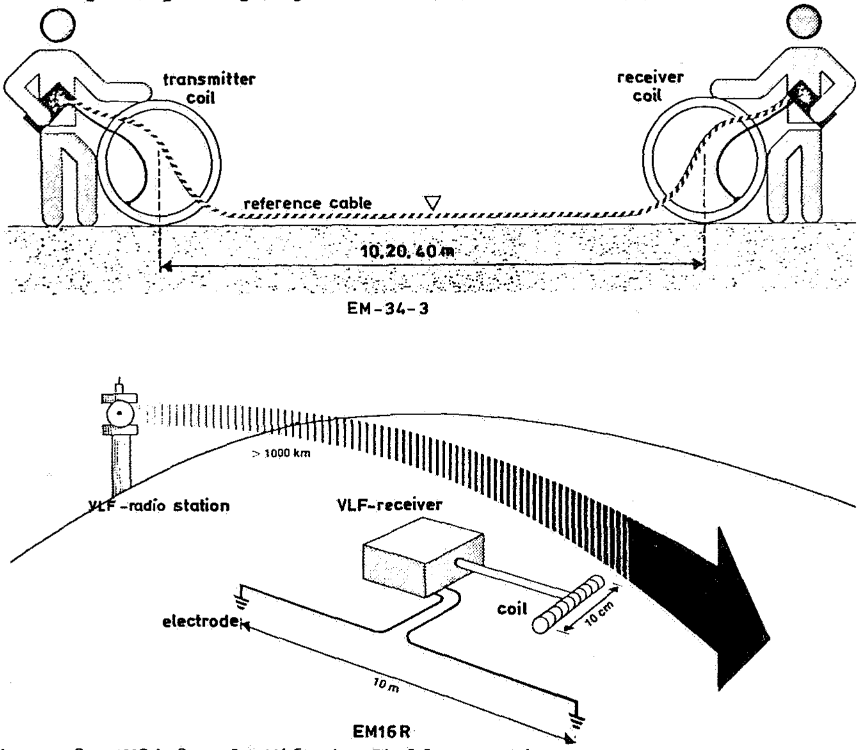

Figure 2 EM34-3 and EM16R in field operation

When the determined apparent resistivities are low this relation no longer holds. Therefore, the measured conductivities can be corrected. However, when the apparent resistivity of the earth falls below 5 ohmm the HL loop mode cannot be used properly.
The determined resistivities apply to the geometry limited exploration depth. The possible exploration depths of the EM34-3 depend on the intercoil spacing (and the respective frequency) and the orientation of the coils. They are shown in Table 1.

Table 1 Possible penetration depths for the EM34-3

Intercoil spacing frequency (m, Hz)	Exploration depth VL (m)	HL(m)
10, 400	7,5	15
20, 1600	15	30
40, 6400	30	60

With the EM34-3 one hundred HL/VL resistivity measurements a day can be carried out by two persons for local hydrogeologic surveys.

The VLF radio wave resistivity instrument, described by Collett (1968), is the EM16R of Geonics. It uses very low frequency radio waves (15-20 kHz) transmitted from distant stations as the primary magnetic field. The EM16R is mainly a radio receiver, measuring the ratio and the phase angle between the horizontal electric and magnetic fields. When the instrument is well oriented with respect to the VLF radiostation (Figure 2), the apparent resistivity of the earth can be derived from the ratio between the horizontal electric field in the direction of the radio station and the horizontal magnetic field perpendicular to that direction using a magneto-tellurical relation. The exploration depth of the VLF method is source limited, therefore, it equals the skin depth. For a fixed radiostation with appropriate frequency the skin depth only depends on the resistivity of the earth (Table 2).

Table 2 Exploration depths of the EM16R (f=20kHz) for various resistivities

Resistivity (ohmm)	Exploration depth (m)
10	12
50	25
100	35
500	80

With the EM16R one person can make about 125 measurements a day for a local survey.
With both HL/VL and VLF resistivity measurements a limited two-layer interpretation is possible. Two-layer interpretations are not made, because of the complex situations at the investigation site and the fact that with DC sounding better results are achieved. Both EM resistivity methods are merely used to do resistivity profiling and mapping.

3 Resistivity profiling and mapping

EM resistivity methods measure, like DC methods, apparent resistivities. When the condition of horizontal homogenity is met or when slow lateral variations are present, they are constant or change regularly and smoothly. When lateral changes over a short distance occur, anomalous apparent resistivities are measured, which by themselves can be used as diagnostic features. This paper treats measurements over targets, which slowly vary laterally as compared to the exploration depth of the resistivity method.
It is of utmost importance to tune the exploration depth of the resistivity method to the depth of the lateral variations to be examined.
The optimum exploration depth for the survey is the depth for which the lateral variations show up best in the measurements. To find the optimum depth for DC resistivity profiling in general DC sounding curves are used, which are measured or calculated for both sides of the transition zone. The depth at which these curves differ most can be taken as the optimum exploration depth for DC resistivity profiling and mapping. In the same way using EM sounding curves for both sides of the transition zone, the optimum exploration depth should be determined for HL/VL resistivity profiling. This method has not been followed. Instead the optimum exploration depth for DC profiling has been used as starting point to choose the exploration depth for HL/VL profiling. As far as VLF resistivity profiling is concerned, it can only be checked if the exploration depth is sufficient. It is advisable to conduct surveys combining many EM resistivity measurements

and a few well-chosen direct current soundings. In this way not only information about the horizontal changes but also about the vertical changes of the resistivity is obtained.

4 Results in four survey areas

4.1 Shallow fresh and brackish groundwater in a saline area (Grijpskerke)

Underneath the islands of the southwestern Netherlands only rarely fresh ($Cl^- < 150$ mg/l) and brackish ($150 < Cl^- < 1000$ mg/l) groundwater occurs. Except for the coastal dunes only Holocene (sand-filled) channels are areas of local infiltration of fresh groundwater. The last are creekridges and are slightly elevated (1-2 m) because of the setting of the surrounding Holocene clay and peat deposits, which are thin (0-3 m) over the Holocene channels and relative thick (5-10 m) besides them. The fresh and brackish groundwater present under these creekridges may be of local importance as a groundwater source for domestic use.

VL and VLF resistivity measurements have been conducted (Ritsema, 1982) over a creekridge near Grijpskerke (Figure 1). Because of the low resistivities of the salted lowlands (< 10 ohmm) no HL measurements have been carried out. The VL and VLF apparent resistivity profiles (Figure 3) show a maximum at the creekridge. The VL measurements with 7.5 m penetration depth and the VLF measurements are the most clear. The lower part of Figure 3 shows the interpretation of the electromagnetic resistivity measurements, three DC soundings and three boreholes. Under the creekridge a thin lens (< 5 m) of fresh groundwater exists. Underneath this lense and above the salted sediments a brackish transition zone (< 15 m) is present.

This example clearly shows that VL and VLF resistivity measurements, combined with a limited number of DC soundings can be used to map shallow fresh and brackish groundwater in salted areas.

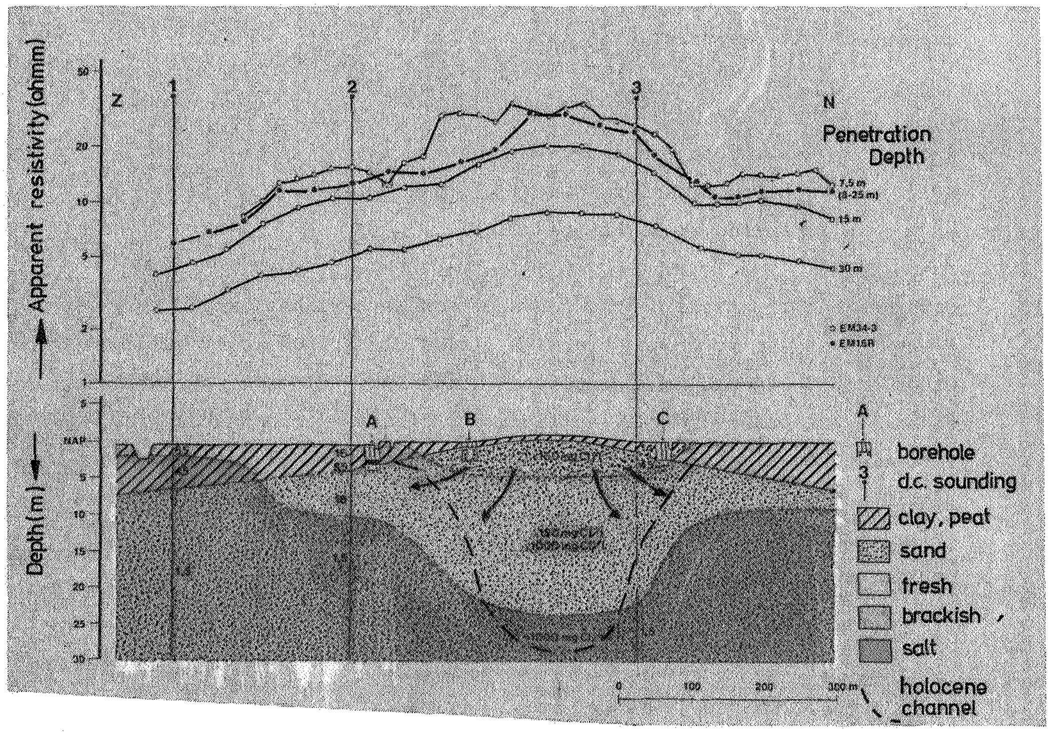

Figure 3 Resistivity profiles (EM34-3, 10, 20 and 40 m, vertical and EM16R) over a shallow fresh water lens in a creekridge at Grijpskerke

4.2 Conductive clay (Marum)

In the northern Netherlands near Marum Pleistocene glacial deposits occur in a wide channel-like structure. Besides sand (60-100 ohmm) these sediments contain at the top a very conductive clay layer (about 10 ohmm), known as potclay, with varying thickness (0-50 m). Because of its high vertical flow resistance it is interesting to know the location and thickness of the potclay. At the request of the Groningen Water Supply five HL/VL (s=20 m) profiles combined with 12 DC soundings have been executed to delineate the west side of the potclay occurrence (Ritsema, 1982). The results of profile 2 is given in Figure 4.

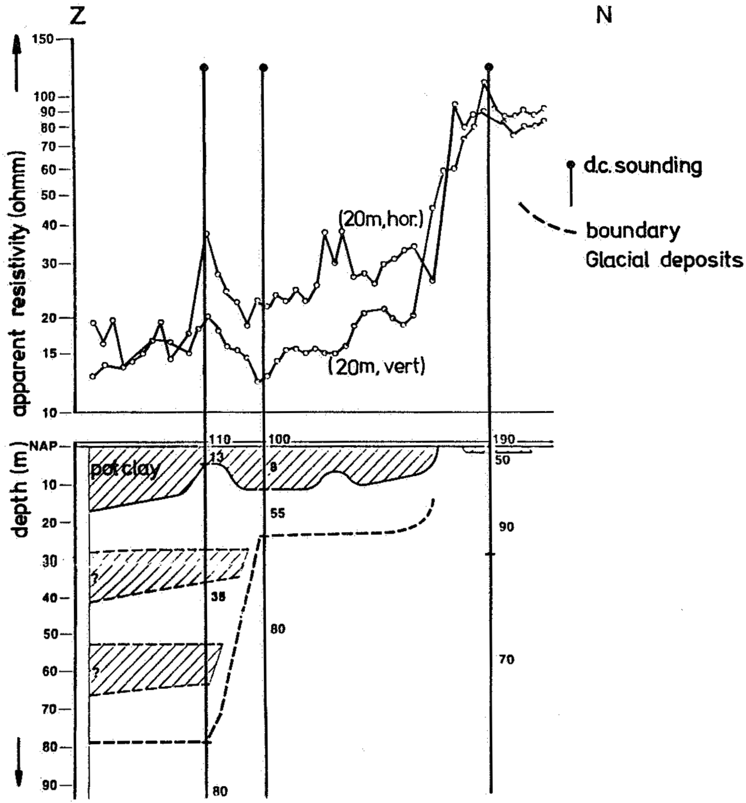

Figure 4 Resistivity profile (EM34-3, 20 m, horizontal and vertical) to locate a conductive clay layer

The sudden increase in resistivity to about 100 ohmm indicates the limit of the potclay formation. The effect of the clay layers occurring deeper than 30 m, is not sensed by the EM resistivity measurements. Though these clay layers are interpreted from DC soundings and borehole information. The DC soundings and the HL/VL profiles resulted in the resistivity map given in figure 5. It clearly shows the limits of occurrence of the potclay and gives a qualitative impression of its thickness.

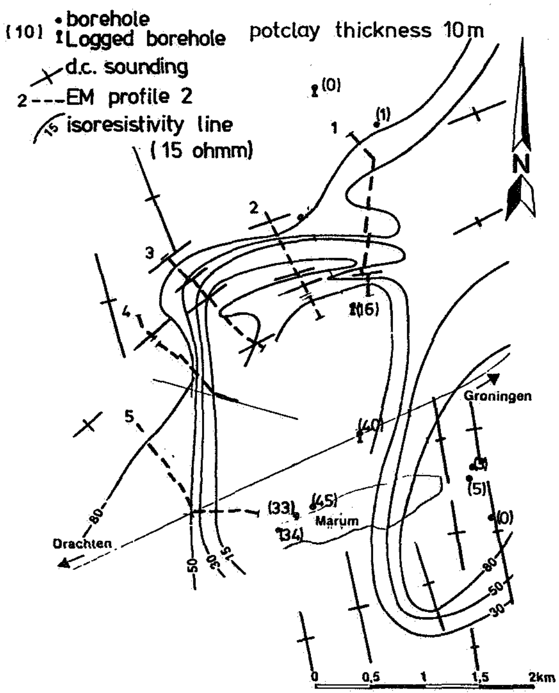

Figure 5 Resistivity map (EM34-3, 20 m vertical) to locate a conductive clay layer at Marum

4.3 Groundwater pollution plume around a waste disposal (Noordwijk)

The investigated waste disposal site lies partly beneath the phreatic groundwater level. It has been built up to about 4 m above surrounding surface, causing a local elevated groundwater level. The fresh water-salt water boundary lies at a depth of 40 m below surface. Above this level contaminated groundwater follows two separated streampaths. One through the upper 15 m, which consists of fine sand and thin loam layers, and another one at about 35 m in the main aquifer, which consists of coarse sand. HL/VL and DC resistivity measurements, conducted for the National Institute for Water Supply in the Netherlands (Overmeeren, 1979, Van Dongen, 1979 and Ritsema, 1981), show that the apparent resistivities vary from 10 ohmm near the waste disposal to about 35 ohmm farther away, caused by groundwater contamination within the upper 15 m. The VLF resistivity measurements (Figure

6) show the same pattern as that obtained by the other methods. Low resistivities (< 10 ohmm) occur at all sides near the waste disposal, due to divergent flow, because of the elevated groundwater level. Further away the values become higher, till they reach the normal or background value of the area (35-40 ohmm). The shallow pollution plume is larger to the south-east, due to the regional groundwater flow. The results have been confirmed by analysis of groundwater samples.

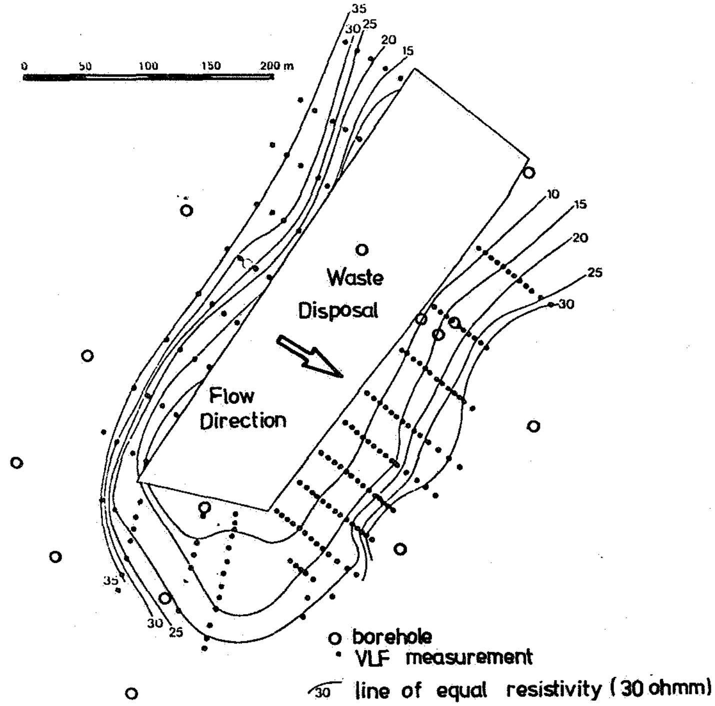

Figure 6 Resistivity map (EM16R) to locate the pollution plume around a waste disposal at Noordwijk

4.4 Groundwater pollution plume around a waste disposal (Utrecht)

This HL/VL resistivity survey is carried out for the Provincial Department for Groundwater in Utrecht. It was the first step in examining the possible influences of the waste disposal on the groundwater quality. The waste disposal is situated beneath the phreatic groundwater level. To a depth of about 40 m a sand aquifer occurs. At the surface a thin clay layer exists. There was some uncertainty about the direction of groundwater flow. It could be westwards according to the regional flow direction, or eastwards influenced by a large draining channel located in that direction. Figure 7 shows clearly that low apparent resistivities occur mainly at the east side of the waste disposal. It is concluded tha the groundwater is contaminated and streams eastwards to the draining channel. This result is confirmed afterwards by the analysis of groundwater samples from a proposed borehole location.

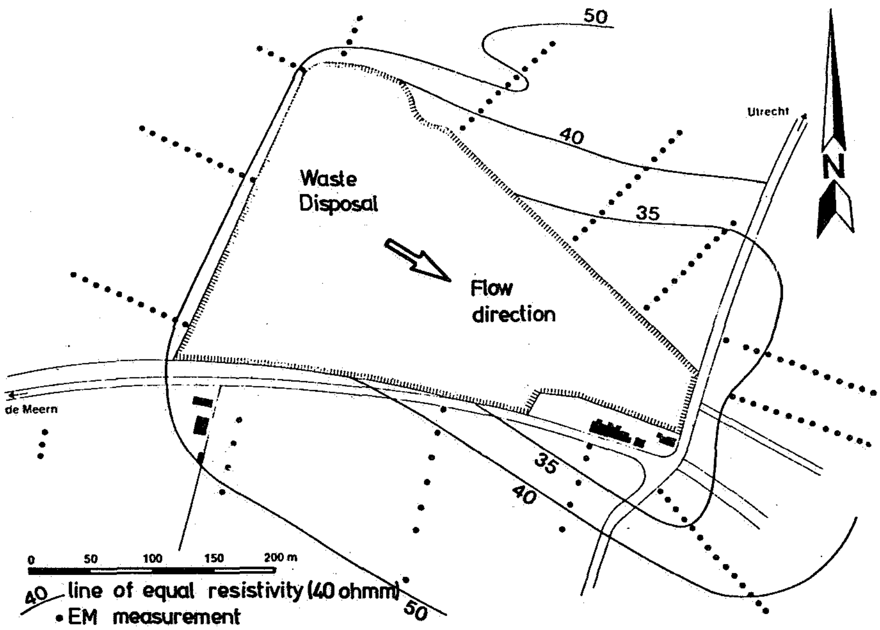

Figure 7 Resistivity map (EM34-3, 40 m, vertical) to locate a pollution plume around a waste disposal at Utrecht

The last two examples clearly show that electromagnetic pollution plume mapping can diminish the costs of realising a monitoring network.

Acknowledgement

The Provincial Department of Groundwater of Utrecht and Zeeland, the Groningen Water Supply and the National Institute for Water Supply in the Netherlands are thanked very much for their permission to publish these data and their assistance during the surveys.

References

Collet, L.S. and A. Becker 1968. Radio-ohm method for earth resistivity surveying. Canadian Patent, no. 795.919.

Dongen, P. van 1979. Proefonderzoek met elektromagnetische apparatuur EM34-3. Groundwater Survey TNO, OS 76-06, Delft.

Hoogeveen, H.H. and H.A. Vermeij 1980. Verslag van een onderzoek naar de toepasbaarheid van een elektromagnetische methode EM31 in het hydrogeologisch onderzoek in Nederland. Free University, Amsterdam.

McNeill, J.D. 1980. Electromagnetic terrain conductivity measurements at low induction numbers, TN-6, Geonics Ltd., Canada.

Müllern, C.F. and L. Eriksson, L. 1981. Testing VLF measurements in order to locate saline groundwater. Proceedings SWIM 1981, Sweden.

Overmeeren, R.A. van 1979. Geo-elektrisch onderzoek vuilstort Noordwijk, GF 112, Groundwater Survey TNO, Delft

Overzee, B. 1981 Toepasbaarheid van elektromagnetische methoden bij de geologische kartering, 81-15, Rijk Geologische Dienst, Haarlem.

Ritsema, I.L. 1981. Geo-elektrisch en elektromagnetisch onderzoek rondom vuilstortplaats Noordwijk. GF-135, Groundwater Survey TNO, Delft.

Ritsema, I.L. 1982. Bepaling van eventuele grondwaterverontreiniging met behulp van geo-elektrisch en elektromagnetisch onderzoek in de provincie Utrecht. OS 82-01, Groundwater Survey TNO, Delft.

Ritsema, I.L. 1982. Geo-elektrisch en elektromagnetisch detailonderzoek naar de verbreiding van ondiepe kleiige afzettingen rondom Marum. OS 82-08, Groundwater Survey TNO, Delft.

Ritsema, I.L. 1982. Onderzoek naar de bruikbaarheid van elektromagnetische methoden (EM34-3, EM16R) voor het lokaliseren van ondiepe zoet- en brakwatervoorkomens in Zeeland. OS 82-19, Groundwater Survey TNO, Delft.

Stewart, M.T. 1982. Evaluation of electromagnetic methods for mapping of salt-water interfaces in coastal aquifers. Groundwater 20, pp. 538-545.

ELECTROMAGNETIC PROFILING IN THE INVESTIGATION OF SMALL SCALE GROUNDWATER FLOW SYSTEMS

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Abstract

The hydrogeological application of electromagnetic measurements on hard rock areas of western Africa and for a sedimentary area in The Netherlands is discussed.
On the Niger and Upper Volta Pre Cambrian Shield groundwater in exploitable quantities is found mainly in the bedrock within narrow zones associated with fractures.
Identification of these fractures in the field using EM profiling has greatly improved the success ratio of new boreholes.
In The Netherlands electromagnetic profiling has been used to study the spatial distribution of contaminated groundwater and the method proved useful when sufficient contrast in conductivity between contaminated and natural water was present.
The speed with which results may be obtained gives the technique a particular advantage over conventional resistivity measurements.

Resumé

L'Application hydrogéologique de mesures électromagnétiques dans des roches cristallines en Afrique de l'Ouest et dans une zone sédimentaire des Pays Bas est discutée.
Dans le socle Pré Cambrien de Niger et de Haute Volta, on trouve de l'eau souterraine en quantité exploitable dans des roches cristallines liées à des zones de fracture.
La localisation des fractures sur le terrain par des profils électromagnétiques a énormément amélioré le pourcentage de succès des nouveaux sondages.
Aux Pays Bas, la méthode électromagnétique est appliquée à l'étude de la distribution spatiale d'eau souterraine contaminée. La méthode donne des bons résultats quand le contraste en conductivité entre l'eau contaminée et l'eau naturelle est suffisant.
Cette méthode présente de grands avantages sur les mesures de résistivité conventionnelles en ce qui concerne la rapidité d'exécution.

1 Introduction

Geophysical methods are normally used for investigation of groundwater flow systems in IWACO projects.
In addition to the widely accepted methods such as refraction seismics, geo-electrical methods and borehole geophysics, another method was introduced three years ago, which until then was only infrequently used for groundwater studies i.e. the electromagnetic (EM) multi-frequency horizontal loop and vertical loop profiling method. This EM method is currently being used within sedimentary areas of The Netherlands and for hard rock areas in western Africa.
Experiments with the EM method were initiated in Upper Volta in collaboration with the University of Utrecht (Palacky et al, 1981).
These proved succesful and the method was consequently accepted as a means of groundwater exploration in hard rock areas, at least in part replacing geo-electrical techniques.
With the availability of new equipment further experiments were carried out in The Netherlands to determine whether EM methods could also be

used in studies involving groundwater quality problems in sedimentary formations. The recent discovery of numerous industrial waste dumps, created in the post-war economic boom, made it necessary to find a rapid method by which the lateral distribution of polluted groundwater could be defined. The EM profiling method has proven its usefulness in these studies.

2 Principles of the EM method and instrumentation.

2.1 Theory

The objective of EM measurements is to locate underground conductors. In hydrological exploration these conductors may represent local groundwater occurrences in fractured zones in the substratum (Upper Volta) or the occurrence of groundwater with relatively higher electrical conductivity due to contamination (The Netherlands).

All equipment systems in present use are completely portable and consist of a transmitter which provokes an alternating electromagnetic field (primary field) and a coplanar receiver connected with the transmitter at a fixed distance. The receiver and transmitter coils can be used both vertically and horizontally (horizontal loop and vertical loop).

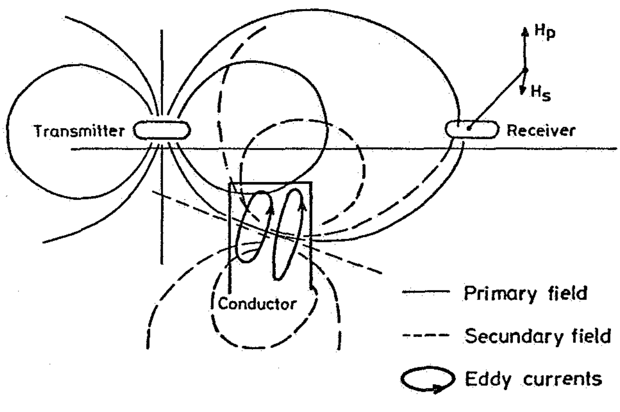

Figure 1 Primary and secondary electromagnetic field. (after Grant and West, 1965)

The primary field generates eddy currents in an underground conductor within its zone of influence, which in turn generates a secondary electromagnetic field (figure 1). The receiver separates the secondary from the total field and displays this as a percentage of the primary field, either as an in and out of phase (quadrature) component or as apparent conductivity.
EM response of a conductor and depth of penetration depend on the coil spacing, frequency and on the electrical and magnetic characteristics of the conductor (Telford et al, 1976).

2.2 Instruments

Three instruments are at present in use: the Apex Max Min from Parametrics Canada, the Slingram 3.6 Khz horizontal loop unit from the Swedish Geological Survey and the EM 34-3 from Geonics, Canada.
With the Apex Max Min five frequences can be chosen between 222 and 3555 Hz, while cables for coil separations of 25, 50, 100 and 200 m are available. The frequency is fixed for the Slingram unit and coil separations of 50 and 100 m are available. The EM 34-3 uses a fixed frequency for three different coil separations of 10 m (6400 Hz), 20 m (1600 Hz) and 40 m (400 Hz).
The Apex and Slingram units measure in and out of phase components separately as an percentage of the primary field, while the EM 34 directly measures the apparent electrical (magnetic) conductivity.

3 Application of EM profiling in Upper Volta

The centre and eastern part of Upper Volta and the western part of Niger are almost completely underlain by igneous and metamorphic rocks of the Pre-Cambrien Shield.
Relief of this old peneplain is rather flat and a thick weathered zone is present, consisting of (sandy) clay and sometimes a well developed lateritic crust. Thickness of the weathered layer generally varies between 0 and 30 m, but may occasionally reach 100 m.

The transition zone from clayey alterite to bedrock has a thickness of 0 to 10 m with both clay and transition zone generally characterized by high porosity but relatively low primary permeability.

Primary porosity of the bedrock is negligible but due to fracturing a sometimes important secondary porosity may be present. The weathered and transition zones in the fractures are thicker and may be more coarsely developed.

The location of these fractures, first with the aid of aerial photographs followed by geophysical surveys in the field, is hence the most important activity in Upper Volta groundwater exploration. As demonstrated by Palacki (1981), the inprecise location of a borehole by only a few meters can make the difference between a good water supply (depending on the area, > 1 m^3/hr) and a poor one.

EM profiles allow the presence of fractures to be readily verified and the exact location of boreholes in the field established.

Two examples of the method applied to urban water supply studies are worthy of mention - Boussé and Toma. The required yields exceed 2 m^3/hr as the wells are to be equipped with engine driven pumps. In Boussé (a migmatite area) the groundwater occurrence was determined by differences in bedrock lithology. For Toma, underlain by granites, the groundwater was found in fractured zones.

3.1 Boussé

In Boussé a study of aerial photographs showed an area with important lineaments which were not visible in the field. Geophysical survey in the area consisted of various EM and geo-electrical profiles of which one is shown in figure 2. The advantage of EM profiling over conventional resistivity profiling is obvious in this figure in that the EM profile clearly shows two anomalies, while the resistivity profile indicates only one broad anomaly. Subsequent drilling showed the conductors to be fractured amphibolite bands in magmatites; boreholes in the amphibolite gave yields of 2 and 7 m^3/hr while in the granite only 0,5-1 m^3/hr was achieved. The direction of anomalies, as deducted from the EM and resistivity profiles, is more or less identical with that for the lineaments deducted from aerial photos, but with differing locations.

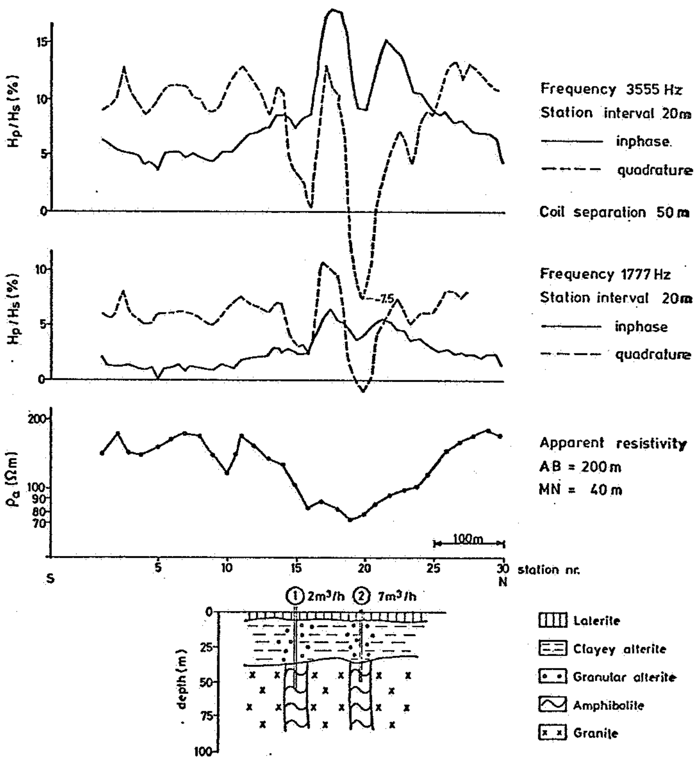

Figure 2 EM and resistivity profile of Boussé

3.2 Toma

In Toma the region for an extensive geophysical survey was also selected with the help of aerial photographs. Vertical electrical soundings showed a thick weathered layer of 40 to 50 m. Various EM and geo-electrical profiles were carried out as well as "star" measurements, in order to establish the direction of anomalies.

Figure 3 presents part of the apparent resistivity map as deducted from these profiles, together with the EM and resistivity data of one section (P7).

Two well marked low resistivity anomalies are evident.

Anomalies on the EM profile are less pronounced than for the previous example, probably due to the thick weathered zone and to a smaller contrast between the conductor and surrounding rocks.

Since the anomaly in the South has a larger extension than the other it was chosen for test drilling and two boreholes producing 4 and 8 m^3/hr were drilled. A third, reconnaissance borehole was drilled in the high resisitivity zone south of borehole 1; the yield of this proved to be less than 1 m^3/hr.

The two good boreholes show a thick transition zone with coarse grained quartz elements and with a fractured and lightly weathered bedrock. The borehole in the high resistivity zone has a weathered zone which is more clayey and a thin transition zone, with no sign of fracturing in the bedrock. The low resistivity anomalies are therefore ascribed to faults in zones with coarser grained granite.

In this example the geophysical anomalies also do not correspond with aerial photo lineaments, though the direction is essentially similar.

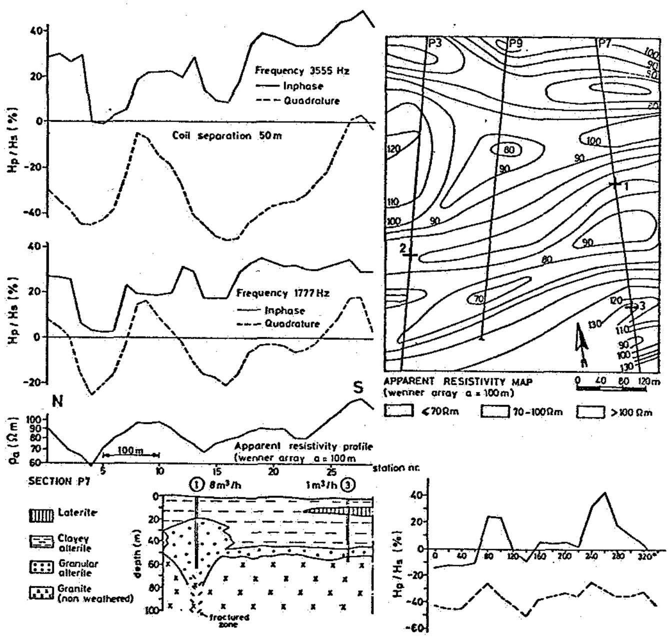

Figure 3 EM and resistivity profile of Toma

4 Application of EM profiling in The Netherlands

The electro-magnetic method was further tested with respect to two groundwater pollution problems in The Netherlands, in co-operation with the department of hydrogeology of the Amsterdam Free University.
In both instances the spatial distribution of contaminated groundwater has to be outlined in order to establish an optimum drilling program.
An EM 34-3 instrument was used in vertical loop mode.

4.1 Hierderdensche Beek

In the Hierdensche Beek district the animal industry produces quantities of manure which greatly exceed those needed for agriculture.
Part of this overproduction of manure is dumped on areas used for maize and grass growing, causing superficial groundwater pollution.

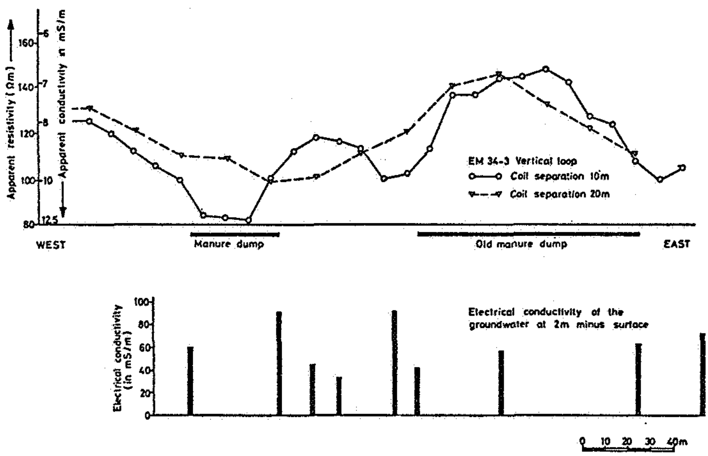

Figure 4 EM data and groundwater resistivities over a manure dump in the Hierdensche Beek area (after Verhoeff, E.K., 1982)

The extent of pollution in one such area was invetigated by EM and geo-electrical methods. Figure 4 presents a section through the dumping area with the EM data (EM 34-4) generated using coil separations of 10 and 20 m. Anomalies of the 10 m separation are more enhanced than for 20 m, indicating contamination to be present mainly in the superficial groundwater. The measured resistivity values correspond well with water resistivities obtained for samples from a subsequent drilling program.

4.2 Solid waste dump

The possiblity of groundwater pollution in a solid water waste dump in the eastern part of the country was again investigated using EM 34-3. Since the phreatic groundwater level lies about 10 m below surface a coil separation of 20 m was chosen. Figure 5 shows an EM section located about 30 m from the dump in the direction of regional groundwater flow and indicates a sharp anomaly at the dump. A borehole was drilled to a depth of 40 m in the centre of this anomaly and was equipped with mini screens from which groundwater samples could be extracted. These are now being analyzed for trace elements. Groundwater conductivities from this borehole show a sharp decrease from about 25 mS/m (40 Ohmm) to 8 mS/m (125 Ohmm) at a depth of 18 m, i.e. a ratio of about 3.
Resistivities at the centre of the anomaly also show a ratio of about 3 when compared with those to the west of the dump.

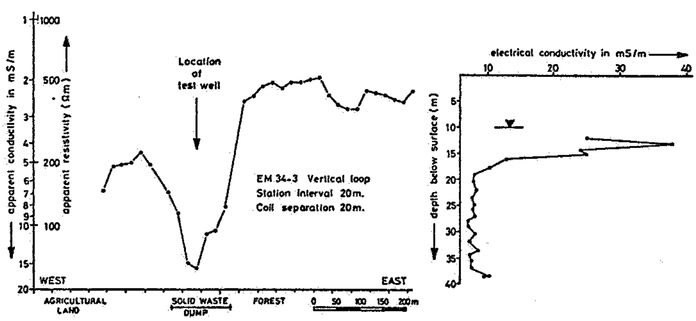

Figure 5 EM section over a solid waste dump and electrical conductivity of groundwater samples from an exploration borehole

The higher resistivities east of the dump are probably due to the presence of forest, while in the west the land is mainly used for agriculture.

5 Conclusions

The advantage of EM profiling over only resistivity profiling in hard rock areas is that in the case of narrow vertical conductors a much sharper anomaly is produced, thus allowing more accurate field emplacement of boreholes.
This has resulted in a success record of over 75% for recent IWACO projects in hydrogeologically difficult areas in Niger and Upper Volta.
However, resistivity soundings and profiling are still necessary in order to obtain an insight into the depth to and character of the bedrock.
Quantitative interpretation of EM measurements is still difficult.
Success of the EM method in groundwater contamination studies in sedimentary formations depend on the contrast in electrical conductivity between the contaminated and natural groundwater.
In the two cases described the contrast was about 3 and the lateral distribution of contaminated groundwater could be established.
Finally, the rapidity with which results may be obtained gives the method particular attraction over conventional resistivity techniques.
However, in urban areas both methods are liable to disturbance from industrial noise.

References.

Grant, F.S. and W.H.S. West, 1965. Interpretation theory in applied geophysics, Ms Grawhil, New York.

Palacky, G.J. et al, 1981. Electromganetic prospecting for groundwater in Precambrian terrains in the Republic of Upper Volta. Geoph.Prosp.29, pp. 932-955.

Telford, W.M. et al, 1976. Applied Geophysics, Cambridge Univ.Press.

Verhoeff, E.K. 1982. Onderzoek naar het voorkomen van vervuild grondwater als gevolg van overbemesting. (Internal report Amsterdam Free University, Institute of Earth Sciences).

ESTIMATION OF PARAMETERS IN GROUND-WATER FLOW PROBLEMS USING A KALMAN FILTER ALGORITHM

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Abstract

Mathematical models for groundwater flow problems contain a number of parameters. In most cases these parameters must be known to obtain solutions of the model equations. With the aid of the Kalman filter algorithm, it is possible to estimate simultaneously the model parameters and the solution of the model equations. The Kalman filter algorithm is an optimal estimation algorithm in a least squares sence (see Sage and Melsa (1971)). In this paper two possible formulations of the Kalman filter algorithm for the parameter estimation problem are presented. As an application a theoretical flowproblem has been worked out.

1 Introduction

The watermovement through the subsoil can be described with the aid of the system theory,in which there are system variables, system parameters and systeminput. The systemvariables, in this paper piezometric heads, can be measured by a measurement network. To evaluate the quality of this network one needs an interpolation technique for estimation of the variables on places where they are not measured. An advanced interpolation technique is the Kalman filter algorithm (Kalman (1960)). This filter algorithm is based on the state-space description which is well known in (linear) system theory (Sage and Melsa (1971)). Some examples on the use of this interpolation technique can be found, among others, in O'Connell (1980), Van der Kloet et.al. (1982) and Van Geer (1982). The interpolation algorithm gives an, in least squares sense, optimal estimate of the state of the system. In each particular problem this state has to be defined in order to use the algorithm. In groundwater flow problems, for example, this state can be defined as a vector, of which the components are phreatic groundwater levels and piezometric heads. In groundwater flow problems the change of the state of the system with time can be deduced from Darcy's law, the equation of continuity and the system input. Hence some parameters are introduced which may or may not be known in advance. If they are not known, they must be estimated. This can be done with the Kalman filter algorithm by enlarging the state of the system. In this paper two parameter estimation methods are demonstrated for a simple model with only one system parameter.

In section 2 a groundwater flow problem is described in a state-space formulation. In section 3 two estimation methods for the enlarged state are given. In subsection 3.1 a successive method, called 'single parameter estimation' is described. First the state is estimated and next the parameter. In subsection 3.2 a simultaneous method is described with which the parameter and the state of the system are estimated at the same time. In these subsections only the formulae are given and discussed. For a detailed derivation of the formulae see Van Geer and Van der Kloet (1982). In section 4 the calculations and the results obtained are given for the chosen example. In the calculations no real-world data are used, but synthetically generated data. Finally in section 5 the results are discussed.

2 Hydrological problem

A strip of land of infinite length is considered, which is bounded by two watercourses, also of infinite length (Figure 1, definition sketch). The strip of land consists of a confined aquifer above an impermeable base. The groundwater flow is one-dimensional only and the positive flow direction coincides with the positive ξ-direction. There are two observation wells at distances L_1 and L_2 from the watercourse at the lefthand

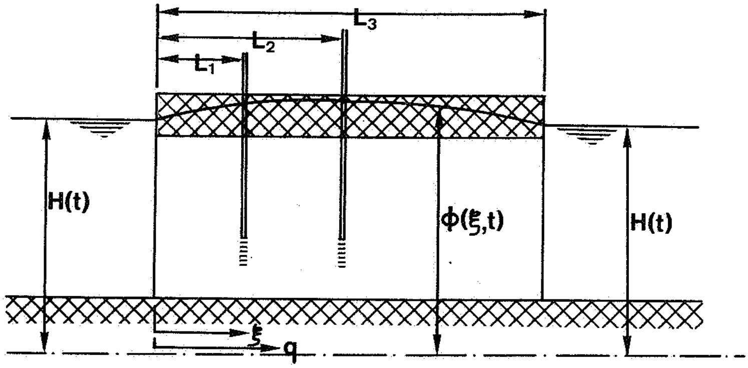

Figure 1 Definition sketch for the chosen example

side. The waterlevels in the watercourses are assumed to be known exactly as a function of time (Figure 2).

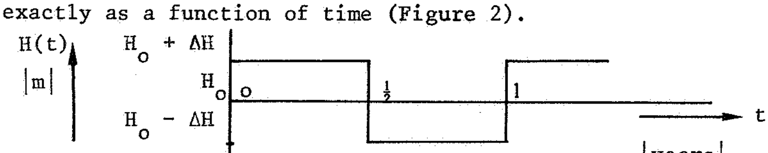

Figure 2 Waterlevel in the watercourses

The equation of continuity and Darcy's law can be combined, which yields:

$$\frac{d\phi(\xi,\ t)}{dt} = \frac{T}{S}\,\frac{d^2\phi(\xi,\ t)}{d\xi^2} \tag{1}$$

where

ϕ = head

T = transmissivity

S = storage coefficient

In order to obtain a state-space formulation with time as a discrete variable, the differentials are approximated by

$$\frac{d\phi(i\Delta\xi,\ k\Delta t)}{dt} \approx \frac{\phi(i\Delta\xi,\ k\Delta t + \Delta t) - \phi(i\Delta\xi,\ k\Delta t)}{\Delta t} \tag{2}$$

and

$$\frac{d^2\phi(i\Delta\xi,\ k\Delta t)}{d\xi^2} \approx \frac{\phi(i\Delta\xi - \Delta\xi,\ k\Delta t) - 2\phi(i\Delta\xi,\ k\Delta t) + \phi(i\Delta\xi + \Delta\xi,\ k\Delta t)}{(\Delta\xi)^2} \tag{3}$$

where

$i = 2, 3 \ldots N - 1$

$k = 0, 1, 2 \ldots$

$\Delta t =$ the timestep

$\Delta\xi =$ the step in distance

Writing $\phi_{i,k}$ for $\phi(i\Delta\xi, k\Delta t)$, $\theta = T/S$ and $\alpha = \Delta t/(\Delta\xi)^2$ and substitution of (2) and (3) in (1) yields:

$$\phi_{i,k} = \theta\alpha(\phi_{i+1,\,k-1} - 2\phi_{i,k-1} + \phi_{i-1,k-1}) + \phi_{i,k-1} \qquad (4^a)$$

In the gridpoints 1 and N one has to adapt equation (4^a) because $\phi_{i,k}$ is not defined for $i = 0$ and $i = N+1$. However, one can use the input of the system, the waterlevels $H(t)$ in the watercourses, to obtain:

$$\phi_{1,k} = \theta\alpha(\phi_{2,k-1} - 2\phi_{1,k-1} + u_k) + \phi_{1,k-1} \qquad (4^b)$$

and

$$\phi_{N,k} = \theta\alpha(u_k - 2\phi_{N,k-1} + \phi_{N-1,k-1}) + \phi_{N,k} \qquad (4^c)$$

where

$u_k = H\{(k-1)\Delta t\}$

In the equations (4^a), (4^b) and (4^c) θ is assumed to be an unknown parameter to be estimated. The state-space description used in the Kalman filter is based on equations (4^a), (4^b) and (4^c). To obtain the filter formulation the state vector $\vec{\phi}_k$ is defined for this case by having as components the heads in the gridpoints. Now (4^a), (4^b) and (4^c) can be written as

$$\begin{bmatrix} \phi_{1,k} \\ \vdots \\ \vdots \\ \phi_{N,k} \end{bmatrix} = \begin{bmatrix} (1-2\alpha\theta) & \theta\alpha & & \\ \theta\alpha & \ddots & \ddots & \\ & \ddots & \ddots & \theta\alpha \\ & & \theta\alpha & \theta\alpha(1-2\alpha\theta) \end{bmatrix} \cdot \begin{bmatrix} \phi_{1,k-1} \\ \vdots \\ \vdots \\ \phi_{N,k-1} \end{bmatrix} \begin{bmatrix} \theta\alpha \\ \vdots \\ \vdots \\ \theta\alpha \end{bmatrix} \cdot [u_k] \qquad (5)$$

Or in vector denotation

$$\vec{\phi}_k = \Phi\vec{\phi}_{k-1} + \Lambda\vec{u}_k \tag{6}$$

where

Φ = state transition matrix

$\vec{u}_k$ = input

$\Lambda\vec{u}_k$ = impact of the input on the state of the system

The model (6) will never describe reality exactly because of the simplifications in the formulae, inhomogeneity, etc. Therefore the system noise $\vec{w}_k$ is introduced as the difference between the model ($\Phi\vec{\phi}_{k-1} + \Lambda\vec{u}_k$) and reality. The system noise represents an unknown random process. With equation (6) this results in:

$$\vec{\phi}_k = \Phi.\vec{\phi}_{k-1} + \Lambda\vec{u}_k + \vec{w}_k \tag{7}$$

This equation (7) is called the system equation. The noise process $\vec{w}_k$ is assumed to be be an independent multi-dimensional random Gaussian process with zero mean and covariance matrix Q. In addition to the system equation (7) there is the measurement equation:

$$\vec{y}_k = H\vec{\phi}_k + \vec{v}_k \tag{8}$$

where

$\vec{y}_k$ = the vector of which the components are the measurements at $t = k\Delta t$

H = the measurement matrix

$\vec{v}_k$ = the measurement noise, representing the measurement errors

It will be assumed that two components of the state vector ($\vec{\phi}_k$), i.e. two heads, are measured. This means that all elements of H except two are zero. The resulting two elements are equal to one. Furthermore the measurement noise is assumed to be an independent multi-dimensional random Gaussian process with zero mean and covariance matrix R. Moreover the system noise and the measurement noise are assumed to be statistically independent. The matrices Q and R are dealt with as known quantities in the calculations.

3 Estimation methods

In this section two methods are described to estimate, among others, the parameter θ. To estimate a general state vector $\vec{x}_k$ for known parameters the Kalman filter can be formulated with the following set of equations:

$$\vec{x}_k^{\,*} = \Phi\hat{\vec{x}}_{k-1} + \Lambda u_k \qquad (9^a)$$

$$\hat{\vec{x}}_k = \vec{x}_k^{\,*} + K_k\{\vec{y}_k - H\vec{x}_k^{\,*} - \bar{\vec{v}}_k\} \qquad (9^b)$$

$$K_k = P_k^* H^T\{H\, P_k^*\, H^T + R\}^{-1} \qquad (9^c)$$

$$P_k^* = \Phi.\tilde{P}_{k-1}\; \Phi^T + Q \qquad (9^d)$$

$$\tilde{P}_{k-1} = \{I - K_{k-1}\, .\, H\}\; P_{k-1}^* \qquad (9^e)$$

with initial conditions $\hat{\vec{x}}_o$ and $\tilde{P}_o$

where

$$P_k^* = E[(\vec{x}_k - \vec{x}_k^{\,*})(\vec{x}_k - \vec{x}_k^{\,*})^T]$$

and

$$\tilde{P}_k = E[(\vec{x} - \hat{\vec{x}}_k)(\vec{x} - \hat{\vec{x}}_k)^T]$$

The derivation of this set of equations can be found, among others, in Sage & Melsa (1971) and O'Connell (1980). $\vec{x}_k^{\,*}$ denotes the prediction of $\vec{x}_k$ and $\hat{\vec{x}}_k$ denotes the optimal estimate of $\vec{x}_k$. The two methods will be discussed briefly only. A complete description and the derivation of the equations of the parameter estimation algorithms are given by Van Geer and Van der Kloet (1982).

3.1 Single parameter estimation

If the parameter θ is unknown the algorithm (9) to estimate the state of the system cannot be used, because the matrices Φ and Λ contain the unknown parameter. Since θ is a scalar and θ appears only linear in Φ and Λ, $\tilde{\Phi}$ and $\tilde{\Lambda}$ can be defined as a solution of:

$$\Phi = I + \theta\tilde{\Phi} \text{ and } \Lambda = \theta\tilde{\Lambda} \tag{10}$$

Substitution of (10) in (7) yields:

$$\vec{\phi}_k = (I + \theta\tilde{\Phi})\vec{\phi}_{k-1} + \theta\tilde{\Lambda}\vec{u}_k + \vec{w}_k$$

and since θ is a scalar

$$\vec{\phi}_k = (\tilde{\Phi}\vec{\phi}_{k-1} + \tilde{\Lambda}\vec{u}_k)\theta + \vec{\phi}_{k-1} + \vec{w}_k \tag{11}$$

Substitution of (11) in (8) yields, if the measurement error $\vec{v}_k$ is neglected

$$\vec{y}_k = H(\tilde{\Phi}\vec{\phi}_{k-1} + \tilde{\Lambda}\vec{u}_k) + H(\vec{\phi}_{k-1} + \vec{w}_k) \tag{12}$$

Define the state estimation error $\vec{e}_k = \vec{\phi}_k - \vec{\phi}^*_k$ then

$$\vec{y}_k = H(\tilde{\Phi}\vec{\phi}^*_{k-1} + \tilde{\Lambda}\vec{u}_k)\theta + H(\vec{\phi}^*_{k-1} + \vec{e}_{k-1} + \tilde{\Phi}e_{k-1}\theta + \vec{w}_k) \tag{13}$$

Equation (13) can be considered as a measurement equation for the parameter θ, where

$$H^*_k = H(\tilde{\Phi}\vec{\phi}^*_{k-1} + \tilde{\Lambda}\vec{u})$$

is the parameter measurement matrix and

$$\vec{v}^*_k = H(\vec{\phi}^*_{k-1} + \vec{e}_{k-1} + \tilde{\Phi}\vec{e}_{k-1}\theta + \vec{w}_k)$$

is the parameter measurement noise. In order to use the algorithm (9) for estimation of the parameter θ one needs a parameter system equation. Since θ is assumed to be invariable with time this equation is very simple:

$$\theta_k = \theta_{k-1} \qquad (14)$$

Now the other quantities which are required to fill in the equations (9) can be calculated. One of these quantities is the variance p_k of the estimation error of the parameter θ. At each timestep the value of $\vec{\phi}^{*}_{k-1}$ should be known in order to calculate the measurement matrix H^{*}_k for the parameter estimation. The value of $\vec{\phi}^{*}_{k-1}$ can be obtained with the measurements and with (9^a) in which the estimate of the parameter ($\hat{\theta}$) is substituted in Φ and Λ.
The assumption $\vec{v}_k \equiv \vec{0}$ is allowed only if the value of $\vec{v}_k$ is small in comparison with the other quantities. To start the calculations one needs initial conditions for the waterlevel ($\vec{\phi}^{*}_o$), the parameter ($\hat{\theta}_o$), the covariance matrix $\tilde{P}_o$ of the state estimation error and the covariance p_o of the parameter estimation error.

3.2 State/parameter estimation

In the state/parameter estimation two Kalman filters are used at each time step. The first filter is the state estimation. This state estimation differs from the equations (9) by replacing the parameter θ by its optimal estimate ($\hat{\theta}_k$) in the state transition matrix Φ. The influence of the uncertainty in the parameter value is taken into account by the calculation of the covariance matrix $\tilde{P}_k$. The second Kalman filter is the parameter estimation. Again the parameter is considered as a state and the same parameter system equation (10) is used

$$\theta_k = \theta_{k-1} \qquad (14)$$

Now the quantity $\vec{y}^{*}_k = H\,\hat{\vec{x}}_k$ is used as a measurement vector (see also O'Connell (1980)) in the parameter estimation and hence the measurement equation is of the form:

$$\vec{y}_k^{*} = H_k^{*}\,\theta_k + \vec{v}_k^{*} \tag{15}$$

Again

$$\Phi = \tilde{\Phi}^{*}\,\theta_k + I$$

$$\Lambda = \theta_t \tilde{\Lambda}$$

Substituting (9^a) in (9^b) one obtains:

$$\hat{\vec{\phi}}_k = (\tilde{\Phi}\hat{\vec{x}}_{k-1} + \tilde{\Lambda}\vec{u}_k)\ \theta_k + \hat{\vec{\phi}}_{k-1} + K_k\{\vec{y}_k - H\vec{\phi}_k^{*}\}$$

Multiplication of this equation by H yields:

$$\vec{y}_k^{*} = H\hat{\vec{\phi}}_k = H(\tilde{\Phi}\hat{\vec{\phi}}_{k-1} + \tilde{\Lambda}\vec{u}_k)\theta_k + H\{\hat{\vec{\phi}}_{k-1} + K_k(\vec{y}_k - H\vec{\phi}_k^{*})\} \tag{16}$$

which is of the form (15). Now the same quantities as in the single parameter estimation (especially the variance p_k of the parameter estimation error) can be calculated.

There are three important differences between the single parameter estimation method and the state parameter estimation method:

- an optimal state estimate is made with a Kalman filter algorithm in the state parameter method;
- the measurement vector used in the parameter estimation filter in the state parameter method is $H\hat{\vec{x}}_k$, while in the single parameter estimation method the measurements ($\vec{y}_k$) themselves are used;
- in the single parameter estimation method the measurement error is neglected, in the state parameter estimation method this error is taken into account.

4 Calculations and results

4.1 Data generation

The two estimation methods described in section 3 are applied to the flowproblem of section 2. First the data series of the observation wells have been generated by a discrete simulation model with a grid as defined in figure 3.

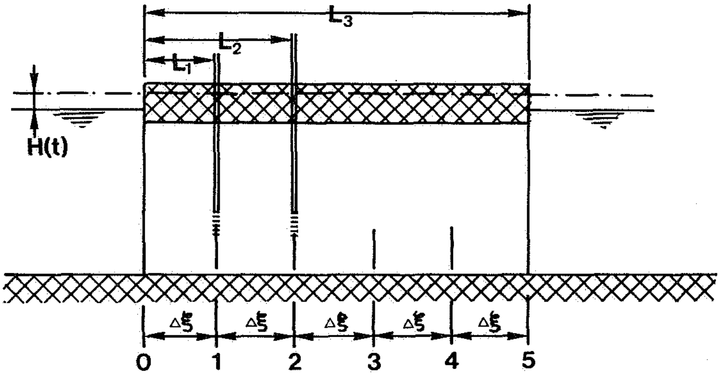

Figure 3 Grid used in the calculations

The data level used, coincides with the average waterlevel in the watercourses. Initially the groundwaterlevel is horizontal. The inputdata of the simulation model are given in table 2.

Table 1 Input data of the simulation model

Parameter	L_1	L_2	L_3	ΔH	σ_w^2
Value	1000 m	2000 m	5000 m	0.6 m	0.0025 m^2
Parameter	σ_v^2	Δt	$\Delta\xi$	T	S
Value	0.0001 m^2	14 days	1000 m	50 m^2/d	0.0002

The heads in the strip of land have been calculated for 100 time steps. After every 13 time steps the water level in the watercourses drops or rises 2.ΔH, as indicated in figure 2. The data which are used in the estimation models are the heads at distances 1000 m and 2000 m from the watercourse at the left side.

4.2 Input of the estimation methods

To perform the calculations of the estimation models one uses the input data of table 1 except the values of T and S. Instead of these parameters an initial estimate $\hat{\theta}_o = 0.25\ m^2/S$ is taken, with variance $p_o = 0.0025\ m^2$. The initial waterlevel is horizontal and if one assumes that all heads are measured at time k = 0 the initial value of the covariance matrix becomes $\tilde{P}_o = R$. The observation wells are located in the first two grid points and consequently the measurement matrix H is known.

4.3 Results

The result of the single parameter estimation method is given in figure 4. This figure shows the estimate of the parameter θ as a function of time. The straight line represents the 'real' value of the parameter. The given 95 % confidence interval is calculated with (17)

$$95\ \%\ \text{confidence interval} = \pm\ 1.96\ \sqrt{p_k} \tag{17}$$

which is permitted if the assumptions of normality are satisfied. Figure 5 gives the same function as in figure 4, but now calculated with the state parameter estimation method. In order to compare the two methods the variance of the parameter estimation error is given in figure 6 for both methods. Finally the variances of the state estimation error are given for the non-measured gridpoints 3 and 4, in figures 7^a and 7^b.

5 Discussion

In principle it appears to be possible to estimate the parameters of groundwaterflow equations with the two methods described. The possibility for practical application depends on the convergence of the confidence interval with time. In other words in how many timesteps does one have an acceptable small confidence interval. This convergence depends for a great deal on the value of the covariance matrix of the system noise and on the accuracy with which this covariance matrix can be estimated in a real situation. The advantages of the parameter estimation with the described methods are obvious when comparing with other methods of parameter estimation:

- in the computerprograms existing measurement series of groundwater levels and heads can be used;
- the computerprograms are much cheaper than field measurements.

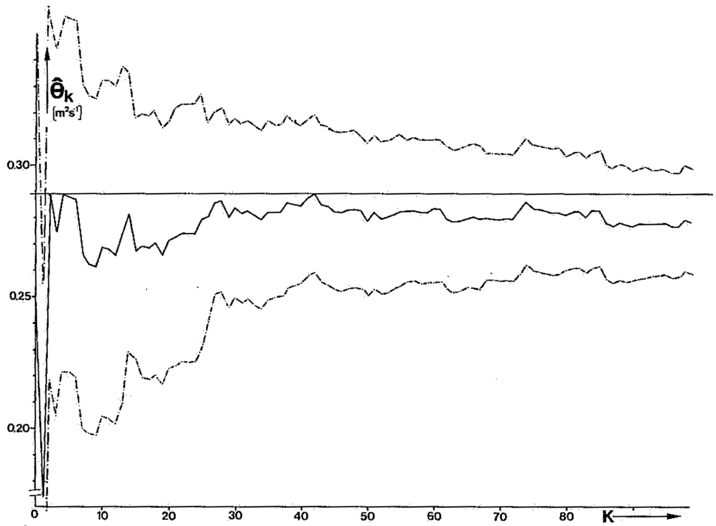

Figure 4 Estimate of θ with single parameter estimation method

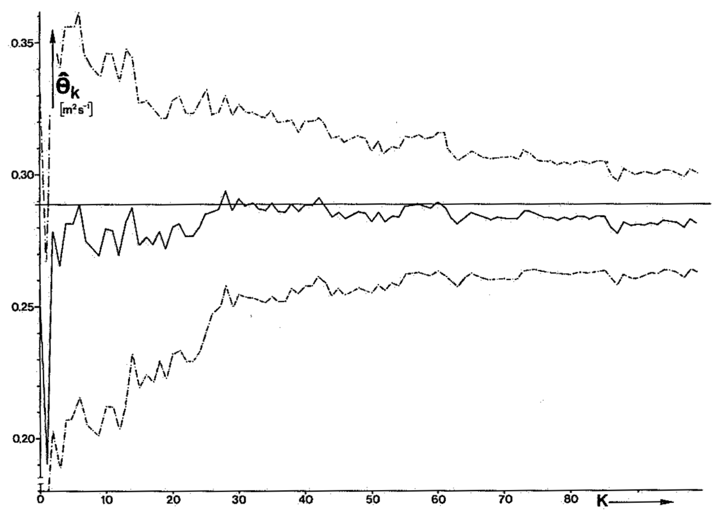

Figure 5 Estimate of θ with state parameter estimation method

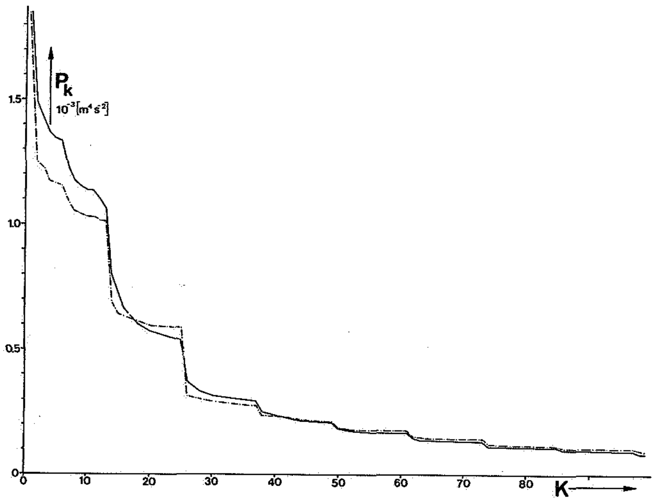

Figure 6 Variance of the parameter estimation errors*)

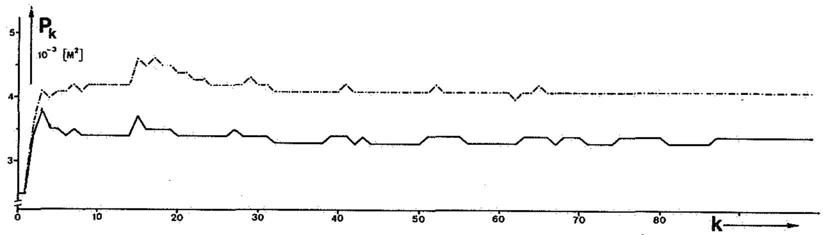

Figure 7[a] State estimation error variances of gridpoint 3*)

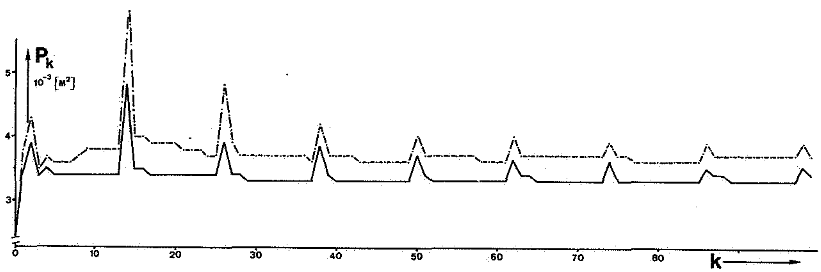

Figure 7[b] State estimation error variance of gridpoint 4*)

*) legend: ———— state parameter estimation

-.-.-.-. single parameter estimation

The single parameter estimation method is simpler (cheaper) than the state parameter estimation method. However it can only be applied when the measurement error is relatively small. Moreover the state estimation error is much larger with the first method than with the second method. This is due to the fact that the second method uses a Kalman filter for a state estimation.

Of course one can use at each timestep, in addition to the sigle parameter estimation method a state estimation with the algorithm (9). After a few iterations a good result will be obtained. If one is interested in an optimal estimate of the state and the parameter simultaneously it is easier to use the state parameter estimation method. The use of the single parameter estimation method is restricted to the estimation of the parameter in advance in case one is not interested in the estimation of the heads.

With the two estimation methods one can get some insight in the variance of the used parameter and hence in the error of the interpolation of the heads. This insight can be useful for the design of a measurement network of heads.

References

Geer, F.C. van 1982. An equation-based theoretical approach to network design for groundwater levels using a Kalman filter. IAHS publ. No. 136, pp. 241-250.

Geer, F.C. van, and P. van der Kloet 1982. Two algorithms for parameter estimation in groundwaterflowmodels. To be published.

Kalman, R.E. 1960. A new approach to linear filtering and prediction problems. J. Basic Engng., May 1960, pp. 35-45

Kloet, P. van der, H. de Niet, and A.G. Posthumus 1982. Determination of a measuring network along rivers. IAHS publ. No. 134, pp. 305-329.

O'Connell, P.E. 1980. Real-time Hydrological Forecasting and Control. Institute of Hydrology, Wallingford, U.K.

Sage, A.P. and J.L. Melsa 1971. Estimation Theory with Applications to Communications and Control. Mc-Graw-Hill.

ESTIMATION OF THE INFLUENCE OF THE CLIMATE FACTORS ON THE MINERAL WATERS IN CARLSBAD BY MULTIVARIATE ANALYSIS

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Abstract

The observational data of the mineral springs in Carlsbad/Karlovy Vary, Czechoslovakia/ and these of the climatic factors are considered as a data matrix characterized by n objects with p characteristics.
The aim of the multivariate analysis is to describe the peculiarities in the behaviour of the objects by plotting them as points in a space. The results of the cluster analysis point to the main trends in the behaviour of the springs when confronted with the climatic factors : Most of the springs show more temperature variation (reflecting so the seasonal changes in the air temperature) than the yield variation which is more pronounced only by one spring. Although the climatic factors do exercise their influence, the general pattern of the displayed cluster-groups manifests the internal stability of the quantitative parameters of the springs.

I Introduction

Presented paper is based on the results of the research work carried out by numerous specialists of the Geological Survey, Prague, Czechoslovakia, of which the author was one. The ensuing studies on the application of the statistical methods to the evaluating of the extensive amount of the observational data in Karlovy Vary was undertaken later on by the author who was enabled to make use of the computer technique abroad.

2 Regional setting

2.1 Location

Karlovy Vary Spa and the surrounding area are situated in the western part of Czechoslovakia and belong to the Bohemian Highland called "The Krušné hory Mountains system" which is an important part of the Bohemian Massif (Svoboda et al., 1964).
The region is montainous ; the maximum altitude of 1,200 m above sea-level is reached at the northern part of Krušné hory. The lowest altitude is reached in the valley of the Ohre river ,(368 m above sea-level) which is the main drain of the region together with its right-hand tributary Teplá river. The occurrence of the thermal springs in this river's bed gave it after all its name - "Teplá" means warm.

2.2 Climate

The position of Karlovy Vary Spa predetermines the character of its climate - the continental and highland type of climate dominates this area. There are long cold winters with the snow cover remaining for 60-80 days. The coldest month is January with an average temperature of -2.2°C. The summer is mostly short and rather cold with the highest temperature in July - this month's average temperature is 16.9°C.
(The data taken from : Climate in ČSSR, 1961).

3 Geological and Hydrogeological features

General geological features of the region are the crystalline complex penetrated by Variscean granites and tectogenetic structure characterized by two main directions of the territory - the NE-SW (Krušné hory Mts direction) and the NW-SE (Sudetes Mts direction). Tectonic movements repeated in the main directions during all important periods created complicated faulted structure with most striking Krušné hory tectonic graben with Tertiary infillings extending NE-SW direction , see Figure 1. The reopening and rejuvenation of the old radial, deep reaching Variscean faults into the Karlovy Vary granite massif during

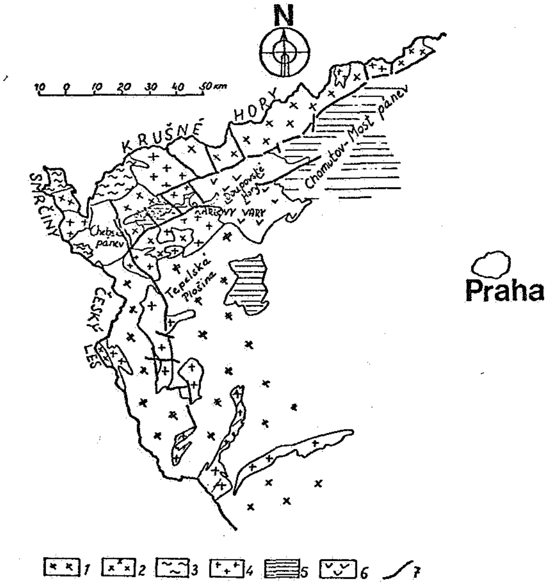

Figure 1 Geological scheme of the Krusně hory Mts system

1 - Precambrian rocks, metamorphosed.
2 - Paleozoic rocks, unmetamorphosed.
3 - Paleozoic rocks, metamorphosed.
4 - Variscean magmatics.
5 - Sediments folded.
6 - Neogene effusive rocks.
7 - Faults, mylonitized zones.

the last Saxonic tectogenetic period and the following volcanic activity were apparently the most favourable conditions for the genesis of the thermal mineral waters in this area.

The ascent of the thermal mineral waters in Karlovy Vary Spa is determined by the crossing of the Karlovy Vary Spring line and the submeridional fault zone and occurs most probably from the deep seated zone of origin up in the common main stream together with the free CO_2 gas near to the surface where finally the waters ramify following the direction of the Karlovy Vary Spring line(Myslil in : Jetel et al., 1964).
The mineral springs differ only in their temperature and their yield, while the chemical composition and the mineralization are almost the same for all the springs.

According to their yield and temperature the springs are divided in two main groups : The "large" springs - with the "Sprudel" as the most famous and attractive one - issue in the centre altogether about 1,500 - 2,000 l/min, their temperature varying between 71-72.5°C. The "small" springs disperse along the river Teplá in the total length of 1,250 m and the altitude variation of 20 m. Their total yield is about 80-100 l/min, the temperature varies widely from 35°C tot 65°C.

4 Selected springs, their characterization and the observational period choice

The more variable group of "small" springs includes nowadays 10 regularly observed springs. Considering their different physical parameters and the various outflow position, 5 of these springs have been chosen as representative : The Castle Spring (highest outflow, highest temperature), the Mill Spring (lowest yield), the Market Spring, the Elisabeth's Spring (lowest temperature) and the Park Spring (border spring). The choice of the tested period was predetermined by the condition of the least possible external interferences with the regimen of the springs and the largest possible continuity of the records for the statistics sake.
The date for the selected period of the years 1922-1930 (month's averages computed from the daily measurements) were entered in the computer in four sequences each comprising three overlapping years, i.e. 1922-1923-1924 ; 1924-1925-1926 and so on.

5 Multivariate analysis

The aim of the multivariate techniques as used here was to attract the attention to certain peculiarities in the behaviour of the objects by representing them as a set of points in 2-dimensional space.
Two alternative approaches that should complement each other were used :
a) the principal components analysis so as to have a representation wherein the major contribution to the global variability - measured as the sum of squared pairwise taken euclidian distances between the objects

was visualized, and

b) a partitioning technique followed by a canonical representation of the groups obtained (De Meuter and Symons, 1975).

That should eventually allow putting in evidence such relevant information that escaped our attention by looking at the plot of the first two principal components.

5.1 Principal component analysis

Given number of springs as n objects with p characteristics the data matrix has been built up in the way that throughout all four sequences each spring was characterized by five variables - the characteristics of the spring and the climatic factors. During all the procedures the indication of the variables is the following :

Variable 1 - yield of the spring
Variable 2 - temperature of the spring
Variable 3 - barometric pressure
Variable 4 - air temperature
Variable 5 - precipitation

To gain the maximum information about the variability of the sets the data matrix will be transformed by rigid rotation so that we obtain a new set of variables that had reduced the multidimensional space to the least possible number of dimensions. The demand on the new set will be that the first new variable explains the largest possible part of the total variability, the second new variable represents the largest possible contribution to the remaining part of it and so on.

The first insight into the structure of our data sets we gain from the total correlation matrix (Table 1).

The total correlation matrix reveals the valuable correlation between the yield and the temperature of the springs and less valuable correlation between the air temperature and the precipitation.

Table 1 Total correlation matrix

Variable	1	2	3	4	5
1	1.00	0.90	-0.01	0.02	0.01
2		1.00	0.01	0.10	0.07
3			1.00	-0.01	-0.08
4				1.00	0.61
5					1.00

For technical reason the tables and the figures are not presented for all four sequences unless the data or the genral pattern differ.

Plotting the object points along the first two principal axes, (Figure 2) , two features are brought to our attention :

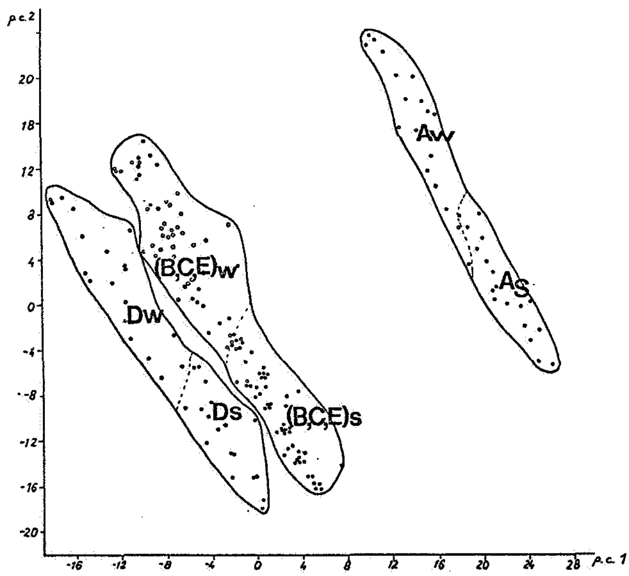

Figure 2 Plot of the springs along first two principal axes
Symbols refer to the cluster-groups characterizing each spring:

A - **Castle Spring.**
B - Mill Spring.
C - Market Spring.
D - Elisabeth"s Spring.
E - Park Spring.

w - winter period o
s - summer period ●
p.c. - principal components.

a) We can distinguish de visu three separate swarms (encircled in the plot). The first one contains all descriptions of the Castle Spring. This is clearly separated from the two remaining ones that lie close together. That one of them that is farthest removed from the Castle Spring-swarm contains all Elisabeth's Spring descriptions, the other one all descriptions from the remaining three springs.
b) In each of the swarms a clear distinction is made between the winter descriptions (upper side) and the summer descriptions (lower side). This plot gives no indication whatever that there should be some systematic difference between those three springs. That such difference exist will only become clear by using the complementary partitioning technique.

5.2 Cluster analysis

The best possible partition of the n objects into a given number of k groups will be defined as the one for which some criterion-function is optimized. Each partition enables us to write the correlation matrix as a sum of two separate matrices : one, noted W, needed to describe the within-groups variability , another one, noted B, needed to describe the between-groups variability.
We did choose the ln of the ratio $|W + B| / |W|$ as the criterion-function to be maximized.
The search for the best partition was done using the strategy developed by Rubin and Friedman (1967).

We propose a comparison between the a posteriori partition into 20 cluster-groups with the a priori partition into 5 springs. The number of cluster-groups, far exceeding the number of springs will result in more homogeneous groups.
The summary of the final best partitions as shown in Table 2 gives us an imagination about the structure of the data sets :
The clustering of the objects into 20 groups is throughout all four sequences quite regular ; most of the objects are spread into 4-5 separate groups, only the objects characterizing the Market and the Park Springs are always found in a common group.

Table 2 Final set memberships for best partition

Sequence	Springs group	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
1922–24	Castle	11	8	5	5	4	3														
	Mill							14	9	9	4										
	Elisabeth's											10	10	8	8						
	Market															15	9	7	4	1	
	Park															5	1	5		9	16
1924–26	Castle	10	8	7	6	5															
	Mill						13	13	10												
	Elisabeth's									14	11	8	3								
	Market													14	11	8	3				
	Park														1			12	11	8	4
1926–28	Castle	10	10	6	5	5															
	Mill						18	11	7												
	Elisabeth's									16	11	6	3								
	Market													10	9	6	6	5			
	Park																11		10	10	5
1928–30	Castle	9	9	6	6	4	3														
	Mill							20	11	5											
	Elisabeth's										12	11	8	5							
	Market														11	9	10	5			
	Park															3		13	10	6	4

For visualizing the relations between and within the groups we can plot them along the canonical axes. Projected on the plane generated by the first two canonical variates the general pattern of the displayed cluster-groups (Figure 3) reveals some relevant information about the variability between and within the groups.
For the separation of the clusters along the horizontal axis is the first canonical variable, i.e. the yield of the springs, responsible. The separation along the vertical axis is due to the second canonical variable - the temperature of the springs.
Most of the clusters form separated ellipsoids elongated in the vertical direction, only the cluster representing the Castle Spring is elongated in wry direction and far distant to the right.

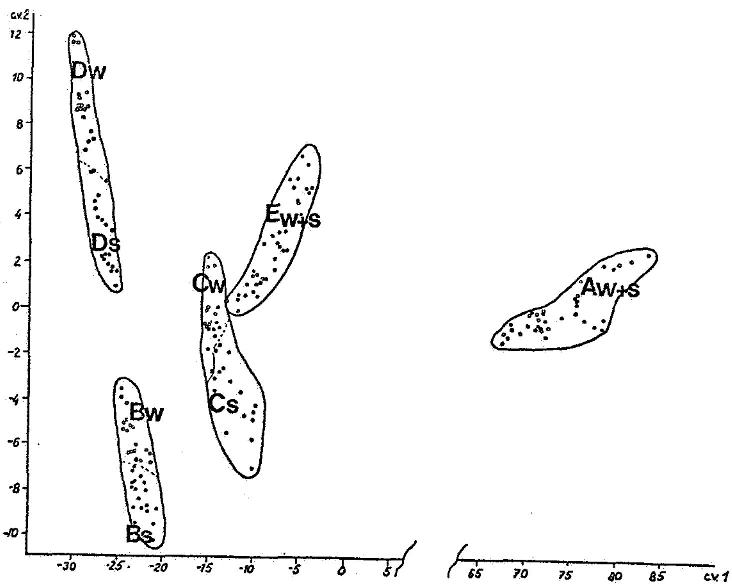

Figure 3 Plot of the springs alongs first two canonical axes
See Figure 2 for meaning of symbols;
c.v. - canonical variate.

The variability of the springs when confronted with the climatic factors can be likewise observed studying the values of their standardized means (Table 3). Throughout all four sequences the yield variations are strongly pronounced by the Castle and the Park Springs, less by the Market Spring and inappreciable by the Mill and the Elisabeth's Springs. Inversely the temperature variations are unnoticeable by the Castle Spring and well pronounced by all the remaining springs.

6 Conclusion

Considering the deductions of the multivariate analysis and particularly the cluster analysis the influence of the climatic factors on the regimen of the examined mineral springs is to be estimated as acting differently on the individual springs.
Most of the springs produce higher temperature variability compared to the variability of their yield indicating so the direct response to the seasonal air temperature variations.
The effect of the precipitation with the reverse related barometric pressure is more difficult to trace and can be explained rather hydraulically (Myslil, Opavská in : Jetel et al., 1964).

The precipitation and the barometric pressure contribute to the oscillation of the surrounding ground water level that in its turn affects the outflow of the spring. The exceptional position of the Castle Spring during the analyses with its high yield variation is to be referred to the reciprocal interaction of the above mentioned climatic factors which is enhanced by the highest issue altitude of the all observed springs.

The withing group variability can be evaluated considering the differences between the utmost points of each cluster. For the clusters of the Mill, the Market, the Elisabeth's and partly the Park Springs the variability along the second canonical axis dominates, while for the cluster of the Castle Spring the variability along the first canonical axis is more pronounced.
Within the clusters of the Mill, the Market and the Elisabeth's Springs

Table 3 Standardized means z in each group.

Spring	Castle		Mill		Market		Elisabeth's		Park	
variable Sequence	1	2	1	2	1	2	1	2	1	2
1922-1924	2.2	1.9	-0.8	-0.4	-0.4	-0.4	-0.7	-1.5	-0.4	-0.4
	1.9	1.9	-0.8	-0.2	-0.4	-0.2	-0.7	-1.0	-0.2	-0.4
	1.8	1.9	-0.8	0.1	-0.2	-0.4	-0.7	-1.5	-0.4	-0.2
	2.1	1.9	-0.8	0.1	-0.3	0.1	-0.7	-0.9	-0.2	-0.4
	2.0	1.9			-0.4	-0.4			-0.4	-0.4
	2.3	1.9								
1924-1926	2.3	2.1	-0.9	0.1	-0.5	-0.4	-0.8	-1.0	-0.3	-0.4
	2.5	2.2	-0.9	-0.1	-0.5	-0.3	-0.8	-1.3	-0.5	-0.4
	2.2	2.1	-0.9	-0.3	-0.5	0.1	-0.8	-2.0	-0.2	-0.5
	2.2	2.1			-0.5	-0.7	-0.8	-1.7	-0.1	-0.7
	2.1	2.0							-0.2	-0.5
1926-1928	2.4	2.3	-1.0	-0.4	-0.5	-0.4	-0.9	-2.1	-0.5	-0.4
	2.3	2.2	-1.0	-0.6	-0.6	-0.3	-0.9	-1.4	-0.4	-0.4
	2.6	2.3	-1.0	-0.1	-0.5	0.0	-0.9	-1.0	-1.2	-0.6
	2.2	2.2			-0.6	-0.4	-0.9	-1.7	-0.3	-0.5
	2.7	2.3			-0.4	-0.1				
1928-1930	2.6	2.1	-0.9	-0.1	-0.4	-0.4	-0.9	-1.7	-0.4	-0.4
	2.6	2.1	-0.9	-0.5	-0.5	-0.4	-0.9	-1.1	-0.5	-0.4
	2.3	1.9	-0.8	0.5	-0.4	0.2	-0.9	-1.5	-0.3	-0.4
	2.0	2.0			-0.5	-0.4	-0.9	-2.0	0.2	-0.9
	2.4	2.0							-0.2	-0.1
	2.1	2.0								

$z = \frac{x - \bar{x}}{\sigma}$ where x = computed mean value, $\bar{x}$ = its mean, σ = standard deviation the division concerning the objects representing respectively the summer and the winter period can be observed.

The general unchangeable pattern of the displayed clusters throughout the observed period gives however evidence of the internal strength and the steadiness of the tested group of springs in view of the climatic factors. This can be regarded as an important aspect supporting the theory of the ascent of the thermal waters in the common huge stream that ramifies only near to the surface making the springs vulnerable only in the very last phase of the ascent.

Practical application of the results of these analyses can give guidance for the maintenance of the springs in the choice of the suitable time for recaptures, repaires or other interferences.

Acknowledgements

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References

de Meuter, F.J.C. and Symons, F.E. 1975. A multivariate analysis of the morphological variability in the foraminiferal species Florilus boueanus (d'Orbigny, 1846) from the "Neogene" of northern Belgium Micropaleontology, vol. 21, No 4, pp. 460-471.

Jetel, J., Myslil, V., Opavská, I. and Pačes, T. 1964. Hydrogeologie sokolovské panve a širšího okolí Karlovych Varů (Hydrogeology of the Sokolov basin and Karlovy Vary environs) Final report, Geofond, Prague.

Kolektiv, 1961. Podnebí ČSSR, Tabulky. The Climat in CSSR, HMU, Prague.

Rubin, J., and Friedman, H. 1967. Cluster analysis and taxonomy system for grouping and classifying data. IBM Corporation, pp. 1-221.

Svoboda, J. a kol. 1964. Regionální geologie ČSSR, I/1, 2. Praha. p. 512 - 532.

THEME 4:

INSTRUMENTATION FOR THE ACQUISITION
AND PROCESSING OF GEOHYDROLOGICAL DATA,
INCLUDING ACCURACY, COST AND MANAGEMENT ASPECTS

MATERIEL PERMETTANT D'ACQUERIR
ET DE TRAITER DES DONNEES GEOHYDROLOGIQUES -
PRECISION, COUT ET GESTION

THEME 4:

INSTRUMENTATION FOR THE ACQUISITION
AND PROCESSING OF GEOHYDROLOGICAL DATA,
INCLUDING ACCURACY, COST AND MANAGEMENT ASPECTS

MATERIEL PERMETTANT D'ACQUERIR
ET DE TRAITER DES DONNEES GEOHYDROLOGIQUES -
PRECISION, COUT ET GESTION

USE OF A NEW DEVICE FOR LOW AND CONSTANT DISCHARGE IN PUMPING TESTS ON DUG WELLS

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Abstract

In India, dug wells are a primary source of groundwater for irrigation and domestic use. Determination of parameters of aquifers tapped by dug wells is thus important for optimal exploitation of the groundwater resources.
Hydrological tests, performed by using the pumps installed by farmers on dug wells, show a sharp decline in the discharge with time. Also the wells are emptied in a few hours because of high capacity of pumps. Satisfactory hydrological tests on dug wells, lasting for several hours and indicating significant aquifer response, are possible only if the discharge is kept low and constant. This objective has been achieved through a new device which involves return of a variable fraction of the output to the well. Details of the device are described. Some examples of use of this device in estimation of parameters of aquifers tapped by dug wells in the Deccan Trap basalt lava formation are described. The analytical solution by Papadopulos and Cooper (1967) was used for interpretation of the time-drawdown data.

1 Introduction

The simple and cheap method of exploiting groundwater practiced in India and other Souht Asian countries since ancient times, involves digging of large diameter shallow wells tapping mostly the near surface aquifers. Even at present, major exploitation of groundwater resources in India is through dug wells. At the beginning of the year 1978, about 79% of the total existing wells in India were dug wells (Baweja, 1979), 18% were shallow bore-tubewells and rest 3% were deep tube wells. Lahiri (1975) has estimated that about 71% of the groundwater draft in India was from dug wells.

Approximately two third (2.4 million sq.km) of the total area of India is covered by hard rocks such as granites, gneisses, schists, basalts and indurated precambrian sediments. The farming community in these areas is heavily dependant on dug wells as a supplementary source for irrigation and domestic use. A better understanding of aquifer characteristics, as reflected in dug wells, is therefore, important for an optimum development of groundwater resources.

A dug well section in hard rock comprises a top layer of weathered mantle or regolith which gradually changes with depth into weathered rock and then to fresh rock which may be jointed and fractured. Some of the dug wells are entirely in the rocks, with a thin soil cover, while some, especially in low lying or valley-fill areas are entirely within overburden or alluvium.

2 Aquifer tests using dug wells

Aquifer tests in dug wells have been carried out by various workers (Raju and Raghav Rao, 1969; Adyalkar and Mai, 1964; Deolankar, 1981). The dug wells itself served as an observation well in most of the cases. Some of the tests were made under an assumption that the discharge from the pumps powered by electrical motors remained constant throughout the test duration. However, in dug wells there is a gradual decline in the discharge because the head of the water, stored above the foot valve of the pump, declines with pumping time. We have studied the variation in discharge with time in the case of several dug wells in basalt flows, and two examples of such a decline are shown in Figure 1.

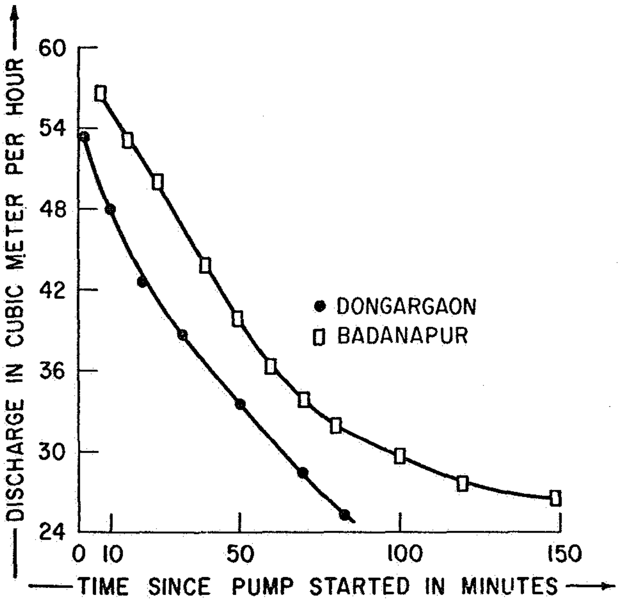

Figure 1 Discharge variation with time in the case of two dug wells in basalts

An additional problem is that farmers generally use high capacity pumps and the wells are emptied within two to three hours of pumping. The rate of discharge through these pumps is very high compared to the rate of inflow to the well (Sammel, 1974). This is particularly true in the case of the dug wells in hard rocks having poor transmissivity.

A satisfactory pump test in dug wells has to last at least for a few hours, in order to get the aquifer response. The discharge is thus required to be kept low as well as constant. One method of achieving this is to introduce a control valve on the output pipe of the existing pumping system. However this is not desirable as it may cause damage to the pumping device, because of the back pressure which develops when the discharge is required to be kept quite low and for several hours. We have therefore fabricated a simple, cheap and portable device, which can be easily fitted on the output line of the existing pump and used for keeping the discharge constant and at a desired (low) value throughout a dug well pump test.

3 Device for low and constant discharge from dug wells

A diagrammatic sketch of the device installed on a dug well is shown in Figure 2. The uncontrolled discharge from the well pump is diverted for transit storage in a barrel of 200 litre capacity. The water is released in the barrel through a pipe (6.5 cm diameter) reaching almost upto its bottom. Two outlets are fixed on the barrel. One of them is in the nature of a V-notch having graduations. This outlet is fixed just below the top rim of the barrel so that the water leaves the drum as an overflow along this restricted channel. The other outlet is a pipe (5 cm diameter) fixed at the bottom of the barrel and is used for returning a fraction of the discharge to the well. This pipe is connected to a flexible hose pipe through a control valve. The hose pipe is inserted in the well below the water level till it almost touches the well bottom, so that the return flow does not give rise to any turbulence. The amount of return flow can be varied through manipulations of control valve. Since the return flow to the well is always less than the amount taken out by pump, the possibility of recharge to the well due to the return flow does not arise.

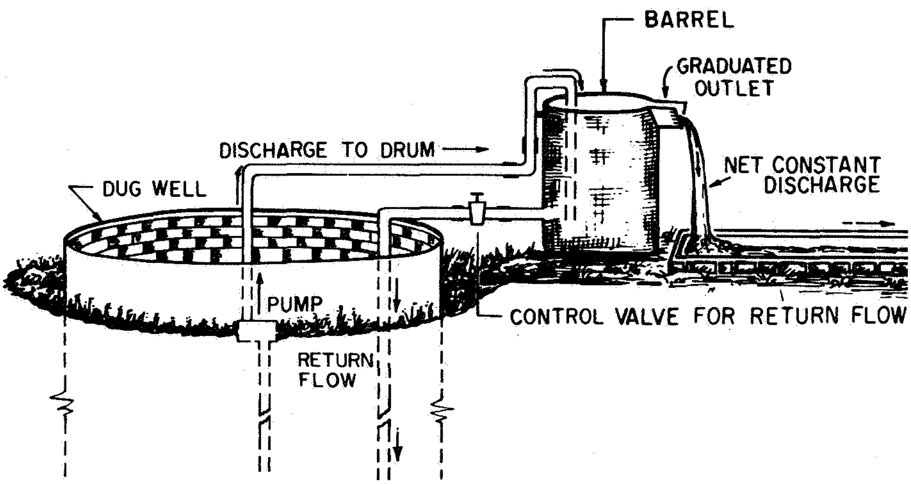

Figure 2 Diagrammatic sketch showing the constant dicharge devices in use

Under this arrangement, the overflow from the V-notch becomes the net discharge from the well during the hydrological test. The amount of overflow is preset and an appropriate mark is made on the V-notch of the inside wall of the barrel. It is ensured that the water level of the overflow is kept at this mark throughout the pumping test. In actual practice, since the initial uncontrolled discharge is rather high, the control valve on the return flow line is kept nearly open at the start of the pumping test. With progress of pumping test the actual discharge undergoes gradual decline and the control valve on the return flow line is gradually closed in tandem with the decline, so that the quantum of return flow to the well also gets correspondingly reduced. The net discharge from the well as measured by the quantum leaving the barrel along the V-notch is thus kept constant throughout the pumping test. As a seperate check on the constancy of the net discharge and also to determine its absolute value, the time required to fill a 100 litre drum, placed below the overflow, is measured at frequent intervals during the pumping test.

4 Analysis of pump test data

The time-drawdown plots of large diameter wells are made of two initial segments. The first straight line portion is a result of the effects of well storage and does not accurately represent the aquifer characteristics. However, the subsequent convex (upward) segment can be used for determining the parameters of aquifers by using the Papadopulos and Cooper (1967) method. In the case of unconfined aquifers, as Prickett (1965) has pointed out, the response will be similar to that of a confined aquifer during the early stage of pumping. Narasimhan (1968) has been applied the Papadopulos and Cooper (1967) method in the case of large diameter wells in water-table aquifers. We have therefore used the Papadopulos and Cooper method to analyse the early stage response of dug wells in water table aquifer.

5 Examples

We have carried out pump tests on dug wells in Deccan Trap basalt flows at 29 places, in the Godavari-Purna basin of Aurangabad district using the constant discharge device. The duration of pumping test varied from 2 to 8 hours. The constant discharge rate for each test was selected on the basis of a discussion with the farmer about the well behaviour. The dug well itself was used for monitoring water level. Time-drawdown curves were plotted on a log-log paper and four such tests are shown in Figure 3. All the time-drawdown curves were matched with the master curves prepared by Papadopulos and Cooper (1967) and the transmissivity and the storage coefficient values were calculated (Table 1) using the equations given by these authors, who state that although determination of storage coefficient is possible by their method, the values have questionable reliability. As has been pointed out by Papadopulos and Cooper (1967) themselves, the determination of the transmissivity value is not very sensitive to the choice of the master

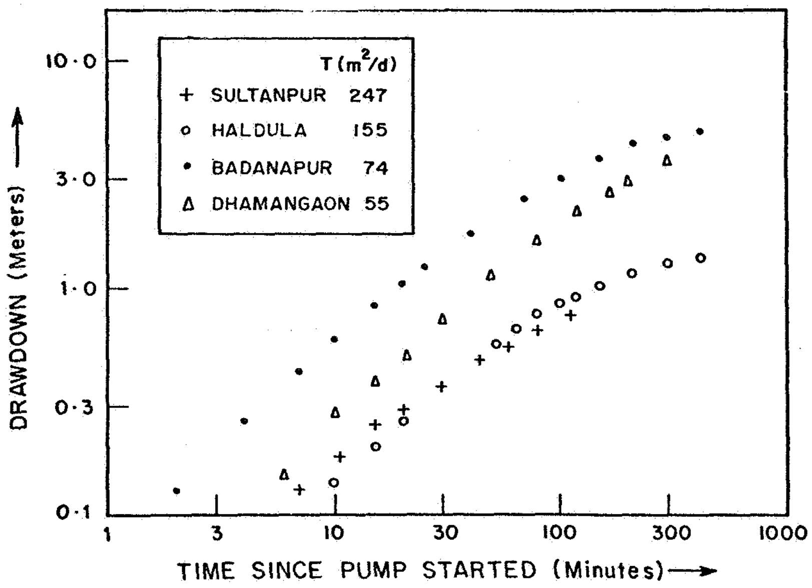

Figure 3 Time-drawdown plots of pumping tests data for dug wells in basalt flows using constant discharge device

Table 1. Transmissivity of aquifers tapped by dug wells in basalt flows

Location	Transmissivity m^2/day	Storage coefficient	Lithology of Well Section
Gevrai	552	0.21	Regolith underlain by jointed and fractured basalt
Dongargaon	434	0.19	Alluvium underlain by brecciated basalt
Boigaon	344	0.05	Alluvium underlain by vesicular basalt
Sultanpur	247	0.13	Alluvium
Chauka	155	0.027	Regolith underlain by jointed and fractured basalt
Akola	215	0.01	Alluvium
Haldula	155	0.010	Regolith underlain by jointed and fractured basalt
Dhopteshwar	151	0.01	Alluvium
Phulambri	123	0.02	Alluvium
Pimpalkhuta	116	0.009	Alluvium underlain by brecciated basalt
Ganori	91	0.012	Weathered vesicular basalt
Devlana	76	0.01	Weathered vesicular basalt
Phulambri	75	0.01	Jointed and fractured basalt
Badanapur	74	0.008	Jointed and fractured basalt
Dongargaon	69	0.001	Weathered vesicular basalt
Relgaon	61	0.013	Weathered vesicular basalt
Dhamangaon	55	0.008	Weathered vesicular basalt
Sarai	53	0.012	Weathered vesicular basalt
Valha	48	0.008	Vesicular basalt
Tajnapur	45	0.01	Massive basalt
Ganori	39	0.003	Vesicular basalt
Bhandegaon	37	0.018	Vesicular basalt
Kingaon	36	0.015	Massive basalt
Jategaon	32	0.023	Vesicular basalt
Khadgaon	23	0.001	Massive basalt
Shelgaon	23	0.03	Massive basalt
Khuldabad	20	0.07	Vesicular basalt
Gokulwadi	18	0.08	Massive basalt
Shirudi Kd.	15	0.014	Massive basalt

curve; however, the value of storage coefficient changes by an order of magnitude when the data plot is moved from one to another adjacent master curve.

In view of the uncertainty in the storage coefficient values thus determined, we have tried to obtain an independant check on their reliability. In the case of 11 wells out of the 29 sites listed on Table 1, water level fluctuation data was also available for the year 1980-1981. The difference in pre and post monsoon water levels was used along with storage coefficient values for calculating the recharge to the phreatic aquifer. These values were compared with those obtained by Athavale et al. (1982) through the Tritium injection technique. The two sets of values (Table 2) show a reasonable agreement in seven out of eleven cases.

This study thus indicates that approximately reliable values of the storage coefficient can be obtained by using the Papadopulos and Cooper (1967) method.

Table 2 Comparison between recharge values by hydrogeological and Tritium injection methods

Location	Waterlevel change h in cm	Specific yield (Sy)	Recharge Hydrogeological: h x Sy (cm)	Recharge: H^3 method (cm) (post monsoon values)
Gadana	140	0.05	7.0	6.2
Takli	215	0.05	10.8	9.7
Ganori	270	0.012	3.2	12.0
Varegaon	250	0.015	3.8	7.6
Phulambri	260	0.02	5.2	7.7
Pimpalgaon	105	0.05	5.2	2.5
Lambkana	125	0.009	1.1	7.2
Pimpalkhuta	130	0.009	1.2	2.6
Wala	140	0.008	1.1	1.1
Selgaon	125	0.030	3.7	3.1
Dhopteswar	165	0.01	1.7	3.2

6 Conclusion

A simple and cheap device has been developed for keeping the discharge low and constant in the case of aquifer tests performed on dug wells. The device has been used in determining parameters of phreatic aquifers in the Deccan Trap basalt lava flows. Results obtained through pumping tests on 29 dug wells show a variation in transmissivity values from 552 m^2/d to 15 m^2/d. The storage coefficient values vary between 0.21 to 0.001.

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References

Adyalkar, P.G. and V.V.S. Mani, 1974. Application of groundwater hydraulics to a basaltic water table aquifer. Jr. of Hydrology, 21: pp. 211-216.

Athavale, R.N., R. Chand and R. Rangarajan, 1982. Groundwater recharge estimates for two basins in the Deccan Trap basalts formations (communicated for publication).

Baweja, B.K., 1979. Hydrogeological set up of India and status of work of the Central Ground Water Board, Lecture delivered at UNESCO regional training course at N.G.R.I., Hyderabad.

Deolankar, S.B., 1981. The Deccan basalts of Maharashtra, India. Their potential as aquifers. Ground Water. 18(5): pp. 434-437.

Lahiri, A., 1975. A long range perspective for India: Water Resources - 2000 A.D., Publication, Operation and Research Group, Baroda, India, p. 43.

Narasimhan. T.N., 1968. Pumping test on open wells in Palar alluvium near Madras city, India: An application of the Papadopulos-Cooper method. Bull. International Association Sci. Hydrology. XIII(4): pp. 91-105.

Papadopulos, I.S. and H.H. Cooper, Jr. 1967. Drawdown in a well of large diameter. Water Resources Research, 3(1): pp. 241-244.

Prickett, T.A., 1965. Type curve solution to aquifer test under water-table conditions, Ground Water 2(3): pp. 5-14.

Raju, T.S. and K.V. Raghava Rao, 1967. New approach to the analysis of pump test data on an open well. Paper presented at Symp. on Ground and Lake Water Resources in India, Hyderabad.

Sammel, Ed. A., 1974. Aquifer tests in large diameter wells in India, Ground Water, 12(5): pp. 265-272.

DETERMINATION OF WATER LEVELS IN OBSERVATION WELLS BY MEASUREMENT OF TRANSIT-TIME OF ULTRASONIC PULSES AND CALCULATION OF HYDRAULIC PARAMETERS

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Abstract

The drawdown of the groundwater level in a pumping test can be determined by measuring the transit-time of ultrasonic pulses in the water. An ultrasonic transducer is placed several meters below the water table in each observation well. The transmitted pulse is reflected by the water surface and detected at the transducer. The design of the measuring equipment; number and spacing of the observation wells; accuracy of the measurements of the water table level; acquisition of other data such as pumping rate, barometric pressure, temperature, and conductivity of the water; and data aquisition and processing are discussed in detail. The results of a pumping test in a very permeable aquifer of coarse gravel in the upper part of the Iller Valley in the Alp foreland (near Ravensburg in Baden-Württemberg) clearly show that the accuracy of the data obtained by this method is decisive for the evaluation of the test. Because of the complicated geometry of aquifers, it is necessary to obtain the data from wells that are spaced as closely as possible in order to be able to delineate the hydraulically effective boundaries. While the influence of the boundaries of a simple, rectangular cross-section can be described analytically by relatively few measurements, the geohydraulic boundaries of the naturally formed cross-section of a valley can be determined only in the course of relatively long study.

Enregistrement des données lors d'essais de pompage à l'aide de la méthode de la "mesure du niveau d'eau par la mesure du temps de parcours des impulsions ultrasoniques". - Calcul des paramètres hydrauliques des aquifères.

Résumé

Les rabattements du niveau d'eau lors des essais de pompage sont déterminés à l'aide de la mesure du parcours d'une impulsion ultrasonique. Dans chacun des piézomètres un vibrateur ultrasonique est pendu quelques mètres au-dessous du niveau de la nappe. L'impulsion émise est réflectée au niveau d'eau et reçue par le vibrateur. Les détails techniques suivants sont traités comme suit: construction de l'appareillage de mesure, nombre et distance des piézomètres qui sont observés simultanément, exactitude de la mesure du niveau d'eau, enregistrement d'autres données (p.ex. débit pompé, précipitation, pression atmosphérique, température, conductivité électrique de l'eau extraite, enregistrement et traitement des données).
Le résultat des travaux dépend de façon décisive de l'exactitude des données obtenues, comme le montre l'exemple d'un essai de pompage dans l'aquifère très perméable d'un gravier grossier situé dans le haut de la vallée de "Iller" (avant-pays alpin dans le district de Ravensburg, Baden-Württemberg). A cause d'une géométrie compliquée une séquence rapide des données est indispensable. Alors que les influences du bord de sections rectangulaires simples se laissent décrire analytiquement avec relativement peu des données, les bords des sections naturellement arrondies des vallées ne se présentent géohydrauliquement qu'au cours d'une longue phase de transition.

1 Introduction

The measurement of the water level in observation wells manually during pumping tests usually provides too little and too inaccurate data and the personnel costs are very high.

The use of the usual automatic instruments for recording the height of the groundwater table makes compromises necessary, since no single instrument meets all of the requirements for optimal measurement of the water level. For example: better accuracy than a water level indicator immediate reaction to rapid changes in the water level, high resolution of time, exact recording of the times of measurement, and low cost for each observation site. In addition, the data is normally presented in the form of long lists or analog recordings. A detailed evaluation requires a large amount of time and the recordings cannot be fed directly into a computer for processing. For these reasons, we have constructed an apparatus for collecting pumping test data that has the following characteristics:

- Significant improvement in accuracy relative to conventional, especially manual methods for measuring the groundwater table in observation wells and reduction of the cost of the equipment per well.

- Measurement and recording of other parameters during the pumping test, e.g. pumping rate, precipitation and air pressure, temperature and conductivity of the groundwater, and water level in receiving streams and other surface waters in the area.

- Automation of the water level and other pumping test parameter measurements; closely spaced measurements at the beginning of the test; recording of the data on a computer-compatible storage medium.

2 Description of the BGR System

The apparatus described here (Fig. 1) is a further development of the equipment described by Dürbaum & Kohlmeier (1970).

2.1 Measurement of the Groundwater Level

An ultrasonic transducer is placed in each observation well at a specific depth measured relative to a fixed point. This transducer, which is to function as both transmitter and receiver, is to be set up so that it will not change position during the entire test. The time an ultrasonic pulse takes to travel from the transducer to the water surface and back is measured with a device for measuring very short time intervals. This transit time, measured with a resolution of 0.1 µs, on multiplication by the velocity of sound in groundwater yields the distance from the transducer to the water surface times two.

Thus, the resolution of the distance measurement is about 0.14 mm. Considering a safety factor of 6, a precision of better than 1 mm is obtained for changes in the water level. Thus the accuracy of the water level measurement is dependent on the accuracy with which the transducer was positioned relative to the fixed point.

A scanner is used to read the data from up to 30 transducers one after the other. With a maximum length of each measurement of about 50 ms, readings can be made for all of the observation wells in about 1.5 s, for all practical purposes, simultaneously.

The measuring unit consists of a pulse transmitter, a pulse receiver (including an amplifier), and an electronic counter for measuring short time intervals (Fig. 1). Each of the transducers is connected to the central measuring and control unit installed in an intrumentation vehicle nearby. The cable lengths should not exceed 600 m.

Depending on the amount of drawdown expected, the transducers are placed 1 - 3 m below the water level. If the drawdown during the test is greater than this, the transducer can be lowered a known amount. This change in the depth of the transducer is then taken into account in the evaluation of the data. For further details, see Dürbaum & Kohlmeier (1970).

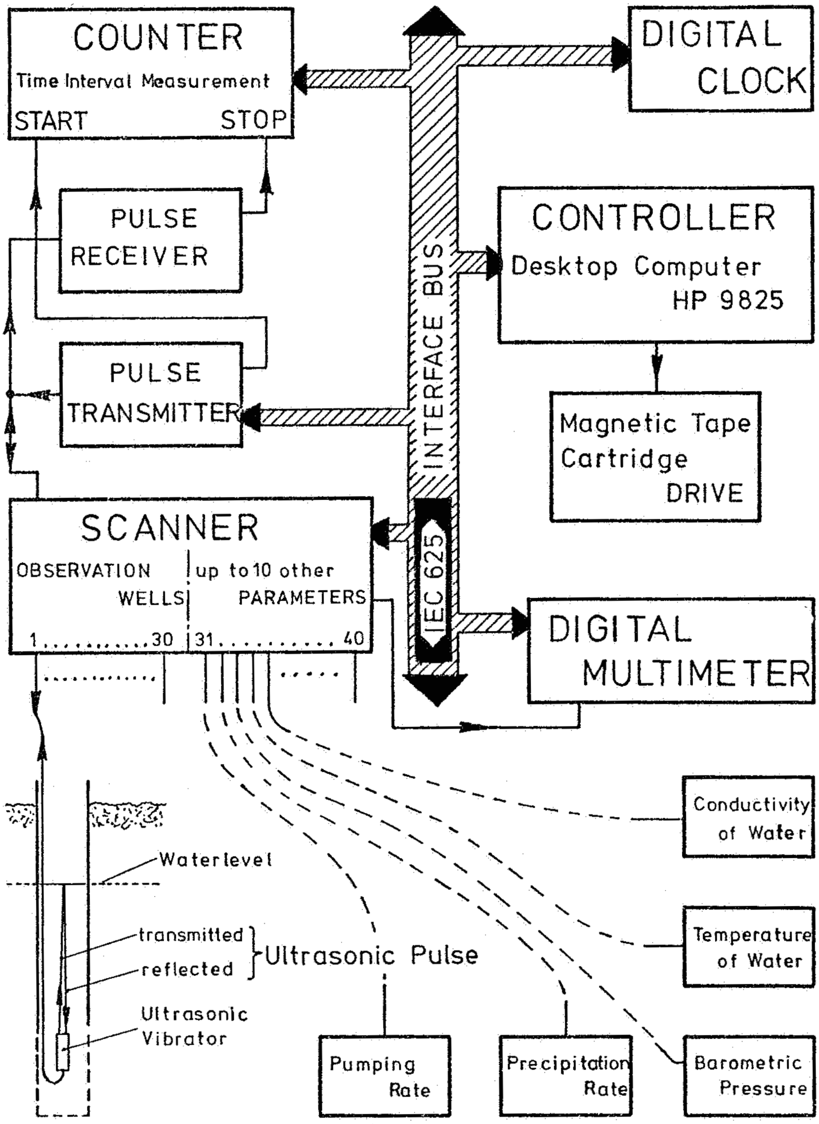

Figure 1 Device for data aquisition during pumping tests; measurement of water level by measurement of the transit time of ultrasonic pulses

2.2 Measurement of Other Pumping Test Parameters

The pumping rate(s) can be measured with a magnetic flowmeter (100 or 200 mm nominal width) and recorded as an electric signal.
When measurements are being made in an existing installation (e.g. pumping tests in an existing water works), instruments belonging to the water works can be connected to the central control unit:
A precipitation gauge (0.1 mm resolution) and a digital barometer, both with an electrical output, can be used to measure and record precipitation and air pressure in the area. A piezoelectric pressure transducer with electrical output can be used to measure and record the water level in a receiving stream. A conductivity transducer and a temperature transducer mounted directly in the discharge pipe can be used to measure and record the conductivity and temperature of the pumped groundwater. The scanner in the central control unit is used to read each of the output signals one after the other. The signals are measured by a digital multimeter and then recorded.

2.3 Control and Data Acquisition Unit

A desk-top computer (HP 9825) is used as control unit for all of the measurements. A standard interface, the "IEC-625-Bus" (IEC-Bus for short) (IEC Publication 625-1 1979), is used to connect all of the individual units.
Starting with time t_s, where t_s is the start of the pumping test or recovery time, the computer controls the start of each measuring cycle. The time is provided by a digital clock. The scanner is activated via the IEC-Bus interface and each transducer is connected with the corresponding measuring instrument and the resulting reading is stored. The start of each measuring cycle during the first hour of the pumping test or of the recovery time is programmed so that when the data is plotted semi-logarithmically, the points will be equidistant on the time axis (Table 1).
After the first hour or during measurements of the water level when the well is not being pumped, the time interval between the individual measuring cycles is provided by the operator. Generally, quarter or half

hour intervals are selected. All of the readings (water level and other parameters) made during a cycle are stored, beginning with date and time, together with other information as a data file on magnetic tape (cartridge). A cartridge can hold 600 data files. This corresponds to the data for about 6 days. The data are then transferred from the cartridges to magnetic tape compatible for use with a large computer, which can then be used to process the data and plot the results (drawdown curves) on a plotter.

Table 1. Starting time for each cycle during the first hour of the test

cycle	t_s + h	t_s + s	cycle	t_s + h	t_s + s
1	0.000	0	11	0.070	252
2	0.002	7	12	0.100	360
3	0.004	14	13	0.140	510
4	0.006	21	14	0.200	720
5	0.008	28	15	0.300	1080
6	0.010	36	16	0.400	1440
7	0.014	51	17	0.500	1800
8	0.020	72	18	0.660	2400
9	0.030	108	19	0.830	3000
10	0.050	180	20	1.000	3600

3. Example of a Geohydraulic Evaluation

3.1 Test Site and Implementation

The test site was in an area strongly influenced by a channel structure carved into the impermeable sandy marl of the Upper Fresh-water Molasse of the Miocene (Figs. 2a & 2b). The channel fill consists of coarse glaciofluvial gravel and can be divided into two parts on the basis of its sand content.
The lower part of the channel fill, except for petrographic differences,

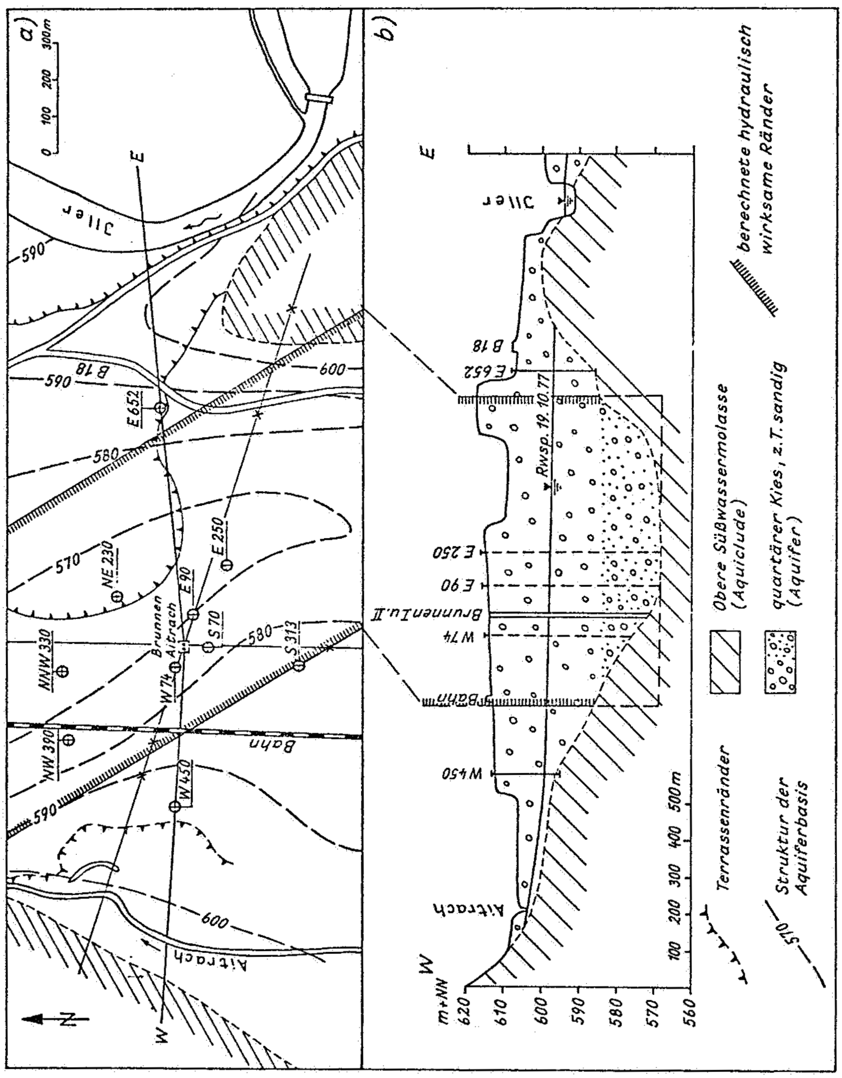

Figure 2 Hydrogeological situation

a) map b) cross section

has a significantly higher sand content than the upper part and is therefore less permeable. The channel fill is two-thirds full of groundwater. In the center of the channel the aquifer reaches thicknesses of 30 m maximum.
The pumping test was done in the two fully penetrating wells used for supplying water to Aitrach near Ravensburg. These wells are close to each other on the west side of the channel structure (Fig. 2b). The wells were pumped at a constant rate of $Q = 0.05\ m^3/s$ for 170 h. This water was fed into the supply line via a water tank placed high on a hill. The rate of drawdown was followed in 12 observation wells, all of which reached to the bottom of the aquifer (Fig. 2a).

3.2 Results

Linear, semi-logarithmic, and logarithmic plots of the drawdown and recovery data were made by computer and evaluated. Only the semi-log plots of the drawdown phase are shown here (Fig. 3). Although the amount of drawdown was very small (between 8 and 17 cm), even after 170 hours of pumping, an excellent geohydraulic evaluation could be made, due to the high precision and large amount of data.

3.2.1 Aquifer Boundaries

All of the curves are, as expected, definitely influenced by the edges of the channel. For this reason, the curves for the more distant observation wells (E 250 - E 652) do not become linear until the end of the test. At this time however, the influence of both of the channel sides had already reached a maximum -- compare the curves for observation wells W 74, S 70, and E 90, which are near the pumped wells (wells 1 & 2). These curves each have three linear sections of differing slopes, separated by long transition stages. The intercepts of these linear sections with each other are labeled t_o (intercept with the normal water level, $s = 0$), t_1, and t_2. These values can be used (Bear, 1979) to determine the hydraulically effective distance to the boundaries of the aquifer as follows:

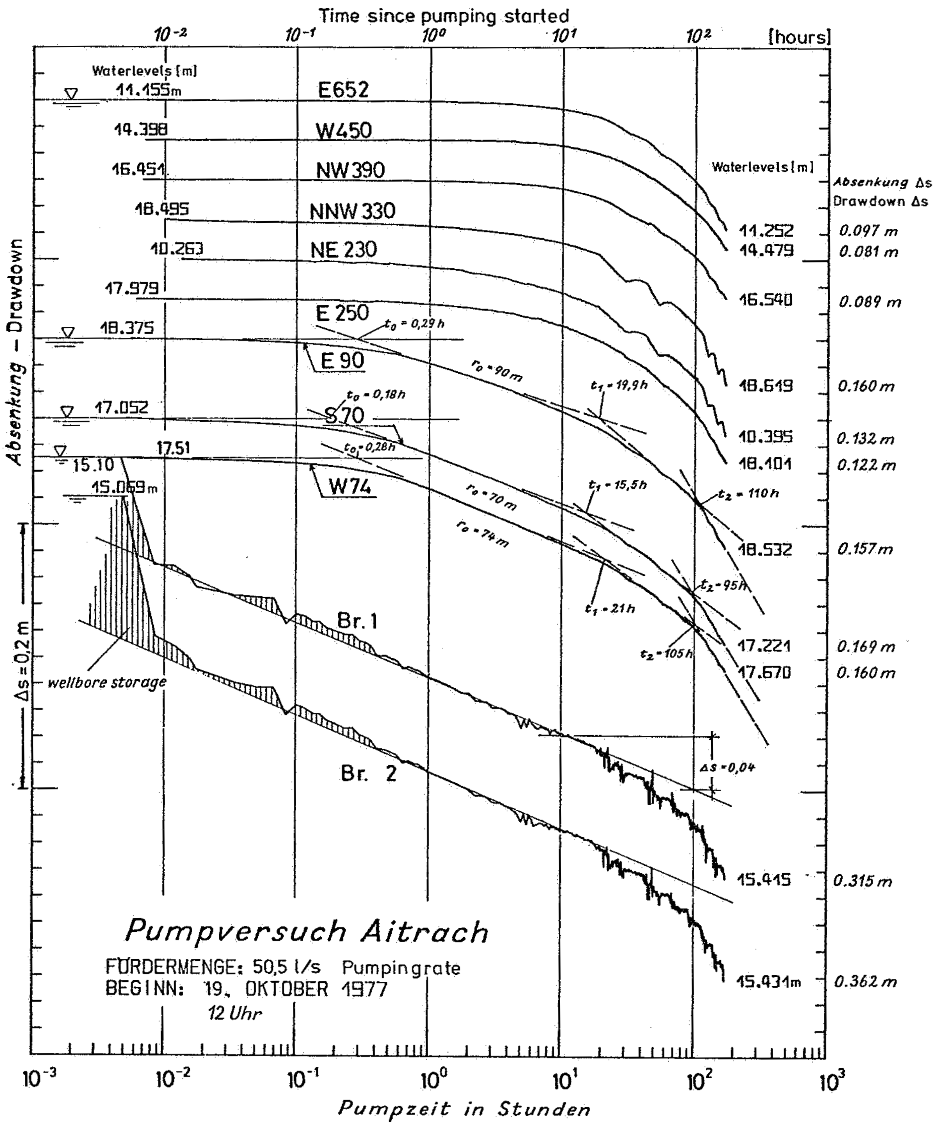

Figure 3 Drawdown data: pumping test at Aitrach

The linear form of the well equation yields the following formula for the radius r of the cone of depression when s = 0:

$$r = \left(\frac{2.25\ T\ t_o}{S}\right)^{1/2} \tag{1}$$

From this, it follows that

$$t/r^2 = \text{constant} \tag{2}$$

Accordingly, the following relationships are valid for the intercepts of the linear sections:

$$\frac{t_o}{r_o} = \frac{t_1}{r_1} = \frac{t_2}{r_2} = \text{constant} \tag{3}$$

If r_o is the distance from the observation well to the pumped well, then r_1 and r_2 are the distances to the first and second imaginary wells, respectively, that mathematically reproduce the influence of the boundaries of the aquifer. The aquifer boundaries are only r/2 apart. Using Equation (3), the distances $r_2/2$ = 877 m, 717 m, and 804 m were obtained for the three observation wells E 90, W 74, and S 70, respectively, where r_o = 90 m, 74 m, and 70 m, respectively. A plot of the aquifer boundaries on the basis of these distances (Fig. 2a) compares quite favorably with those of the structure determined hydrogeologically. The hydrogeological section (Fig. 2b) also clearly shows that the area of a rectangular cross-section corresponds closely to that of the actual aquifer cross-section.

3.2.2 *Aquifer Parameters*

To determine the transmissivity T and the storage coefficient S, only the first linear part of the curve, which represents about 20 hours, is necessary. This section is not yet influenced by the aquifer boundaries. The following values were determined from the slopes and intercepts of these curves:

Table 2 Transmissivity and storage coefficient values

well	aquifer thickness D (m)	transmissivity T (m^2/s)	storage coefficient S
1 & 2	26.0	0.231	not determinable
E 90	28.5	0.298	0.086
W 74	20.5	0.237	0.098
S 70	22	0.250	0.074

The transmissivity value for a well is quite certainly influenced by the effects of well loss, as well as "wellbore storage". The relatively small S value (under 0.1) for an unconfined aquifer indicates a strong z-anisotropy of the hydraulic conductivity, and a type-curve evaluation according to a method by Neuman (1975) actually yielded anisotropy factors between 4 and 10.

The fact that the aquifer is divided into two parts with differing permeabilities prevented the determination of a mean hydraulic conductivity k_f. A simple mean would not provide the desired information.

But if the proportions of "lower" and "upper" channel fill (Du and Do) were determined for the aquifer thickness (D), k_f values could be roughly estimated for the two parts of the aquifer when difference in the T values are considered.

Table 3 Transmissivity and aquifer thickness values

well	transmissivity T (m^2/s)	Du (m)	Do (m)
E 90	0.298	15	13.5
W 74	0.237	7	13.5
S 70	0.250	8	14.0

Using the difference in the Du values and in the values of T between the wells E 90 and W 74 and between the wells E 90 and S 70, k_{f_u} values of

$7.63 \cdot 10^{-3}$ m/s and $6.86 \cdot 10^{-3}$ m/s, respectively, were obtained, which differ little. The corresponding k_{f_o} values are $13.57 \cdot 10^{-3}$ m/s and $14.13 \cdot 10^{-3}$ m/s, respectively. That means the lower part of the channel fill is only half as permeable as the upper part.

References

Bear, J. 1979; Hydraulics of Groundwater; McGraw-Hill, New York, 567 pp.

International Electrotechnical Commission 1979; Byteserielles bitparalleles Schnittstellensystem für programmierbare Meßgeräte, Publication 625-1; or Institute of Electrical and Electronics Engineers, Inc. 1975, IEEE Standard Digital Interface for Programmable Instrumentation, IEEE Std 488-1975

Dürbaum H.-J., and R. Kohlmeier 1971; Digitale Registrierung von Wasserspiegelständen mit Hilfe von Ultraschall ...; Symposium Hydrometrie, Koblenz, 1970; Besondere Mitteilungen zum Deutschen Gewässerkundlichen Jahrbuch Nr. 35, pp. 1 - 10

Neuman, S.P. 1975; Analysis of Pumping Test Data from Anisotropic Unconfined Aquifers Considering Delayed Gravity Response; Water Research, Vol. 11, No. 2, p. 329.

MESURE AUTOMATIQUE DE LA SUCCION DANS UN SOL

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Résumé

Une station de mesure automatique de la succion dans un sol a été construite et installée dans un bassin versant de l'Alberta. La mesure de la succion se fait à l'aide de tensiomètres reliés à un robinet à voies multiples. Un capteur de pression réagit à la succion de chaque tensiomètre à travers le robinet à voies multiples; les valeurs de la succion aux tensiomètres sont enregistrées sur bande de papier. Un micro-ordinateur commande les fonctions de la station. Un panneau solaire permet le fonctionnement de la station de façon continue. On envisage d'équiper la station de telle sorte que les données puissent être transmises à un ordinateur central qui traitera les données. Le micro-ordinateur utilisé peut l'être pour commander la mesure d'autres paramètres tels que niveau de l'eau dans un puits ou un piézomètre, températures, pression, conductivités etc...

Abstract

An automatic soil suction measuring station has been constructed and installed in a basin in Alberta. Soil suction measurements are done with tensiometers linked to a fluid switch. A pressure transducer reacts to the suction at each tensiometer through the fluid switch. Values of the suction measured at each tensiometer are recorded on a strip-chart recorder. A microcomputer controls all the functions of the

station. A solar panel allows the functionning of the station on a continuous basis. Future plans are to equip the station in such a way that the data can be transmitted to a central computer. The micro-computer in this application could be used, with the proper interfaces, for the measurement of other parameters, such as water levels in a well or piezometer, temperatures, pressures, conductivities etc.

1 Introduction

Dans le cadre du développement d'un système automatique d'acquisition des données dont on parlera plus loin, nous avons étudié, développé et construit une station automatique de mesure et d'enregistrement que nous avons appliqué au départ à la mesure et à l'enregistrement des succions à des tensiomètres. A ce sujet la littérature offre un certain nombre d'exemples de stations qui ont été construites dans différents pays: entre-autres celles décrites dans les articles de Watson (1967), Bureau de Recherches Géologiques et Minières (1972), Vachaud et al (1974), Williams (1978).
Cette station est installée dans le bassin expérimental de Pine Lake en Alberta, Figure 1. Le bassin en question est situé à environ 150 kilomètres au sud-est d'Edmonton.

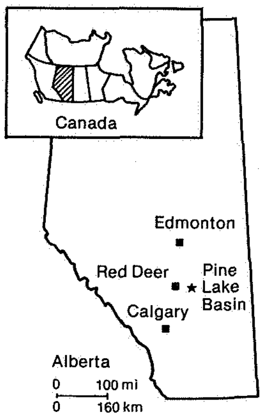

Figure 1 Localisation du basin du Pine Lake

Ce bassin de 230 km^2 de superficie fait l'objet d'une étude qui continue depuis 1978, Garven (1980), avec l'installation récente d'instrumentation pour étudier plus particulièrement:

a) la distribution des perméabilites, porosités, lithologies et épaisseurs des principales zones aquifères du bassin (grès paléocènes) à l'aide de forages et d'essais de pompage.
L'acquisition des paramètres hydrauliques aux forages est

importante pour la calibration de modèles numériques qui seront appliqués plus tard à d'autres régions de la province.

b) Le mouvement de l'eau dans la zone non saturée; et pour ce faire nous avons installé sur 9 sites des tubes d'accès dans lesquels des profils hydriques sont faits de façon périodique à l'aide d'une sonde à neutrons (Troxler). Des tensiomètres ont été aussi installés pour la mesure des gradients, et nous suivons les fluctuations de la nappe phréatique à un certain nombre de puits pour déterminer l'évapotranspiration.

Notre département veut donc développer un système automatique d'acquisition de données hydrogéologiques qui pour des raisons de coûts pourrait être développé et construit localement. Les raisons d'être d'un tel système sont les suivantes:

- Dans maintes régions de la province de l'Alberta (qui fait plus de 660,000 km^2 avec une population d'un peu plus de 2 millions d'habitants), l'accès est inexistant et on est limité à l'utilisation de l'hélicoptère ce qui est, de même que la main-d'oeuvre, très couteux.
- Un tel système éliminerait beaucoup d'erreurs humaines (reports de limnigrammes à la main par exemple).
- Les limnimètres que nous utilisons ne fonctionnent bien que pour des niveaux piézométriques qui se trouvent à une profondeur de moins de trente mètres.
- Le climat de l'Alberta qui en hiver est assez rude, avec des températures qui peuvent atteindre parfois - 40^{0}C, est aussi une autre source de mauvais fonctionnement d'instruments mécaniques.

Dans le futur nous avons donc en vue, un système qui, avec les "interfaces" voulus, nous donnerait la possibilité de mesurer et d'enregistrer des niveaux, des pressions, des températures, pH, conductivité, succion, ainsi que des paramètres météorologiques.

Dans ce qui suit nous décrivons dans un premier temps la station et les différentes parties qui la composent et, dans un deuxième temps l'évolution du système d'acquisition des données tel que nous le concevons dans le futur.

2 Description des différentes parties de la station dans un premier stade de développement

Dans un premier stade de développement la station comprend Figure 2:

a) Un micro-ordinateur construit par la compagnie Transwave Corporation de Pennsylvanie, U.S.A., Vanderbilt 8000 series, K8073. Ce micro-ordinateur utilise un micro-processeur de National Semi Conductor INS8073 MPU à tach orienté. Ce micro-ordinateur a une horloge à temps réel, un programmeur d'EPROM. Le langage de programmation est "NSC Tiny Basic". Le micro-ordinatuer possède 1 K byte de mémoire RAM qui peut être étendue à 9 K bytes. Il a aussi 2 K de mémoire de fonctionnement sur ROM. Il a évidemment en plus une prise de type RS 232 pour connection avec un terminal, et la possibilité d'avoir jusqu'à 24 signaux différents.

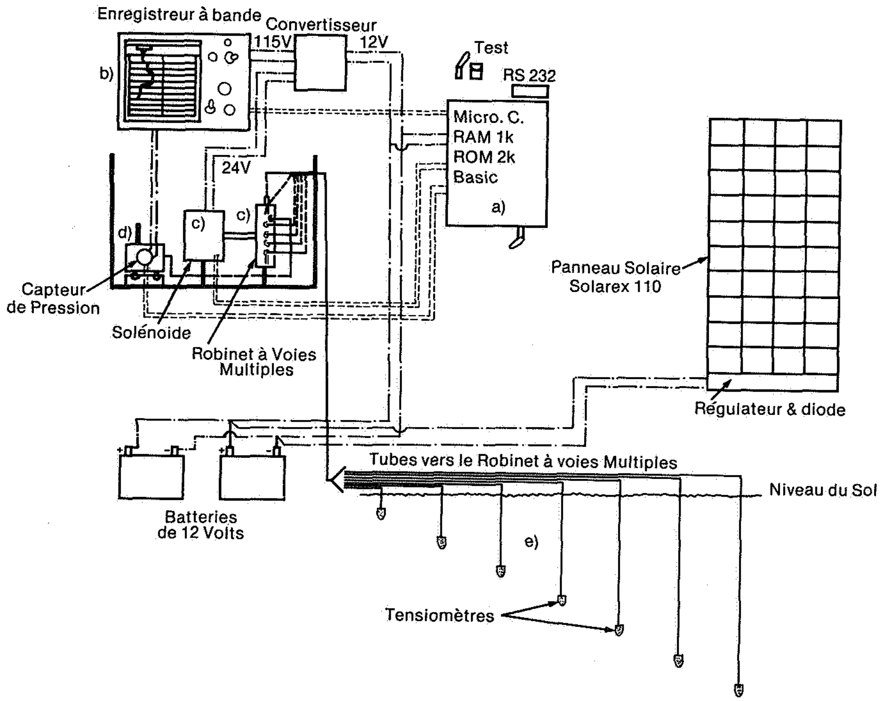

Figure 2 Schéma de la station automatique de mesure de la succion

b) Un enregistreur sur bande de papier (pour le moment!) distribué par la firme Markson de Californie. Cet enregistreur fonctionne sur 110 volts qui rechargent une batterie interne de 12 volts. Son échelle de sensibilité va de 1 millivolt à 10 volts. Et la vitesse de déroulement du papier peut varier de 10 centimètres à la minute à 0,5 centimètres à l'heure. Il est assez précis pour cette application.

c) Un solénoide permet le fonctionnement d'un robinet à voie multiples (12 voies). Sept de ces voies sont utilisées actuellement et sont reliées à un capteur de pression qui mesure la succion aux tensiomètres. Solénoïde, robinet à voies multiples et capteur de pression sont fabriqués ou distribués par la compagnie Scanivalve Corporation de Californie.

d) Le capteur de pression différentiel avec référence à la pression atmosphérique a une fourchette de mesure de -1033 à +1033 millibars. Ce capteur de pression a été calibré au laboratoire où l'on a utilisé une pompe à vide, un manomètre à mercure et un millivoltmètre pour déterminer la courbe de calibration.

e) Sept tensiomètres sont installés à la station; à un pas de 0,3 mètre; leurs longueurs respectives sont les suivantes: 0,19; 0,345; 0,495; 0,645; 0,955; 1,265 et 1,565 mètres. Leurs bougies en porcelaine font 5,5 centimètres de long. Ces tensio- sont fabriqués par la compagnie Soilmoisture Corporation de Californie, U.S.A.

3 Séquence de fonctionnement de la station

Le micro-ordinateur est programmé sur l'"EPROM, Table 1. Ce programme règle la séquence des évènements qui conduisent à la mesure des succions aux tensiomètres. Les mesures sont faites trois fois par jour, et la durée des mesures est de 19 minutes pour chacune des trois périodes de la journée qui ont été choisies.

Le micro-ordinateur qui est le seul element qui fonctionne continuellement, a une horloge à temps réel qui détermine le moment de la journée auquel les mesures seront faites. Lorsque le moment de la mesure est arrivé la mise en circuit de l'enregistreur et du capteur de

Table 1 Automatisation des mesures.

```
  5   X=#3F00
 10   $X="08000830120012301600163000000000000"
 15   @#B803=#9A : B=#B802 : L=7 : Y=0 : R=12
 20   GOSUB 135
 25   J=1 : GOTO 50
 30   @B=1 : J=2 : GOTO 50
 35   DO : Y=Y+1 : @B=5 : @B=1 : P=0
 40   DO : P=P+1 : DELAY 1000 : UNTIL P=120
 45   UNTIL Y=L : GOTO 15
 50   I=(@(X)-48)*10+(@(X+1)-48)
 55   N+(@(X+2)-48)*10+(@(X+3)-48)
 60   IF I=0 GOTO 5
 65   IF IS+H GOTO 95
 70   IF NS+M GOTO 95
 75   X=X+4
 80   IF J=1 GOTO 30
 85   IF J=2 GOTO 35
 90   STOP
 95   S=@#B802 : S=S/16 : IF S=15 GOTO 150
100   T=#3FC0 : LINK #83B0 : IF J=1 GOTO 125
105   IF J=2 GOTO 130
110   H=(@(T+5)-48)*10+(@(T+6)-48)
115   M=(@(T+7)-48)*10+(@(T+8)-48)
120   GOTO 65
125   @B=8 : @B=0 : GOTO 110
130   @B=9 : @B=1 : GOTO 110
135   FOR Z=0 TO R : @B=2 : DELAY 500 : @B=0 : NEXT Z
140   RETURN
145   REM TEST ROUTINE
150   GOSUB 135
155   S=@#B802 : S=S/16 : IF S=15 GOTO 5
160   @B=1
165   S=@#B802 : S=S/16 : @B=9 : @B=1 : IF S=14 GOTO 165
170   @B=5 : DELAY 500 : @B=1 : GOTO 155
```

pression est faite automatiquement. Au début de l'expérimentation on laissait chauffer l'enregistreur pendant 30 minutes, mais l'expérience acquise nous a permis de constater que c'était inutile, l'enregistreur n'est maintenant laissé à chauffer que pendant cinq minutes. La voie numéro un, qui correspond au tensiomètre le moins profond, est déjà mesurée pendant ces cinq minutes. Puis le solénoïde est activé, l'on passe alors à la voie numéro 2; cette voie d'ailleurs comme les autres est ouverte pendant 2 minutes car nous avons constaté qu'il faut un certain temps de stabilisation du circuit du tensiomètre qui est mesuré. Donc toutes les deux minutes un tensiomètre est mesuré; arrivé à la voie numéro 8, l'enregistreur, le solénoïde et le capteur de pression sont alors désactivés jusqu'à la période de mesure suivante où le cycle recommence.

Nous avons donc une mesure de voltage maximum à chaque voie, Figure 3, cette mesure est pour le moment convertie manuellement en millibars de pression négative à l'aide de la courbe de calibration du capteur de pression, dont on a parlé plus haut.

Lorsque l'appareillage est installé ou bien après que les tensiomètres aien été purgés il faut pouvoir être sûr d'ajuster le zéro de l'en egistreur. Ceci est fait de la manière suivante:

la voie numéro 8 est connectée à l'air libre; comme nous utilisons un capteur de pression à référence atmosphérique, lorsque la voie numéro 8 est ouverte nous avons donc la pression que nous considérons comme notre zéro. Un test peut être fait à la main à l'aide d'un interrupteur qui permet de faire fonctionner le solénoïde à la demande. Lorsque l'on est à la séquence de test et si l'on s'est arrêté à une voie quelle qu'elle soit sauf la no. 1, et que l'on passe alors à la séquence automatique, le solénoïde ramène le robinet à voies multiples à la voie numéro un automatiquement (ceci est fait par le programme).

Au printemps 1983 la voie numéro un sera la voie qui sera à la pression atmosphérique. Cette pression sera enregistrée et elle nous servira de repère pour les autres mesures. Une autre modification nécessaire sera aussi l'envoi d'un signal à l'enregistreur qui produira une marque verticale à la fin de la période de mesure de chacun des tensiomètres,

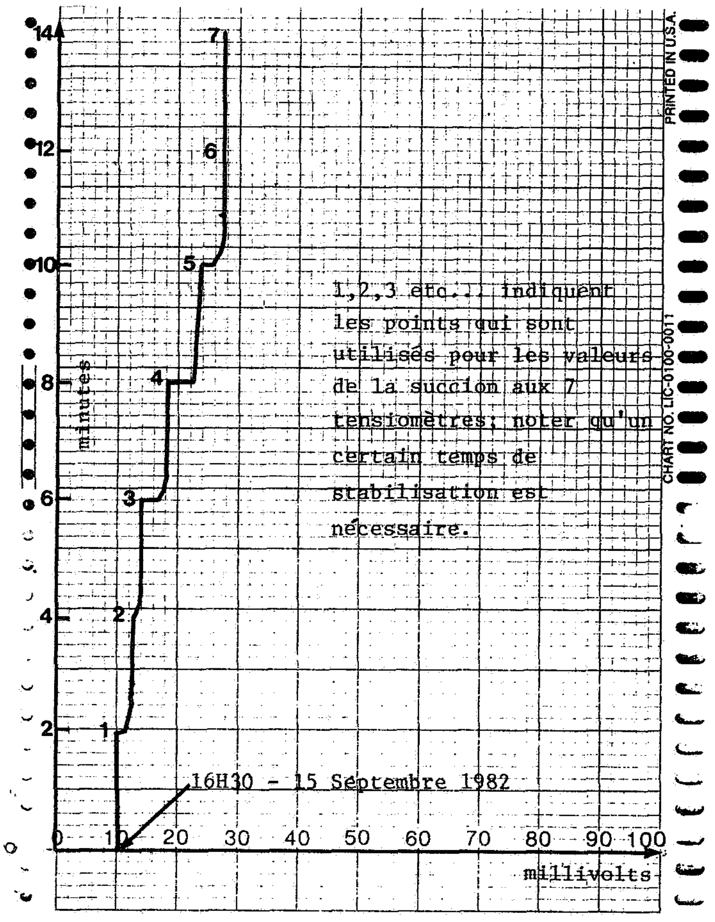

Figure 3 Fac-simile d'un enregistrement de la succion aux sept tensiomètres

car à l'endroit où la station est installée, les pressions mesurées à certains niveaux dans certains tensiomètres sont les mêmes et on ne peut les séparer.

Lorsque cet article paraîtra, la station aura été équipée d'un panneau solaire construit par la compagnie Solarex du Maryland, U.S.A. Ce panneau fournit 17,25 volts dans des conditions d'ensoleillement maximum de trois heures par jour avec un ampérage de 2,1 ampères. Ce panneau rechargera les batteries de la station; un régulateur de tension et une diode seront ajoutés de telle sorte que d'une part, le voltage aux batteries ne dépasse pas 14 volts et d'autre part pour éviter une inversion du courant des batteries vers le panneau solaire. Equipée ainsi la station devrait fonctionner de façon continue sans aide extérieure.

La station aura aussi fonctionné pendant une partie de l'hiver pour voir si les composants résistent bien au froid; les tensiomètres auront été déconnectés (ils l'auront été depuis octobre 1982).

4 Plans pour le futur

Dans le futur (1983-84) nous voulons remplacer l'enregistreur sur bande de papier par une mémoire interne ou bien par un magnétophone, ou encore une mémoire à bulles (il nous faudra évaluer la facilité de fonctionnement de chacune de ces options dans des conditions climatiques extrêmes). La mémoire interne sur RAM sera la plus intéressante et probablement la moins sujette aux problèmes de températures surtout si l'équipement est construit en utilisant les normes militaires (-55^0C à 50^0C). Huit ou 10K de mémoire devraient permettre l'enregistrement des informations pour une période de temps très longue.

A peu près en même temps nous étudierons ou ferons étudier et développer un système d'acquisition de données basé sur les mêmes principes. Il serait composé de deux unités, l'une laissée sur un puits et qui pourrait, à l'aide "d'interfaces" appropriés à chaque paramètre, mesurer les niveaux, la température, le pH etc... ou bien encore durant un essai de pompage mesurer et enregistrer une série de niveaux dans une série de puits d'observation en plus d'un débit, par exemple à l'aide d'un débimètre Halliburton au puits de pompage. L'information ainsi recueillie en mémoire interne, selon un taux d'échantillonage donné (et que l'on pourrait faire varier) peut être lue par une deuxième unité qui est une unité intelligentent de lecture et qui est capable de rendre cette information intelligible en unités S.I. m, m^3/s, ^0C etc... Cette unité intelligente devrait être capable de certains calculs ou encore de présenter de façon visuelle les résultats obtenus. De retour au bureau cette unité intelligente peut-être connectée à notre ordinateur central (VAX 70 de Digital) pour la préservation et la manipulation des données. A partir de ces données une représentation graphique automatique des rapports d'essais de pompage peut être faite (par exemple), pour ensuite en faire l'interprétation.

Plus loin encore dans le futur, nous voudrions pouvoir avoir un accès direct à l'information à l'endroit où elle est recueillie. Dans le cas de paramètres qui sont mesurés continuellement, nous pourrions utiliser les ondes radio, une ligne téléphonique, la retransmission par satellite ou bien encore utiliser les propriétés de retransmission à partir d'essaims de météores. Dans ce cas l'accès aux données serait donc direct et elles pourraient donc être traitées rapidement au lieu d'avoir à attendre par exemple plusieurs mois pour pouvoir les récupérer.

5 Conclusion

Bien qu'on ne soit pas encore arrivé à un système final, le fonctionnement de cette station automatique pour le mesure et l'enregistrement de la succion à des tensiomètres nous paraît être correct. L'emploi d'un micro-ordinateur permet de contrôler la station de façon efficace. Par des changements apportés au programme on peut très facilement changer les séquences de mesures. Cette station n'est qu'un premier pas vers un système d'acquisition des données plus sophistiqué qui sera dévloppé dans le futur.

Crédits

Nous remercions vivement J. Kinasewich et K. Croft de l'atelier d'électronique du Conseil de Recherches de l'Alberta pour leur travail de conception et de réalisation de certaines parties de l'électronique de la station.

Références

Bureau de Recherches Géologiques et Minières. 1972. La station expérimentale d'étude des mouvements d'eau dans la zone non saturée au service géologique national à Orléans-la-Source. Publication No. 72 SGN 295 AME. 10 pages.

Garven, G. 1980. Hydrogeology of the Pine Lake Basin, Alberta,Canada. Unpublished. U. of Arizona, M.Sc. thesis, Tucson, Arizona.

Vachaud et al. 1974. Station automatique du bilan hydrique in situ. Bull. BRGM deuxième série, Section III, No. 2, pp. 145-153.

Watson, K.K. 1967. A recording field tensiometer with rapid response characteristics. Journal of Hydrology No. 5, pp 33-39.

Williams, Lee T.H. 1978. An automatic scanning and recording tensiometer system. Journal of Hydrology No. 39, pp 175-183.

EQUIPMENT FOR WIRE-LINE PACKER TESTS OF LOW-PERMEABILITY FORMATIONS

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Abstract

The technical arrangement which is chosen for permeability tests, particularly when low-permeable deep-lying formations are investigated, is an essential factor regarding both the cost of the project and the reliability of the results.
The technical arrangement and function of a single and straddle-packer wire-line tool, pumping arrangement and data acquisition system are described.
This equipment was used to depths of 550 m b.s. in 9 inch uncased boreholes.

Resumé

Le choix de l'installation technique à utiliser pour les essais de perméabilité, en particulier dans les études de couches peu perméables à grande profondeur, est un facteur essentiel pour calculer les coûts du projet et la crédibilité des résultats.
La mise en place et la fonction d'un seul packer et d'un "straddle - packer" attaché au câble, le dispositif de pompage et le système d'acquisition de données sont décrits.
Ce matériel a été employé à des profondeurs de 550 m au-dessous du niveau de la mer, dans des puits de 9", sans tubage.

1 Introduction

The proper choice of testing equipment, particularly when deep boreholes are considered, is essential for the evaluation of the costs and results of an investigation.
The testing equipment for the determination of transmissivities in the open section of a borehole usually consists of one or more packers, pressure transducers and a pumping arrangement.
The equipment can be attached either to a drill pipe or to a wire. The experience regarding the testing equipment and the data acquisition system, which the authors acquired during the hydrological investigation of the tight Chalk formation at Mors (Northern Jutland, Denmark,) is discussed here. More than sixty tests were performed in four 550 meter deep boreholes.
The results of the investigation were reported earlier by Gosk, E. (1981), Andersen et al. (1981a) and Andersen et al. (1981b). Additional information about the Mors-investigation can be found in two other communications presented at this conference: Bull et al. (1983) and Gosk, G. (1983).

2 Wire-line packer test equipment

During the Mors-investigation, several technical arrangements were tested. Due to the low transmissivity of the tested Chalk formation, it was necessary to eliminate the effect of storage in the borehole. In one of the first arrangements, this was done using packers attached to the drill-pipe. The results showed however, that even drill-pipe storage made evaluation of transmissivity difficult. Furthermore, the round trip with drill-pipe was time consuming and problems with its tightness, particularly during injection tests, occurred.
In order to save time and achieve more reliable results, a wire-line apparatus was designed, Fig. 1.
There were two versions of the wire-line equipment. The technical arrangement, shown in Fig. 1, is the straddle-packer version, but the single-packer version was very similar and for this reason only the shown arrangement will be described.

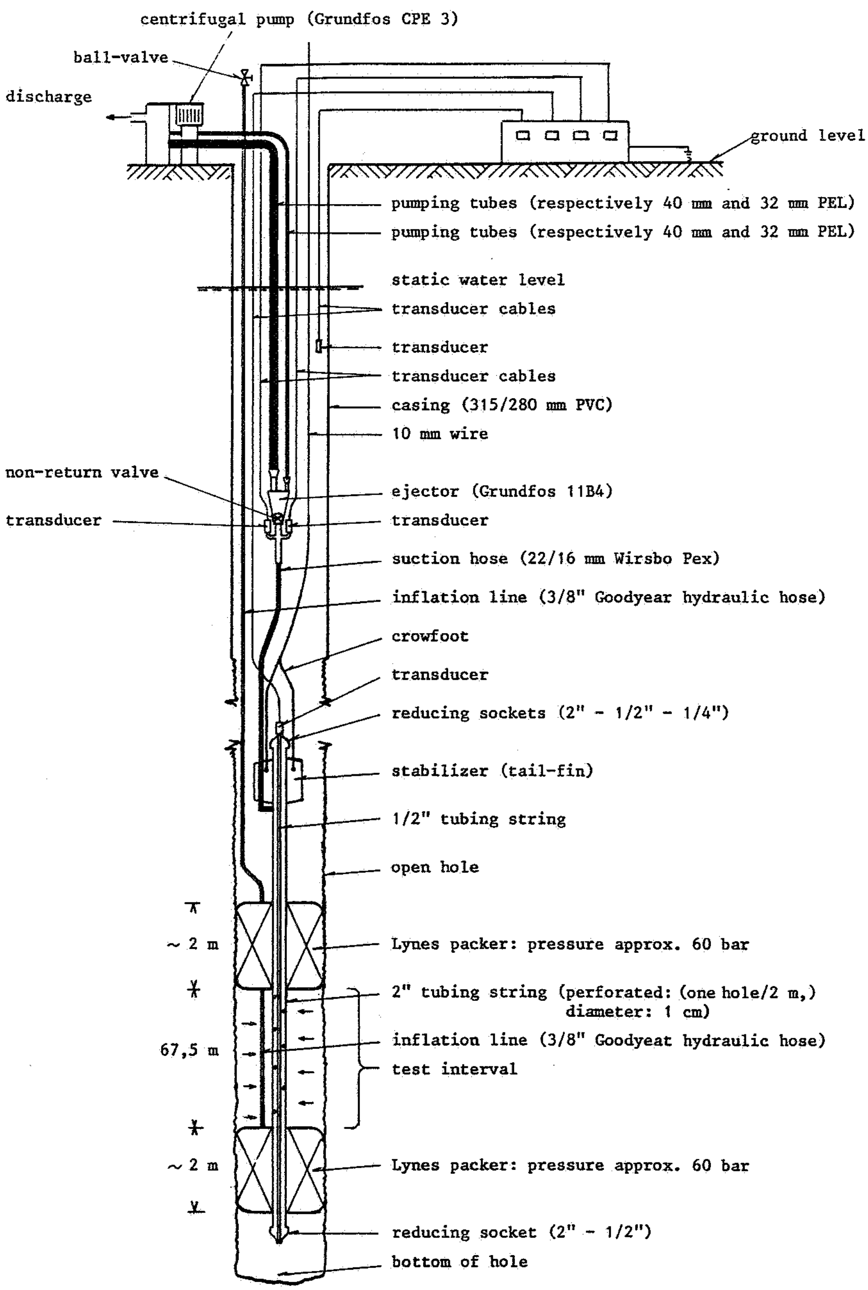

Figure 1 Technical arrangement for Straddle Packer Test

The equipment consists of:

- two LYNES packers connected by approximately sixty meters of perforated 2" pipe.
- an ejector (Grundfos 11B4) attached to the packers by a 22/16 mm or 15/10 mm suction hose.
- two 60 bar Drück pressure transducers measuring pressures below the lower packer and within the tested interval.
- a 1 bar transducer for detection of the water level in the casing.
- a centrifugal pump (Grundfos PE3) connected to the ejector by the pumping tubes (40 and 32 mm in diameter).
- a 10 mm steel wire.
- a data acquisition system HMS300 (Hydrological Measuring System).
- an injection line (for certain tests only).

The LYNES packers proved to be reliable and well-suited for the purpose. During one of the tests, an attempt to set the packer in a large diameter interval resulted in bursting of the rubber coating, but the repair (exchange of the rubber) was done on the site and took only 1/2 hour. The inflation pressure was normally 60 bars.

The pumping arrangement, consisting of an ejector and a centrifugal pump, gave great flexibility with regard to combinations of flow rates and lifting capacities which is very useful for testing formations with highly varying properties.

The pressure transducers were calibrated and their characteristics, including temperature correction, were coded into a microprocessor (see chapter 4).

3 Testing methods

The equipment shown in Fig. 1, was used for four different types of tests:

- constant rate pumping tests.
- step drawdown tests.
- step injection tests.
- pressurized slug tests.

Constant rate pumping tests were performed either as single or straddle-packer tests and the duration of pumping and recovery periods ranged typically from a few hours to a few days. Pressure drawdowns from 50 to 120 m of water column and flow rates from 0.1 to few l/min were characteristic for these tests. The transmissivity for these tests was calculated from the recovery data.

Step drawdown tests were used to determine pressure-dependent permeability. Several quasi steady state situations (constant drawdown and flow) were obtained by introducing additional flow resistance (gradual closing of the valve on the pumping tube). The variation of transmissivities, corresponding to the different steps, was small and it was concluded that permeability did not depend upon the pressure.

The purpose of step injection tests was similar to that of step drawdown tests. A hand-operated piston pump was used to provide constant injection rates and constant overpressures. Due to the low permeability of the chalk, $\sim 10^{-10}$ m/sec, pressure oscillations with every stroke of the pump occurred and it was difficult to obtain quantitative results.

During the pressurized slug test, a pressure pulse was applied in the tested interval by injection of a small amount of water. The pressure decline as a function of time was used for an evaluation of the transmissivity.

The performance of the equipment during all the above-mentioned permeability tests was satisfactory.

4 Data acquisition system

The data collected during wire-line packer tests consisted of pressure and flow rates measured as a function of time.

For the investigation, a special Hydrological Measuring System (HMS300) was designed by the Danish firm of Gunnar Larsen.

The HMS300 consists of:

- pre-programmed microprocessors for the control of the system.
- data logger.

- chart recorder.
- Keyboard Send-Receiver (KSR) data terminal.
- emergency power supply.

High pressure and temperature accuracy was required and an extensive calibration of the transducers, together with their cables, was carried out. The characteristics of each transducer within the range of 0-60 bar were obtained and coded into the microprocessor. In the field situation, the incoming signals were corrected for temperature and non-linear behaviour using the calibration functions stored in the computer. For each of the six 0-60 bar input channels, it was possible to achieve an accuracy for absolute measurement, of better than 0.03% and a resolution of 25 mbar. For relative measurements, the resolution was 2.5 mbar. The temperature was measured with a 0.06 ^{o}C accuracy and a 0.01 ^{o}C resolution. The computer control of the HMS300 provided the user with an option to define several successive data scanning frequencies and made it possible to run tests of long duration without an operator on the site.

The choice of an optimal scanning frequency is essential if the amount of data is considered. During hydrological tests, the pressure readings had to be taken very frequently at both the start, and immediately after, the stop of the pump. After these highly transient phases of the test, the frequency may be drastically reduced without any loss of information. The HMS300 provided the possibility to use five different frequencies for five different time intervals. As the transmissivity evaluation was based on recovery data, it would have been costly to lose these data as the scanning frequency was usually low at the moment when the pump stopped. One of the preprogrammed routines of the HMS300 helped to overcome this problem by resetting the scanning sequence after the pump was stopped.
As unattended testing requires control of the equipment's performance as well as security control on the site, the HMS300 was provided with several emergency functions:

1) a switch on the emergency power supply.
2) activates the high frequency scanning of data in case of pump

failure.

3) activates an alarm in case of intrusion on the site.

All these functions were followed by a telephone call to a specialized company which took further action according to instructions received beforehand.

A warning about the end of tape on the data logger and the end of paper on the terminal was also included on the HMS300.

The keyboard terminal proved to be a very important part of the equipment. It proved an easy means of communication with the system and made it possible to keep a detailed track of events at the drilling site. All the messages (alarms, resets, etc.) were printed out, giving an excellent documentation for reporting, and the messages typed by the operator helped to reconstruct the events. Printouts of the pressure records were used to make plots and preliminary evaluation of the tests.

5 Summary and conclusions

The experience with the wire-line packer equipment and the data acquisition system may be summarized as follows:

1) The wire-line packer equipment can be used for different tests: constant and stepwise pumping or injection, slug tests etc.
2) The equipment proved to be reliable under different test conditions.
3) For the constant rate pumping tests, only the recovery data can be used for transmissivity evaluation.
4) The accuracy of the measurements is significantly improved if a proper calibration of the transducers is performed.
5) Savings in cost and time can be obtained if the computer is used to control the tests.
6) The Keyboard Send-Receiver (KSR) data terminal is an important piece of the recording system as it provides excellent documentation for the events which take place at the test site.
7) Duplication of measurements is advisable, as it can save a round trip in case of the transducer's failure.

References

Andersen, L.J., N. Bull and E. Gosk 1981. Results of a Labelled Slug Test in a Low Permeability Formation of White Chalk, Mors, Denmark . Proceedings of the Workshop on Siting of Radioactive Waste Repositories in Geological Formations, Paris 19-22 May, 1981.

Andersen, L.J., L. Byrne and P. Frykman 1981. Results from Impression Packer Technique for Location of Fissures in Boreholes in White Chalk, Mors, Denmark . Proceedings of the Workshop on Siting of Radioactive Waste Repositories in Geological Formations, Paris 19-22 May, 1981.

Bull, N., E. Gosk and L.J. Andersen 1983. The Labelled Slug Test . Proceedings of the International Symposium on Methods and Instrumentation for the Investigation of Groundwater Systems, Noordwijkerhout, 2nd-6th May 1983.

Gosk, E. 1981. Hydraulic Testing of Low Permeable Chalk Formation, Mors, Denmark. Proceedings of the Workshop on Siting of Radioactive Waste Repositories in Geological Formations, Paris 19-22 May, 1981.

Gosk, G. 1983. Computer programs for processing data from the Labelled Slug Test and the Spinner Flowmeter Test . Proceedings of the International Symposium on Methods and Instrumentation for the Investigation of Groundwater Systems, Noordwijkerhout, 2nd-6th May 1983.

SAMPLING TECHNIQUES OF GROUNDWATER FROM WATER WELLS

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Abstract - Resumé

Several types of groundwater sampling techniques are described and discussed. Packer samplers able to collect groundwater from arbitrary levels of an open or screened section of a water well are used. The first version utilize a triple-zone-packer unit whereas the two other types use a auto-setting single sampler. All equipment requires pumping and contain an auto-sampling collecting vessel, emptied either by nitrogen or by the pressure of the pumped water.
Groundwater sampling is itself as importent as the analysing procedure. Water samples from fully penetrating wells in aquifers with varying head may not represent the original water of the aquifer.

De nombreux techniques employés pour les prises d'échantillons sont décrits et commentés. Des échantillonneurs packer qui peuvent prendre des échantillons d'eau souterraine à des profondeurs arbitraires d'une section ouverte ou protégée d'un puits sont utilisés. Le premier modèle utilise un triple-zone-packer, tandis que les deux autres modèles prélèvent les échantillons automatiquement. Les échantillonneurs sont vidés par l'azote ou par la pression de l'eau pompée. Le prélèvement d'échantillons d'eau souterraine est aussi important que son analyse postérieure. Les échantillons de puits à pénétration complète dans des aquifères à charge variable ne représentent pas toujours l'eau originelle de l'aquifère.

1 Introduction

Analyses of groundwater and formation water are expensive and with the increasing pollution of subsurface water, water samples will be collected in increasing numbers in the future for monitoring and localizing groundwater pollution. An important task related to both groundwater monitoring and groundwater quality is groundwater sampling. Representative samples are absolutely necessary in order to obtain usable results, and where this is impossible, analyses should be omitted since non-representative samples are of no value and make accurate interpretation difficult to impossible.

When sampling groundwater in connection with drilling, care should be taken with respect to pollution of the sample from the drilling fluid. Mixing of groundwater from different levels may be difficult to eliminate when sampling from existing water-supply wells.

Three types of samplers using a packer-technique developed and used at the Geological Survey of Denmark will be described and discussed below.

2 Triple Zone Sampler

This sampler is described elsewhere by Andersen (1979). (Figure 1).

Four inflatable packers separate three zones of a screened well or an open borehole. A water sample is collected from the middle zone during continuous pumping from the upper and lower zones. The pumping, as described above, creates divergent flow lines of the groundwater within the sampling interval. The inflatable vessel under the lower pressure, governed by the drawdown from the pumping, collects the groundwater from the middle zone without the possibility of pollution from leakage around the packers. If leakage does take place, the direction of the leakage would be from the middle zone to the upper or lower zones. The tightness of the packers therefore does not adversely influence the quality of the sampled water. However, in cases of high leakage, the groundwater head from the middle zone could be approaching that of the zone from which the water leaks. This could result in a low hydraulic gradient between the sampling zone and the collecting vessel and a slow inflow, if any, to the collecting vessel. However, if any water at all

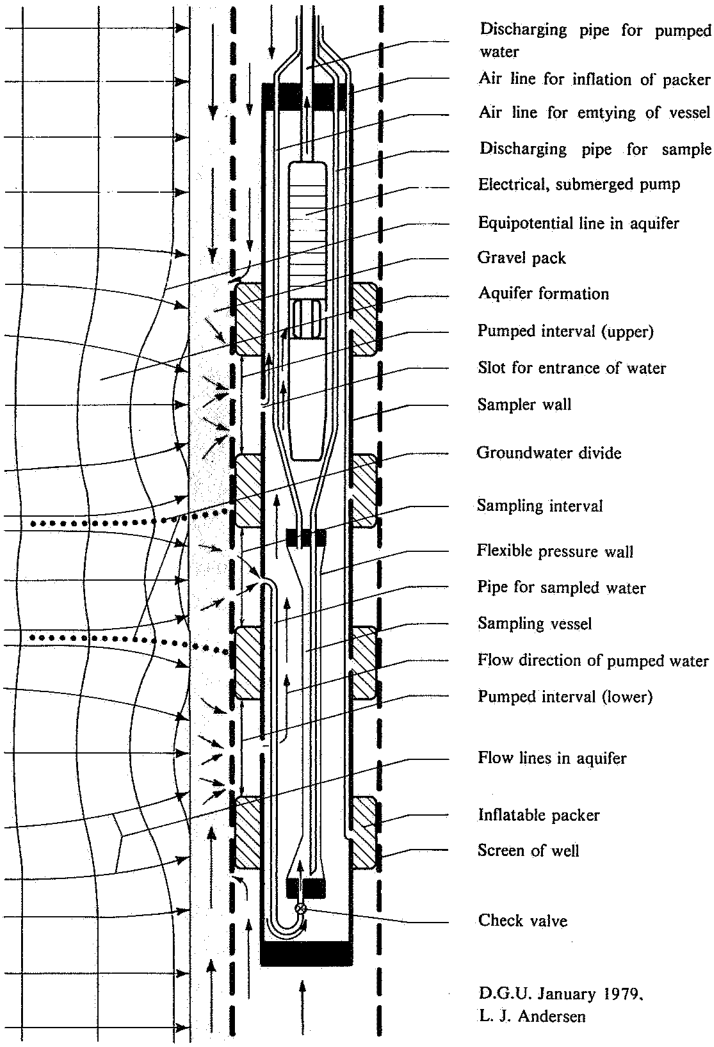

Figure 1 Schematic illustration of the Triple Zone Sampler

is collected in the collecting vessel, it can be concluded that it derives from the middle zone. The vessel has to be emptied a number of times, dependent upon the ratio of the volume of the vessel and that of the sampling interval. The sampled water in the vessel is discharged to the surface using compressed nitrogen.
This type of sampler has been used successfully for localizing the nitrate front in water-table aquifers of meltwater sand penetrated by screened water wells by Andersen and Kristiansen (1982).

3 Single Packer Sampler

The single-packer devices described below make use of the same sampling principle as the Triple Zone Sampler. A first version described by Andersen (1982), (Figure 2), uses an inflatable packer combined with a collecting vessel of the same type as that of the Triple Zone Packer. The sampler is set by the pumped water and starts sampling just after the setting. The water sample is discharged by compressed nitrogen. A second version described below, (Figure 3), is auto-setting and auto - sampling and the sample is brought to the surface using the pumped water pressure as the driving force. The collecting vessel is placed inside the packer between two rubber sleeves, surrounded by water of the same head as inside the well during pumping. Piercing tubes in the bottom of the equipment secure this equalization. A belt of porous material located at the middle part of the packer limits the zone of sampling. A submersible pump produces drawdown in the well, sets the packer and initiates a water flow from the sampling zone to the collecting vessel (eg. the space between the sleeves). The pressure of the pumped water is directed to the exterior part of the collecting vessel through a valve which is electrically or mechanically activated from the pumping system. The total sampling equipment consists of a submersible pump, the single-packer-sampler unit, and only three connecting lines to the surface, (eg. the discharging pipe for pumped water, the electrical cable to the pump and the discharging pipe for the sampled water).
This equipment is usable for sampling shallow groundwater to a depth of about 50 meters. In case of greater depths, the samples should be stored

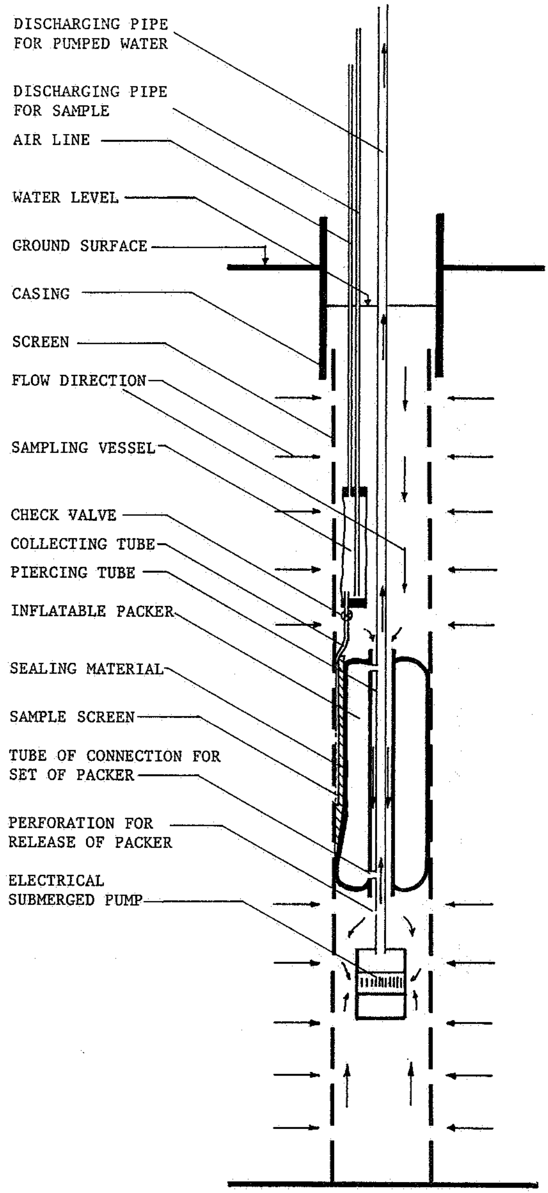

Figure 2 Schematic illustration of 1. version of a single packer sampler

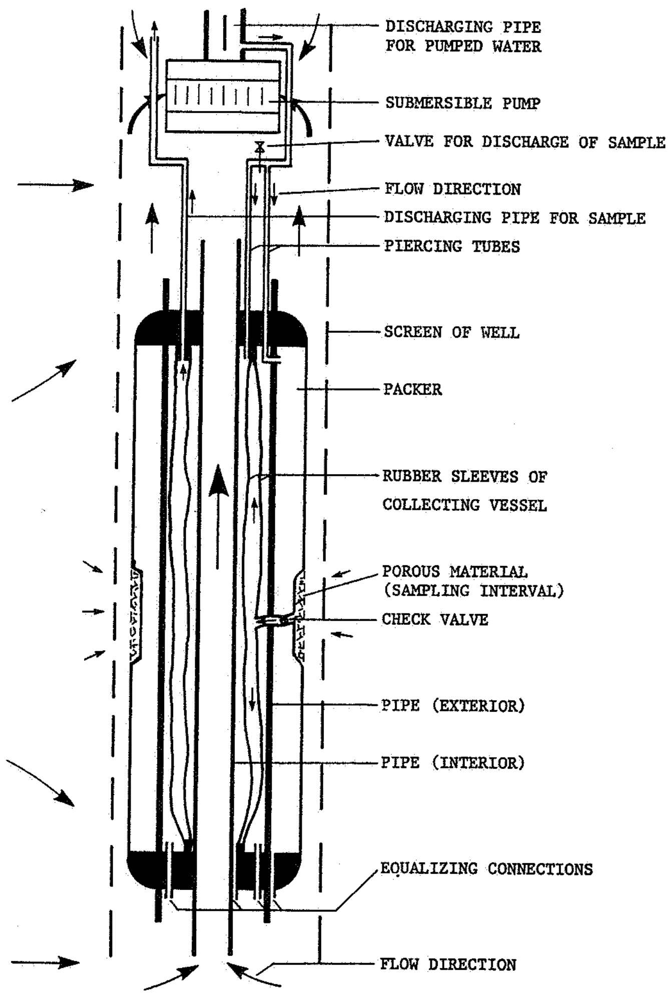

Figure 3 Schematic illustration of auto-setting, auto-collecting and semi-auto-sampling single packer sampler

in collecting vessels, able to maintain the natural pressure of the groundwater. In order to get the samples to the surface the whole equipment should be removed from the borehole.
The design and development of the equipment, Figures 2 and 3, are speculative at present, and have not yet been tested in the field.

4 Limitation and recommendations

The penetration of an aquifer by water-supply wells may connect zones of different head. When such wells are not pumped, a flow of water takes place from one zone to another through the well. Within the discharging zones, water sampled by the equipment described above will be representative of the formation water. Within zones of invasion, water samples will only be representative after back pumping of the invaded water.

5 Conclusions

The groundwater samplers described in this paper are able to collect formation water without mixing with the liquid inside the well. In case of invasion however, representative samples of formation water may only be sampled after the invaded water has been removed.

References

Andersen, L.J. 1979. A semi-automatic level-accurate groundwater sampler. Geological Survey of Denmark, Yearbook, 1978, Copenhagen, 1979, pp. 165-171.

Andersen, L.J., N. Kelstrup and H. Kristiansen 1980. Chemical profiles in the Karup water-table aquifer, Denmark. IAEA. Nuclear techniques in groundwater pollution research, Vienna, 1980, pp. 47-60.

Andersen, L.J. and H. Kristiansen 1982. Nitrate in groundwater and surface water related to land use in the Karup Basin, Denmark. International Symposium IAH, Prague Czechoslovakia 1982. Impact of Agricultural Activities on Groundwater, Prague 1982, pp. 14.

DIGITAL HYDROLOGIC DATA ACQUISITION SYSTEM

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Abstract

A data aquisition system which monitors groundwater or surface water levels and loads the recorded data into a computer is described. It features a novel bore water level transducer and a portable multi-purpose data reader. The equipment is relatively inexpensive, adaptable to other applications and has proved to be reliable and accurate in a field study. The system requires minimal operator time to load and verify the data.

Résumé

Une description d'un système d'acquisition des données qui contrôle les niveaux d'eaux souterraines et d'eaux de surface, et qui alimente un ordinateur en données enregistrées. Il y a un nouveau transducteur de niveaux d'eau souterraine et une mémoire portative et multi-usages pour fournir des données. L'équipement est relativement bon marché, adaptable à d'autres applications, et s'est révélé fidèle et précis au cours des essais sur le terrain. L'alimentation et la vérification des données demandent peu de temps dans ce système.

1 Introduction

A study of groundwater recharge from a regulated stream overlying a heavily exploited aquifer has been undertaken jointly by the Engineering and Water Supply Department of South Australia and the University of Adelaide. Flow exchange between the stream and the groundwater was found to depend on the position of the phreatic surface adjacent the stream. This was monitored at selected observation bores. Initially a chart recorder with a pulley and float arrangement for detecting water level changes was installed at each site. However this produced staircased records due to pulley stiction and the small water plane cross-section of the float, whose size was restricted by the casing dimensions. Charts had to be replaced weekly and the recorders were unreliable in the humid environment in which they were placed. There remained the onerous task of manually digitizing the charts and entering the data into computer files for subsequent analysis. Alternative commercially available recording equipment was beyond the study's budget. To resolve these problems a digital recording system was designed, developed and constructed chiefly by Mr. S. Woithe an instrumentation technician at the University of Adelaide. This system has been in routine operation for nine months, recording groundwater levels at six observation bores and stream stage and water temperature at several river gauging stations. The recorded data is fed into computer files.

2 System Operation

The data aquisition system is composed of five pieces of equipment in addition to a computer. See figure 1. The transducer and recorder are located at the observation bore or gauging station. At regular intervals the battery and solid state memory of the recorder are replaced and date, time and an independent measurement of water level are recorded on a field data sheet. Warning lights on the recorder indicate whether the recorder is operating correctly. The portable reader is plugged into the recorder at site visits to give a direct reading of momory contents as an additional check.

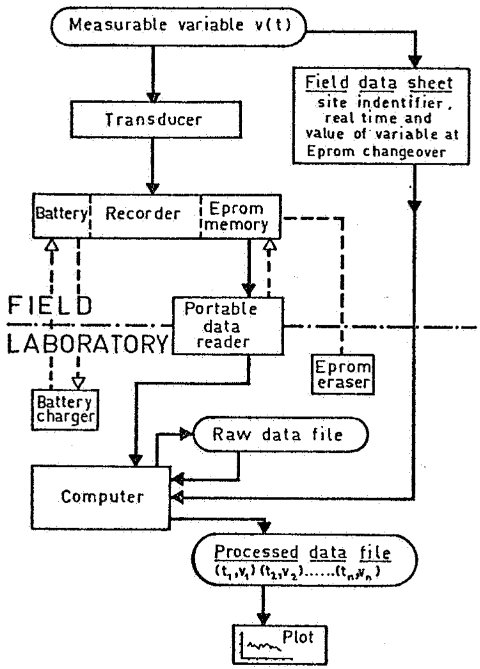

Figure 1 Data aquisition system block diagram

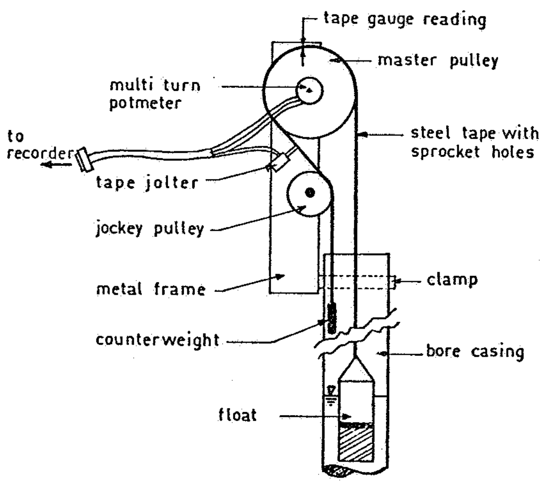

Figure 2 Groundwater level transducer

Later back in the laboratory the memory is plugged into the reader and this is coupled to a port on a PDP11 computer. The data is fed down the line and stored on a floppy disk. An interactive editing program is engaged on a computer terminal and the contents of the field data sheet are keyed in. The raw data file is automatically edited, scaled, reformatted and saved as a processed data file. The program also performs basic checks on data quality. A calcomp (automated) graphical plot is obtained as an additional check. Such checks are a vital component of any recording system (Brown, 1980). The verified data is then merged into the data bank. The solid state memory is erased using ultraviolet light and the battery is recharged ready for the next change over.

3 System components

3.1 Transducer

A rise in groundwater level lifts the float (figure 2) attached to a punched stainless steel tape which is wrapped around a sprocketted master pulley. As the pulley turns, the shaft of a multi-turn potentiometer rotates causing a variation in electrical resistance. The voltage across the potentiometer terminals is the transducer output. A jockey pulley directs the counterweight down inside the bore casing. A tape jolter is activated prior to the recorder taking a reading and the impact is sufficient to overcome pulley stiction and friction between float and casing. The jolter and potentiometer circuits are only powered for a short time before and during readings. Where water levels are monitored in bore casings greater than 250 mm diameter the jolter and jockey pulley are not required. The transducer has a range of 3750 mm with a resolution of 1 mm and is linear to ±2 mm over the full range. Transducer output is insensitive to temperature from 0° to 50°C. In straight casing the jolter allows floats as small as 50 mm diameter to give water levels within ±2 mm of their true value for static and dynamic tests. For water temperature recording a thermocouple is used as the transducer.

3.2 Recorder

The recorder is of modular design and consists of four standard size printed circuit boards. These provide regulated power supplies, microprocessor control, a transducer interface and solid state memory.

3.2.1 Power supply

The power supply board produces a number of independent regulated supplies for operating the recorder and transducer.

3.2.2 Microprocessor control

A reprogrammable chip on the microprocessor board controls the operation of the recorder and gives it remarkable versatility. The recorder takes readings at time intervals of either 10 seconds, 1 minute, 10 minutes or 1 hour, selected by setting a switch on the recorder. Each reading is started by activating the jolter, and pausing. The voltage reading across the potentiometer is then converted to a digital value.

If the reading is different to the last reading stored in the solid state memory by more than a specified threshold value, then this reading and the time increment number are stored in the next memory location. If more than a specified period of time has passed since the last value was stored then the new reading and time increment number are stored regardless. If neither of these conditions apply no new values are stored and the recorder 'rests' until the next time interval has elapsed. At the time of removing the solid state memory, a manual over-ride ensures that the current data are stored.

This data compression procedure reduces the number of data stored without loss of information, giving more efficient use of memory storage in the recorder and later also in the computer. If spectral analysis is required the threshold values are set to zero to comply with Fast Fourier Transform routines.

After 65000 time increments (45 days at one minute time increments) the time value is reset to zero and continues counting. Therefore only data storage and power supply capacity limit the length of unattended field operation. In the event of a power failure the memory is not erased.

3.2.3 Transducer interface

One board is used to interface the water level transducer with the recorder. This contains scaling potentiometers which enable very fast calibration of the transducer. The analogue voltage from the transducer is converted to a digital signal on this board. When monitoring temperatures only this board need be replaced.

3.2.4 Solid state memory

The fourth board contains a bank of eight semi conductor chips (eproms) each of which store 500 pairs of time and data value readings. Thus one board can record a total of 4000 reading pairs. The time reading is stored as a four digit hexidecimal number (resolution 1:65000) and the data value is stored as a four digit decimal number (resolution 1:10000). The eproms are erasible using ultra-violet light. Two boards are required for each recorder so that one can be brought in for reading while the other has its turn in the recorder.

3.3 Reader

A microprocessor unit mounted in a briefcase serves as a recorder checking tool and as a computer interface for the eprom boards. When plugged into a recorder it displays the last time and value stored in the memory and the current time and value. Back in the laboratory the contents of an eprom board are fed into the computer by plugging the board into a socket on the reader which is connected to a computer terminal line. No additional hardware is required. One reader can service a data network of many recorders.

3.4 Memory Eraser

A pair of ultraviolet light tubes are used to erase up to four eprom boards simultaneously. Erasure takes approximately 30 minutes. The reader is used to check that each board is completely erased and ready for re-use.

3.5 Battery charger

A safe unit to fast charge sealed rechargeable batteries has been developed. A fully drained 8 Amp-hour 12 Volt battery can be fully recharged in 6 hours. The buildup of gas within the battery is minimized with a consequent reduction in the likelihood of explosion. Lead-acid batteries are used as they are relatively cheap and do not have the "memory" problems of Nickel-cadmium batteries. Each gives about 8 weeks power supply for a transducer and recorder at one minute resolution.

4 Example of data aquisition

The Little Para River recharge study is located in a metropolitan area so the likelihood of undesirable interference with equipment was averted by concealing it underground. Concrete boxes 400 mm deep were constructed and covered by steel-rimmed fitted concrete lids 600 mm x 450 mm set at ground level. The top of the observation bore casing penetrated the bottom of the box. Temperatures ranging from 3° to 44°C were observed and humidity frequently reached dew point. This harsh environment did not impede recorder performance.

Thresholds of 50 mm and 4 hours were programmed in the recorder microprocessor as the maximum change in groundwater level and maximum time lapse between successive data entries in memory. A time increment of 1 minute was set. Eproms and batteries were exchanged at intervals of about 6 weeks. These values of the thresholds were chosen to allow detection of diurnal fluctuations in groundwater levels due to evaporation and transpiration of phreatophytes flanking the stream. The data value threshold was only triggered when streamflow commenced after the bed had been dry or when nearby irrigation bores were in use.

Examples of the field data sheet, raw data file, processed data file and calcomp plot for a short period of record from an observation bore adjacent the river are given in Figures 3, 4, 5 and 6 respectively.

5 Costs

This system was designed to keep the capital outlay below Aus$600 per field installation and to minimize operator intervention at all stages of system operation. The materials costs and the labour involved in producing the equipment is outlined below:

	AUS$	Manhours
Transducer - bore	50	10
- temperature*	20	2
Recorder**	420	18
Reader + battery charger + eprom erasor	560	40
Batteries (2)	80	-

*includes temp transducer interface board

** includes bore water level interface board and 2 eprom boards.

Prices are in 1981 Australian dollars and labour times cover fabrication and an electronics technician's time in assembling and testing the equipment.

AUS $1 = 2.6 Dutch Guilders; 6.6 Fr Francs; 2.4 Deutsche Marks; 0.56 Pounds; US$0.98 (August 1982).

Operation costs depend principally on travel time to reach the recording equipment. Changeover of memory and battery and equipment checks at a field station take only ten minutes. Reading the data and producing a processed data file requires about twenty minutes per station. Perusing the calcomp plot and sorting the processed file into the data bank requires a further ten minutes. Memory erasure and battery recharging require minimal operator time.

Little Para recharge study field data sheet.

Bore No. 36 Eprom No: 12 Battery No: 8

	ON	OFF	Δ=CHANGE ($\frac{MM}{Mins}$)	ERRORS
Date }T Time	18/9/81 1805	9/10/81 1524 }	30079	δ_T = ΔT − ΔET δ_V = ΔTG+(ΔEV)
GL (m)	4·928	4·840	−88 ±14	D=mean(TG−GL)
TG (m)	8·535	8·442	−93 ±2	= 3·602 m. (15 readings)
ET (min)$_{16}$	0001	7592	30097 (dec)	δ_T = −18 min
EV (mm)	1865	1961	96	δ_V = 3 mm
checks	✓	✓		
initials	PJD	PJD		$GL_1 = TG_1 - D$ = 4·933 m

GL = sounding gauge reading (st error = ±7 mm/reading)

TG = tape gauge reading (st error = ±1 mm/reading)

ET = eprom recorded time

EV = eprom recorded data value

File Names: Raw data file: R361081 Processed data file: P361081

Figure 3 Example field data sheet

time	value	time	value	time	value	time	value
Ø001	1865	ØØF1	1863	Ø1E1	1869	Ø2D1	1868
Ø3C1	1869	Ø4B1	1869	[illegible]	[illegible]		
			[illegible]	7261	1978	7351	197Ø
7441	1964	7531	1961	7592	1961	FFFF	FFFF

Figure 4 Example raw data file

```
M.T. LAGGED R.T. BY      -18 MINS
TIME SCALE FACTOR =   0.999
TIME CORRECTION APPLIED
ERROR IN LEVEL CHANGE =    3 MM
% ERROR IN LEVEL CHNG =   -3.2 %
LEVEL CORRECTION APPLIED
36   180981 1805   GL   4.933
36   180981 2204   GL   4.935
36   190981 0204   GL   4.929
36   190981 0604   GL   4.930
36   190981 1004
                   GL   4.837
36   091081 1347   GL   4.840
36   091081 1524   GL   4.840
```

Figure 5 Example processed data file

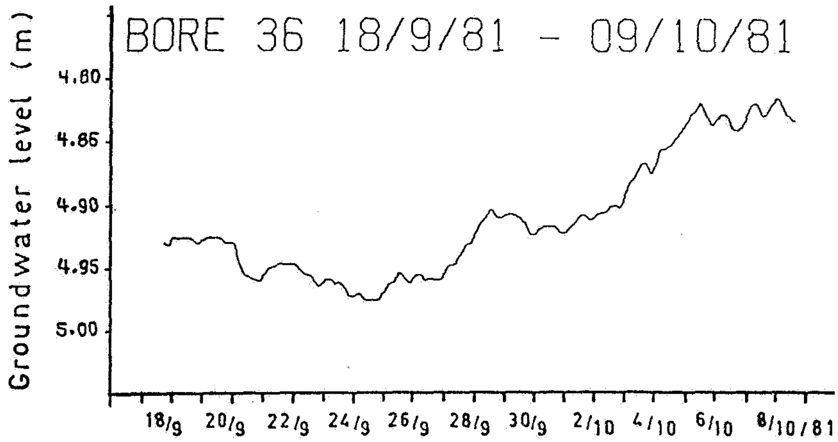

Figure 6 Example calcomp plot

6 Summary

The system described in this paper has proven reliable and accurate and was economically viable for a field study of groundwater recharge. Interested instrument manufacturers and users of hydrologic data aquisition systems are invited to correspond with the author.

Acknowledgements

The author expresses his appreciation to Mr. S. Woithe who undertook most of the design, development, construction and testing of the recording system. The support of the Director-General and Engineer-in-Chief of the Engineering and Water Supply Department of South Australia which funded the field study is gratefully acknowledged.

Reference

Brown, J.A.H. 1980. A review of the design of water resources data networks. Australian Water Resources Council Technical Paper No. 52, 372 pp.

GROUND-WATER MONITORING SYSTEM

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Abstract

The programs of the U.S. Geological Survey (USGS) require instrumentation for collecting ground-water-level data continuously and unattended for long periods. At present, no single cost-beneficial system exists to control the data collecting and recording processes for monitoring ground-water levels on a long-term basis, or for short- to long-term aquifer tests, where observations are needed at very short time (few seconds) intervals. The Geological Survey is developing a Ground-Water Monitoring System (GWMS), to meet these specific needs. The GWMS is a system of complementary instruments (a shaft encoder and pressure transducer for monitoring water levels, a data recorder, and a data retriever) arranged to fit these ground-water data-collection needs. The system recorder will incorporate programmable microprocessor features with solid-state memory, and be capable of operation with an optical incremental shaft encoder or pressure transducer. GWMS will be flexible, in that size and programming alterations may be made easily with data need or technology changes.

1 Introduction

The U.S. Geological Survey (USGS) is the principal Federal organization responsible for providing water resources information. Through a system of interrelated data-collection programs, measurements of ground-water are made at remote field locations throughout the nation. These data are entered into a computerized data base system for analysis, storage and dissemination.

Specific programs of the USGS require instrumentation for collecting ground-water-level data continuously and unattended for long periods, as well as for specialized needs such as aquifer tests. At present, no single cost-beneficial system exists to control the data collecting and recording processes for monitoring ground-water levels either on a long-term basis, or for aquifer tests where observations are needed at very short time (few seconds) intervals.

The Geological Survey is developing a Ground-Water Monitoring System (GWMS), to meet the specific ground-water needs of the Survey. GWMS is a system of complementary instruments arranged to fit the ground-water data-collection needs. The system recorder will incorporate programmable microprocessor features with solid-state memory, and be capable of operation with an optical incremental shaft encoder or pressure transducer. GWMS will be flexible, in that size and programming alterations may be made easily for specific data needs, and as technology changes.

2 System components

The GWMS consists of three main components as illustrated in Figure 1. These components are the water-level sensor (either an optical incremental shaft encoder or pressure transducer), the recorder and the retriever.

2.1 Water-level sensor

2.1.1 *Incremental Shaft Encoder*

The incremental shaft encoder monitors water level with a small diameter float that is connected to a counterweight via a tape that runs over the wheel of the shaft encoder as shown in Figure 1. The shaft of the encoder is supported by two ball bearings and has very low starting and running torque specifications. This makes the encoder suitable for ground-water measurement because the small-diameter floats used in ground-water wells provide very little starting torque. The encoder is designed to monitor

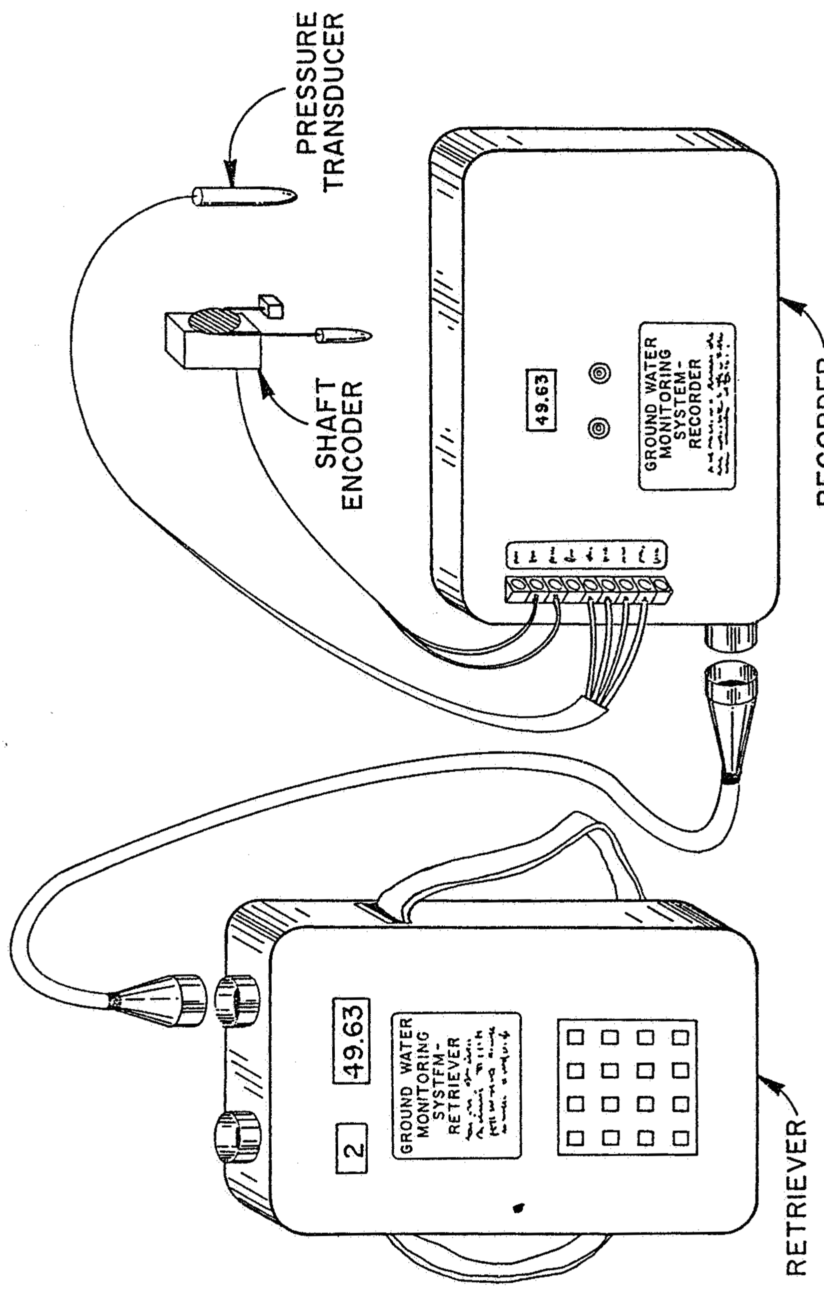

Figure 1 Groundwater monitoring system - Conceptual design

ground-water levels to 0.01 ft resolution. The shaft encoder uses two light-emitting diode (LED) optical interrupter modules and a slotted wheel to monitor ground-water level. As the wheel turns in response to water level changes, the slots are detected by the light beams in the interrupter modules. This information is sent to the microprocessor in the recorder which counts the slots and keeps track of the wheel position. The encoder has two light beams phased at 90 degrees with respect to the wheel slots. Two light beams are necessary so that the microprocessor can sense which direction the wheel is moving. There are 25 slots around the circumference of the wheel; however, since the microprocessor senses both the spaces and the lobes with two light-beam signals, it can detect 100 counts per revolution. Since the circumference of the wheel is one foot this translates to a resolution of 0.01 ft.

2.1.2 *Pressure Transducer*

Pressure transducers that sense water level changes through pressure are currently being evaluated for the GWMS. The precision of measurements using pressure transducers depend upon the range of water level to be monitored and the precision of the pressure transducer. The ranges in water level chosen will be selectable from 35, 70, 115 and 230 feet. Transducers chosen for this system must have an accuracy between 0.025 and 0.1 percent of full scale. The design goal is to hold the probe diameter to less than 3/4 inch to assure its use in small diameter wells. This is a difficult goal because commercial transducers are larger in diameter than 3/4 inch. The transducer will be a differential type, equipped with an air-vent tube in the electrical cable that runs from the transducer to the land surface. This feature will eliminate the need for corrections due to changes in atmospheric pressure. Pressure transducers have several types of voltage, amperes and frequency outputs. The recorder is designed to interface with most of them.

2.2 Recorder

2.2.1 *Operation*

The recorder's use is two-fold: (1) monitor water levels on a long-term basis (observation or network wells) or (2) monitor aquifer-test water levels. It is anticipated that most units will be configured for the first application.

The recorder will have two switches, and a 6-digit display that will allow an observer to view both present and past values of time and data in the recorder. The functions of the switches and display will be software programmable and thus can be changed to suit the user's needs.

a. Water-Level Monitor

   This version will be programmable to record/monitor water levels throughout a frequency range of once every twenty-four hours to once every hour.

b. Aquifer-Test Monitor

   This version will be programmable to record water levels in response to drawdown and recovery dynamics during pump tests. The program for an aquifer test will permit a recording of twenty-four-hour antecedent conditions; drawdown and recovery water levels, with initial readings in each phase as frequent as five seconds; and will span a total test period of seven days. The aquifer-test program will be pre-programmed into the recorder's memory.

2.2.2 *Configuration*

The recorder has four printed circuit cards. The first card will contain a microprocessor, a six-digit Light Emitting Diode display, 5-volt serial input/output (I/O) port, program memory and randomaccess memory (RAM). The second card contains the memory (4K byte capacity) for data storage. It will use a nonvolatile Electrically-Erasable Read Only Memory (EE PROM) to prevent data from being lost in the event of a power failure, and contain the voltage power supply to operate this type of memory.
The third card is a power-supply card. This card accepts twelve volts from an external battery and converts it to a five-volt signal for operation of the other cards. A highly efficient switching regulator, although more expensive than other types, will be used, but will double the battery life over the use of conventional regulators.

The fourth card is the signal conditioner. This card accepts the input from the probe(s) and converts it to a signal that the microprocessor will accept. Several types of this card will be available.

2.3 Retriever

2.3.1 *Operation*

The retriever's functions are to program the recorder, and read, extract and transport the data from the recorder to a computer or other data handling devices. The user will enter the function he desires through the keyboard, selecting that function from an instruction table. Once the function is selected, the retriever will direct the user through a sequence of instructions by automatically advancing to the next instruction when the preceeding one has been completed. These instructions prompt the user for necessary information required by the recorder to perform the selected function. Such information includes data rates, time, etc. The retriever can detect some input errors that will be flagged by sounding a buzzer and displaying an error code number. The instruction table can then be consulted to let the user know the reason of the error.

The retriever will have a RS232C serial I/O port (see Figure 2) which will allow the retriever to be attached to many different data-handling devices. These devices could be a mini or microcomputer, a phone modem, a RS232C cassette tape recorder, or a computer terminal. However, some devices may require special software in order to format the data in an acceptable way. Software for the most common I/O devices will be provided in the minimum configuration. The retriever will have enough memory to extract and transport data from several recorders.

2.3.2 *Configuration*

The retriever in its minimal form will contain five circuits cards. The first card has a microprocessor, five-volt serial I/O port, program memory and RAM memory. This card will control all of the functions of the retriever.

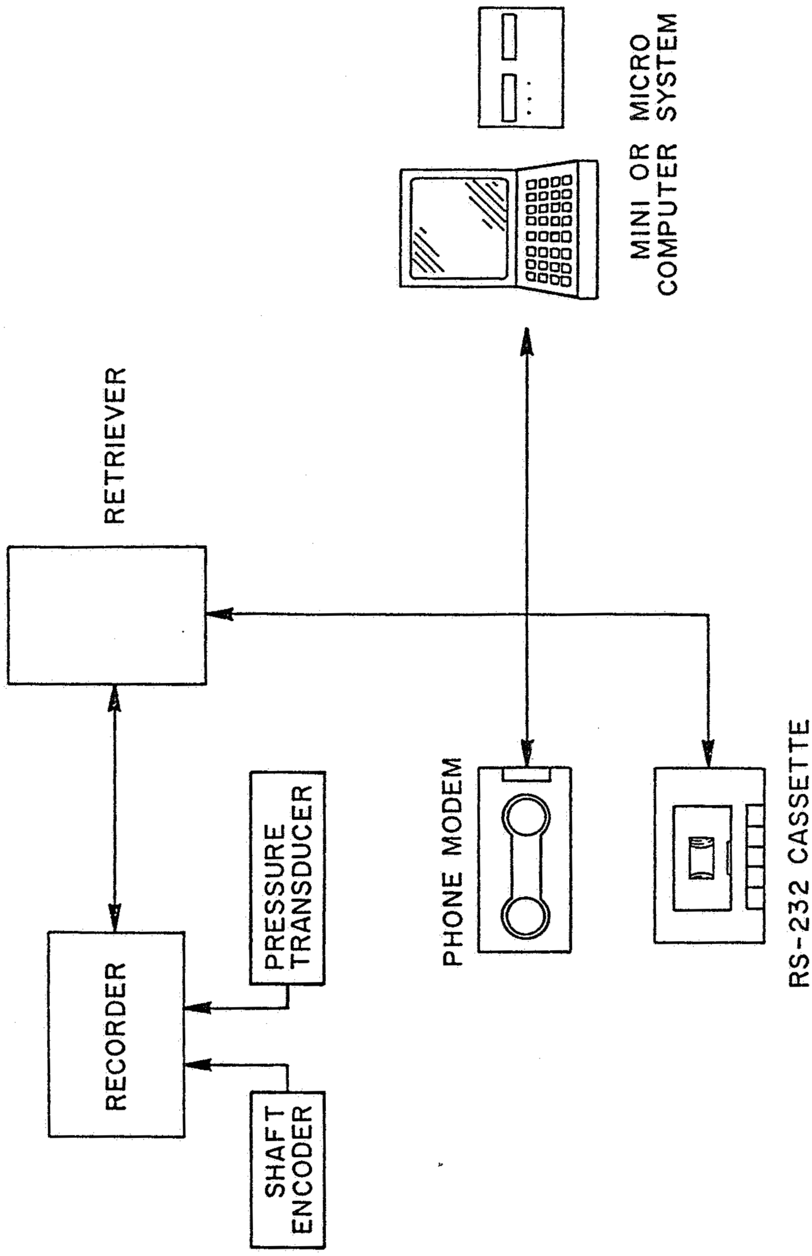

Figure 2 Groundwater-monitoring system and data-handling options

The second card contains the memory to store the data from the recorder. This card will be similar to the one in the recorder except that it will have a capacity of 16k bytes instead of 4k bytes used in the recorder. The third card is a power-supply card. This card will convert the twelve-volt battery power to 5 volts, which is necessary to power the other cards.

The fourth card in the retriever will have a 16-key keyboard interface, a RS232C interface, and a buzzer. This card contains the I/O functions particular to the retriever. Use of this card will allow use of the recorder's microprocessor board in the retriever.

The fifth card in the retriever contains two Liquid Crystal Displays (LCD's), a 4-digit instruction display and an 8-digit data display.

3 Summary

The GWMS is being designed to be a low-cost and flexible ground-water monitoring system. Alterations in programming may be made easily, practically, and as technology changes. Additional memory may be required as data-collection responsibilities and field-investigational areas expand. Additional sensor inputs and satellite-telemetry capability are possible future options for modification of the system.

COMBINED WATER SAMPLING AND HYDRAULIC TESTING IN POORLY PERMEABLE ROCKS

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Abstract

A wireline double-packer system has been developed which can measure the hydraulic conductivity, specific storage and hydraulic pressure within a well-defined zone in a borehole. The system incorporates the ability to sample groundwater from this zone. Using the system, which has capabilities down to about 1000m, uncontaminated groundwater samples have been collected from depths of 300m in a fractured crystalline rock. As an example one sample obtained was dead to carbon-14 with near-zero tritium. The system also allows the collection of samples for analysis of dissolved gasses.

The system includes a pump line from the test zone which bifurcates immediately above a pneumatically actuated shut-in valve usually situated about 10m below the borehole water level. With the valve closed water can be flushed from the near-surface tubes using gas pressure. Once empty the shut-in valve is opened leading to an instantaneous change of head in the test zone. Thus as water is derived from the test zone a slug test is performed which incorporates a knowledge of the hydraulic properties and the equilibrium pressure in the zone. A profile of these properties versus depth identifies where groundwater is entering or leaving the borehole and thus the most suitable zones for geochemical sampling. For chemical sampling purposes the slug test procedure is repeated until constant field chemical parameters are recorded:- usually 3-5 inter-packer volumes.

Résumé

Un système à câble avec doubles packers a été développé qui peut mesurer la conductivité et la pression hydrauliques ainsi que le stockage spécifique d'une zone définie dans un forage. Le système a la capacité de prendre des échantillons d'eau dans cette zone. Le système, qui fonctionne jusqu'à un profondeur d'environ 1000m, a été utilisé pour la prise d'échantillons d'eau souterraine sans contamination à un profondeur de 300m dans une roche cristalline fracturée. Pour donner un exemple, un échantillon montrait un niveau de ^{14}C en dessous des limites de détection avec presque aucun ^{3}H. Le système permet également la prise d'échantillons pour l'analyse de gaz dissous. Le système comprend une pompe, sous le forme d'un tuyau qui monte de la zone sous examen puis bifurque immédiatement au-dessus d'un soupape pneumatique, qui se situe normalement 10m environ en-dessous de la surface d'eau dans le forage. A soupape fermé, l'eau auprès de la surface peut être expulsée en introduisant du gaz comprimé. Une fois vidée le soupape est réouverte, ce qui mène à un changement instantané de la pression dans la zone sous examen. Ceci veut dire que lors de la prise d'eau de la zone un 'slug test' est exécuté, ce qui donne des informations sur les caractéristiques hydrauliques et la pression à équilibre dans cette zone. En traçant les variations de ces caractéristiques selon le profondeur on peut identifier l'endroit où l'eau entre et sort du forage et, de là, les endroits qui conviennent le mieux pour une prise d'échantillons géochimique. Pour la prise d'échantillons chimique le procédé 'slug-test' est répété jusqu'au constat de paramètres chimiques stables:- normalement 3-5 fois le volume entre les packers.

1 Introduction

A double packer, wire line testing system has been developed, as part of a research programme into groundwater movement through poorly permeable media, to measure hydraulic conductivity (K), specific storage (S_s) and environmental pressure (ΔH) in boreholes. K and S_s give the rate at which groundwater may be abstracted whilst ΔH indicates where groundwater enters into and exits from a borehole. This system has been modified to allow controlled abstraction of samples for geochemical analysis and is suitable even for collection of dissolved gases.

2 Hydraulic testing

The testing system, described in Holmes (1981), has been slightly altered as shown in Figure 1 and now acts as a gas pump.

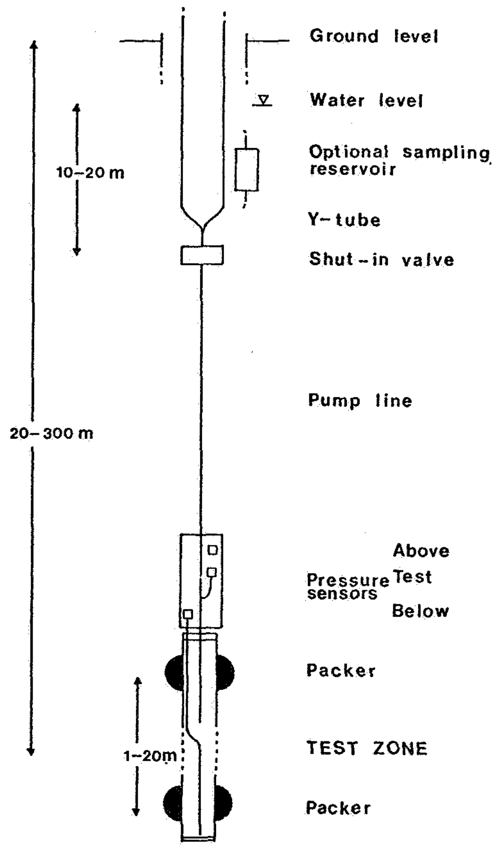

Figure 1 Diagram of the wire line straddle packer system

A specific groundwater zone, isolated by inflating hydraulic packers, is connected to the surface by a flexible pump line. This bifurcates about 10-15m below water level, immediately above a pneumatically actuated shut-in-valve (SIV). This is closed to isolate the pump line from the gas pump above. This gas pump comprises two tubes, one of which opens into a larger diameter steel tube, connected to a compressed nitrogen gas supply and venting valve. The entire assembly is suspended in the borehole on commercially available logging cable which also transmits information from down-hole pressure sensors to the surface.

The procedure to abstract groundwater is as follows. After successful emplacement of the packers, SIV is closed, isolating the pump line from the gas pump. Nitrogen gas, at a pre-selected pressure, is introduced to the input-tube above SIV causing water to exit from the other tube. After total water removal, the gas flow is stopped and both tubes are vented to atmospheric pressure. This created head change is transmitted to the isolated zone when SIV is reopened. Groundwater flows into the zone and up the pump line in order to equalise the enforced pressure change. The cycle of isolation and gas pump operation is repeated until a representative groundwater sample is obtained.

Down-hole sensors measure above, between and below packer pressures enabling the movement of groundwater around and into the isolated zone to be carefully monitored. Above and below the packers, pressures may fall during abstraction indicating leakage around the packers. If such falls are large and occur rapidly after a drop in between-packer pressure, serious leakage is indicated and a new zone should be selected. A slow and consistent fall indicates that leakage is present through the rock mass which may be tolerated. Between-packer pressures follow a regular pattern as groundwater is abstracted (Figure 2).

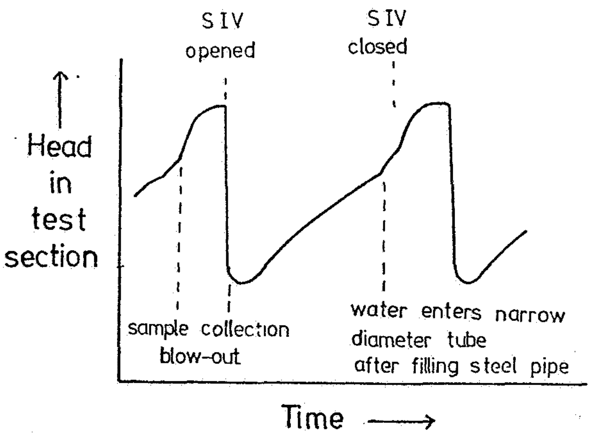

Figure 2 Between-packer pressure sensor recorder trace for a slug test

Any deviation from this indicates equipment malfunction. The trace can be analysed and the whole cycle represents a repeatable slug test. The variant of recovery with SIV reclosed immediately after opening constitutes a pulse test. This can be analysed for hydraulic data in lower permeability zones in much shorter times than are necessary for the large volume flow into the whole system in the slug procedure. Sampling zones are selected from borehole profiles of K and ΔH. Zones having positive environmental pressure and $K > 10^{-8} m\ s^{-1}$ are normally chosen to keep sampling times reasonable. As an example a borehole into Moine metasediments with granitic sheets, at Altnabreac, Caithness Scotland, produced the profiles illustrated in Figures 3 and 4.

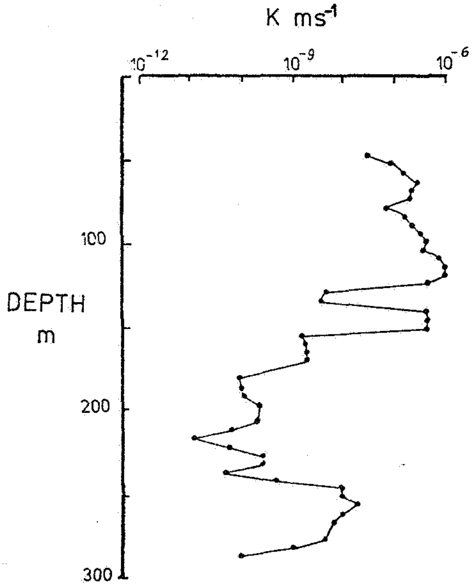

Figure 3 Hydraulic conductivity (K) profile of borehole ALB

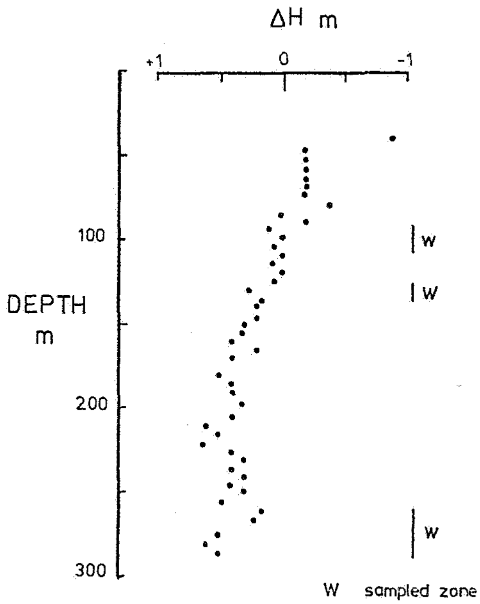

Figure 4 Environmental pressure difference (ΔH) profile of borehole ALB

From Figure 3 it can be seen that hydraulic conductivities range from about 10^{-6}m s^{-1} to 10^{-11}m s^{-1}. Figure 4 shows that the environmental pressures are negative above about 90m depth and positive below. They are generally about +0.5m below 150m. This means that in the open borehole water tends to flow into the bottom and out near the top. Superimposition of these two diagrams identifies suitable zones for chemical sampling. The three zones identified in Figure 4 are those from which chemical examples are given below.

3 Chemical sampling

During abstraction it is usual to measure the temperature, electrical conductivity, pH, Eh and selected specific ions of the fluid as it arrives at the surface. These values, together with abstracted volume and sampling times, can be conveniently noted on the pressure sensor recording trace. When the above values reach equilibrium, samples can be collected for any chemical, gas or isotopic analysis which is required. The rate of water abstraction can be controlled by the gas pressure applied from surface with SIV closed.

Two criticisms can be made of such a simple system as regards sampling groundwater, especially for gases. Firstly there is direct contact between the purging gas and the sample and secondly groundwater is held in the system at lower than its environmental pressure, during which time degassing may occur. Compressed nitrogen gas is used to eject the water sample. If a long steel tube is used, as in this system, then the gas/water interface is minimized. Evidence is given later that if the first and last litre of sample is not collected the remaining fluid is not contaminated, nor, for example, is there loss of radon.

As groundwater flows through the equipment it experiences changes in both environmental pressure and temperature which could cause gases to exsolve from the water. The gas volume lost might be expected to be dependent upon the time within the equipment between leaving the sampling zone and collection at the surface. This residence time depends on:

a) the volume of water in the pump line compared with that which is removed in each gas pump cycle.

b) the time between each gas pump cycle. This is controlled by the rate at which groundwater enters the packer isolated section and is related to the hydraulic conductivity and specific storage of the rock.
As an example, the equipment used at Altnabreac ejected 6 litres of groundwater for each gas pump cycle. The pump line (7mm ID Nylon II tubing) has a volume of 11 litres when sampling from 300m below ground level. Thus two gas pump cycles were required to bring water from this depth to the surface. A sample zone hydraulic conductivity of approximately 1×10^{-6} m s^{-1} allowed gas pump cycle lengths of about 5 minutes. An experimental procedure was performed which showed that degassing, on this time scale, was not a problem.

3.1 Sampling for dissolved gases

When well-head conditions had stabilised, samples were taken from a particular zone after varying transit times to surface. Results are shown in Table 1.

Table 1 Dissolved gas data from borehole ALB, zone 94-8 - 106m (in Moine metasediments). The interpacker volume was approximately 105 l. The zone was continuously isolated throughout collection of the cumulated volumes (vol.)

No.	Vol. l	Method	Date	Time	He	Ne	Ar	Rn pCikg^{-1}	T T U
					cc STP/cc H_2O $\times10^{-8}$	$\times10^{-7}$	$\times10^{-4}$		
1	72	SIV	9.4.81	2011	201	2.61	4.93	5397	7.1
								4864	
2	90	SIV	10.4.81	1020	207	2.62	4.95	4809	
								4543	
3	174	SIV	10.4.81	1240	165	2.66	4.95	3414	0.6
								4019	

After nearly one inter-packer volume (i.p.v.) air was used to flush samples for He and inert gases (collected in 1cc glass tubes between two taps) as well as for radon (1 litre bottles). Fourteen hours later, during which time SIV was left open, the first flush was discarded and the second volume was sampled using nitrogen. This had therefore been resident between the packer zone and the surface for 14 hours. It can be seen from Table 1 that the He, Ne, and Ar values obtained are almost identical. Ne would be a sensitive indicator of any air contamination, which clearly has not occurred for samples 1 and 2. It should be noted that samples are routinely taken after discarding the first and last litre. As a further precaution, N_2 has always been used subsequent to this first extraction.

Radon contents of these two samples lie on a decreasing trend when compared with the later abstracted sample 3 (Table 1). This is attributed to the near-field of the matrix rock having a larger specific surface area (from which radon can be recoiled) and/or greater rock uranium contents than the further field involved when larger volumes are abstracted. The lower He content for sample 3 supports this view.

Data for a zone where Rn was seen to increase are listed in Table 2, and the Rn contents are plotted against the abstracted volumes in Figure 5.

Table 2 Field chemical parameters and radon contents from borehole ALB, zone 128.5 - 132.7m in Moine metasediments. The inter-packer volume was approximately 40 l. The zone was continuously isolated throughout collection of the cumulated volumes (vol.)

No.	Vol. l	Method	Date	Time	pH	Eh mV	K μScm^{-1}	Rn $pCikg^{-1}$
	30	SIV	15.7.82	1500	7.1	+215	522	-
4	115	syphon	15.7.82	1840	7.0	+255	520	8566
								8999
5	240	syphon	16.7.82	1000	7.05	+240	507	9980
								9350
6	710	syphon	17.7.82	1050	-	-	-	9920
								10077
7	720	SIV	17.7.82	1158	7.05	+215	508	9034
								10446

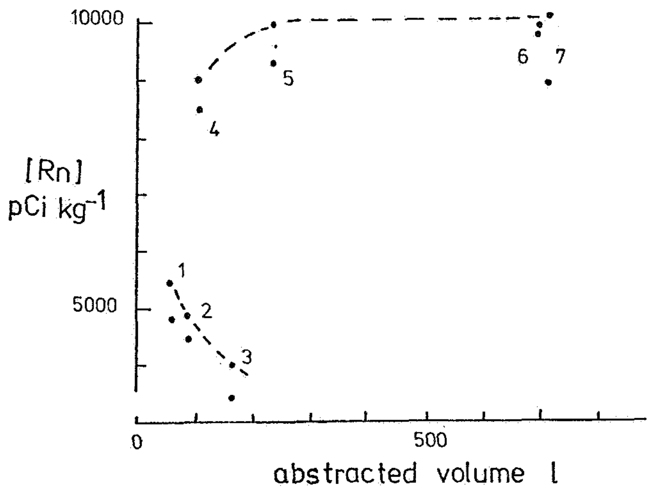

Figure 5 Radon content plotted against volume abstracted for two zones in borehole ALB. Sample numbers are indicated (Tables 1 and 2)

In this zone a smaller inter-packer volume (~ 40 l with K 1.5×10^{-6}m s^{-1}) allowed sampling to continue until 18 i.p.v. had been abstracted. Field conditions remained very stable. Inert gas data from this sample suite are not yet available. A large positive ΔH (+0.5m) combined with a convenient relief difference between the casing top and a nearby stream (~ 3m) made it possible to operate a syphon and thus compare SIV sampling and continuous abstraction wherein there was no gas interface (c.f. samples 6 and 7, Figure 5).

The differences between replicates are believed to be due to diffusive losses during transport to the laboratory and from Rn loss in transfer of the sample into the extraction line. The maximum value is thus given greater weight in Figure 5. The similarity in Rn contents of samples 5, 6 and 7 (Figure 5), collected by syphoning and SIV, shows that the SIV method does not affect dissolved gas contents.

The proof that both technical and zone specific difficulties can sometimes be overcome lies in the chemical data obtained from the bottom zone in the same Altnabreac borehole (259-281m into granitic sheets within the Moine). Here a very low tritium content (4 ± 4 T.U.) accompanied a <13% modern carbon ^{14}C determination. The age derived, >11,000 years, is the present detection limit for the small counter system employed (Otlet et al., 1982). Its He content (~ 300 x 10^{-8}cc STP/cc H_2O) was used to calculate an age. With radioelement contents of the rock matrix inferred from core material, this is ~ 3 x 10^4 years. This zone is believed to yield such good samples because it was exposed for the shortest times to high heads during drilling and subsequent operations. Invasion has been further reduced by the natural flushing at the bottom of this borehole. Elsewhere very large volumes may have to be abstracted to obtain representative samples from boreholes in poorly permeable media. This of course means drawing upon a large storage and may pose the problem that the water environments are very different from those represented by borehole core material.

Acknowledgements

The authors wish to thank colleagues of the IGS for their patience and diligence during testing and sampling programmes. Gas analyses were performed by the University of Bath under the direction of Dr J.N. Andrews and tritium and ^{14}C were determined by the Nuclear Physics Division U.K.A.E.A., Harwell.

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References

Holmes, D.C., 1981. Hydraulic testing of deep boreholes at Altnabreac: development of the testing system and initial results. Rept. Inst. Geol. Sci. ENPU 81-4.

Otlet, R.N., Huxtable, G., Evans, G.V., Humphreys, D.G., Short, T.D. and Conchie, S.J., 1982. Development and application of the Harwell small counter facility for the measurement of ^{14}C in very small samples. Procs. 11th Int. Radiocarbon Conf. Seattle, U.S.A. (in press).

A MARINE GEOPHYSICAL UNIT FOR STUDYING SUBMARINE GROUNDWATER DISCHARGE

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Abstract

The paper considers the methods and techniques used for the investigation of the submarine discharge of groundwater in the Caucasus shelf of the Black Sea.

1 Introduction

The study of the groundwater discharge into the seas and large lakes is essential in the general investigation of their salinity, temperature and water regimes.
Two basic methods can be used for this study. The first one is the method of extrapolation. Data obtained during the onshore hydrogeological survey are extended to the sea region. This method is very convenient when applied to the sea research because drilling of wells, test pumping, regime observations and other traditional observations of the hydrogeological survey are very expensive and labour-consuming under sea conditions. Application of this method is restricted, however, to the regions where onshore hydrogeological data is reliable and sufficiently completed.
The second method is a set of hydrogeological operations proper, in the open sea. In general, it is impossible to directly observe and measure groundwater manifestations on the sea bottom, in contrast to the onshore analysis of springs and hydrogeological wells. That is why the sea hydrogeological survey is based on the

analysis of various anomalies in physical and chemical fields generated as the groundwater passes through the sea bottom (Jamalov, Zektzer, Meskheteli, 1977).

The present techniques used in the sea hydrogeological survey allow to measure with great accuracy basic hydrogeological parameters, such as temperature, heat flow, electric conductivity, natural electromagnetic and electric fields, and also to define microcomponent, chemical, gaseous and isotopic composition of the porous solutions in the bottom deposits and in the sea water. Anomaly in the distribution of each parameter is, by no means, a direct indicator of the groundwater seepage through the sea bottom. The scope of information significantly increases when we measure simultaneously at one site the parameters of several physical and chemical fields or various components of one and the same field. Informativity also increases with the increase in the frequency of measurements, which, in an ideal case, can form a continuous profile. A correct choice of the optimal set of parameters is also essential in the sea hydrogeological survey. The set of parameters should carry the necessary information about the submarine discharge of the groundwater. The scale of the sea hydrogeological survey, number of sites, frequency of profiling and the optimal set of applied methods depend on the stage of the research, techniques and concrete aims of the research, and also on the conditions of the sea bottom in the region under investigation. To provide for the adequate interpretation of the anomalous physical and chemical parameters it is necessary to know the geological and tectonical structure of bottom sediment deposits. This structure can be obtained by the seismoacoustic profiling.

2 Methods and techniques in the investigation of submarine groundwater discharge

The set of instruments for the sea hydrological survey comprises a seismoacoustic profilograph, a probe transported along the bottom, and a recorder on board the ship. Temperature, electric conductivity and pressure sensing devices are installed in the probe.

Results of the complex profiling in the combination with the geochemical and isotopic testing of the porous solutions and near-bottom water at separate sites, allow to perform the sea hydrogeological survey and to map with great confidence submarine groundwater discharge. Seismoacoustic profilograph provides for the geological and tectonic data and, in some cases, data on the facies composition of the structure of bottom deposits for the first hundreds of meters.

The apparatus has the following algorithms of the operations.

Some energy is accumulated in the high-voltage condensers. Then, it is transmitted to the source of excitement by a commutator, monitored by a synchronization system. An acoustic wave originates in the results of the electric discharge in the source of excitement. This wave reflects from geological strata with various coefficient of reflection and dispersion, and is accepted by the piezoelectric detector. After the signal is intensified, it is transmitted to the recorder. At present, we know several methods to generate oscillations in water medium for seismoacoustic profiling. In our case, we used the group source of excitement with a uniform trace of energy supply to the elementary sources. This source is simple in production and has a high resolving power.

The excitation block comprises a high-voltage rectifier and commutation system which permits to discharge the condensers at definite moments. Accumulation capacity of the excitation block is 10mf, voltage is 10kw, total energy of the sources is 500 J. The practice verified that this energy is sufficient to obtain the reliable information about the geological structure of the bottom deposits to the depth of 200 m.

The optimal towing rate for the seismoacoustic profiling is 4-6 km per hour. This rate provides for the best correlation between the time of the profiling, the quality of seismograms and the possibility to avoid the detachment of the seismic detector.

The next component of the sea hydrogeological set of instruments is a bottom-towing probe which carries the detectors of electric conductivity, temperature and pressure. The probe should secured against detachment and its detectors should be protected from mechanical damaging, obstruction with muds and against corrosion. To meet all these requests the probe was produced of bronze. It has the shape of the ellipse, and all detectors are mounted in its tail-end. Curved metallic rods were used to protect the detectors. Fastening of the probe to 10 mm steel cable by four 10 mm bolts and special hermetic junctions prevent it from detachment.

During the sea hydrogeologiacl survey in the shelf area the depth, as a rule, does not exceed 500 m and we can use for hermetization of the probe the tephlone laying and hermetic paste. This method of hermetization was tested down to 2000 m and was a success.

Four-electrode measuring cell was used as a detector of the electric conductivity of water. In some cases, under specific conditions on the sea bottom we used a two-electrode cell (Grilikhes, Filanovsky, 1980). Thermistor, placed in a metallic protection box was used as a temperature detector. The properties of the thermistor and geometry of the box allowed to obtain the time constant of the detector which is about a tenth of a second, thus it meets the request of the research with a towing instrument. To control the depth of the towing we used the standard detectors of pressure which provided for the relation:

$$R_p = f(P_0)$$

where

R_p = resistance of the rheostat in the detector

P_0 = pressure at the given depth

Proper mounting of the probe and stabilizing loads are of great importance.

Now, we can consider the behaviour of the probe during its towing when the mounted load is placed in various positions and the length of the steel cable is three times greater than the sea depth in situ. The probe weights 20 kg, the frontal load weights 20 kg, the stabilizing - 10 kg. If we place the loads in one place with the probe, the probe will move with jumping, that is with periodic uplift from the bottom. To avoid this type of the motion, the frontal load is advanced at 25-30 m from the instrument along the steel cable. In this case, the probe will zig-zag over the horizontal plane and it will affects the quality of the measurements and recording. To remove these horizontal oscillations, the stabilizing load is mounted in the tail-end of the probe, at the distance of 3-5 m.

When towing, the probe can be anchored to the sea bottom due to its roughness and can be detached and lost. That is why the instrument should be rather simple in production and low in price. It is expedient to mount only sensible elements to the probe, and to process the signals, transmitted by the steel cable, on board the research vessel.

The towing rate, taken for the seismoacoustic profiling, is suitable for the measurements of temperature and electric conductivity, because the time constant of the detectors permits to recognize the change in temperature and electric conductivity from site to site, removed from each other at 2 m. The greater rate of the towing reduces the accuracy in these measurements and can cause uplift of the probe from the bottom. So, the rate of 4-6 km per hour is optimal for the simultaneous seismoacoustic profiling, temperature and electric conductivity measurements - the factor basic for the success of the sea hydrogeological survey.

The electronic block, which receives and transfers the signals transmitted by the steels cable from the detectors is built with the usage of the precise operative amplifiers. Their parameters allow to reach high accuracy in the measurements of electric conductivity, temperature and pressure. The accuracy in the measurements of temperature and salinity during the towing is $0.05^{\circ}C$ and 0.1%, accordingly.

Before the commence of the profiling, the calibration test of the detectors was performed in the laboratory. The calibration curves were plotted in the result of the estiblished correlation between the voltage output and the electric conductivity, temperature and pressure. Measured parameters are defined by these curves.

Multi-channel recorders were used in the research. The run-rate and duration of recording was similar to the recorder of the seismoacoustic profilograph.

Another prospective way of data processing is data computering directly on board the research vessel. Computer processing of data does not require the intermediate operations between the stages "sensible elements - means of information recording".

This fact is very important, because even the high-precision integral microschemes introduce errors for measurements. The simple commutator provides for the successive input of signals from the detectors mounted at the probe in the analogue-digital transformator. After the analogue signal is coded and transformed, it is forwarded to the computer, where it is processed according to the given program.

The computer allows to use the plotter to map salinity and temperature in situ.

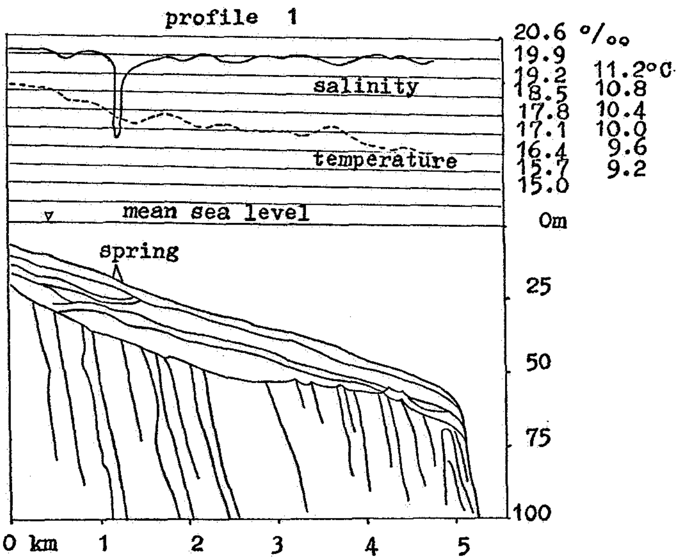
3 Results of the research

The described methods and techniques were verified near the Caucasus shore of the Black Sea. The polygon was set in the region of Gudauta basement uplift , manifested in the bottom topography as a bank composed of the Cretaceous and Palaeogene-Neogene sediments. The polygon is located on the sea continuation of Bzyb artesian basin opened towards the sea. Low Cretaceous and Upper Cretaceous-Palaeogene carbonate karstified rocks are main aquifers. The groundwater discharge goes by the ascending springs along the shore and on the sea bottom. In some springs the debit reaches several hundreds of cubic meters per second (Meskheteli, 1980).

Two complex profiles recorded during the sea expedition (Gudauta bank near the town of Novy Afon, March, 1981) provide clear illustration (Figure 1). The distance between the profiles is 2500 m. After the data on electric conductivity was transferred into salinity data, and temperature and salinity data were verified by sites, we plotted the distribution curves for these parameters and the geological cross-section obtained from the seismoacoustic profiling. Both profiles show distinct areas of salinity decrease at 1500 m off the shore. Curves of temperature distribution along the profiles have no sharp changes and temperature gradually decreases from 10.9-11.0^{o}C to 9.5-8.0^{o}C, as the depth increases off the shore.

Profile 1 shows the sharp fall of salinity to 16.5% and, then, more gradual increase in salinity to its average on the profile, equal to 19.9 %.

The temperature curve on profile 1 has an insignificant lowering, shifted towards the open sea as compared with the minimal value of the salinity curve. The seismogram shows coincidence of the anomalous salinity and temperature values and the concentrated karst spring recorded by the seismoacoustic profilograph. The mouth of the concentrated karst spring is located at the depth of 19 m and is attributed to the wedging of the pack of the layers covered by thin Recent sea



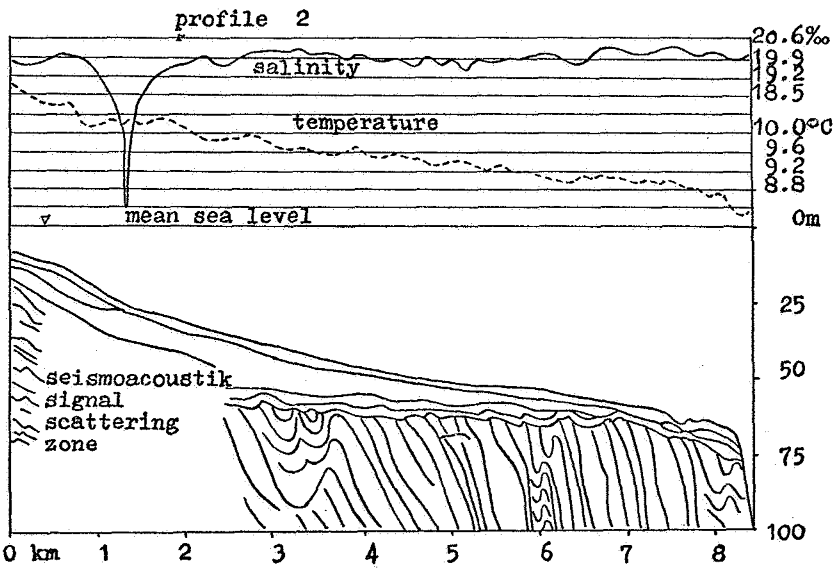


Figure 1 Complex frofiles of marine hydrogeological survey

deposits composed of sand and sandy muds. On the seismogram the area of concentrated karst spring is a narrow cone with the height of 3 m from the bottom. The reflection of the signal is possible because of the great density of the stream in the submarine karst spring which has greater pressure as compared with the surrounding sea water. The mouth of the area of karst spring concentration is not recorded by the seismoacoustic profilograph, as the diameter of the mouth seems to be less than its resolving power. Insignificant shift of the anomalous area on the temperature curve relative to the position of the karst spring concentration can be explained by the southeastern stream observed in the moment of the recording.

Seismoacoustic profile 2 shows a distinct zone of the signal scattering. Its clear boundaries coincide with the boundaries of the salinity anomalous area. Such scattering of the signal can be explained by the ascending gas-saturated karstic groundwater.

We believe that the karstic groundwater which is under great pressure arised by the karst cave or tectonic discontinuity and is discharged through the filter of the porous Quaternary deposits.

Relatively small temperature anomalies on profile 1 and their absence on profile 2 can be explained by the fact that we performed our investigations in early March when the karstic and sea waters in the near shore region have almost the same temperature equal to 10-11°C.

4 Conclusions

Complex profiling, several examples of which we tried to describe in this report, is the basis for the sea hydrogeological survey. Performed according to the net, it allows, in combination with the hydrochemical and isotopic sampling at sites, to plot well-grounded maps of submarine discharge of groundwater, to define the type of submarine source and to detect the mechanism and volume of the submarine groundwater discharge.

References

Grilikhes, M.S. and B.K. Filanovsky, 1980. Contact conductometry. Khimiya, Leningrad (in Russian).

Jamalov, R.G., I.S. Zektzer and A.V. Meskheteli, 1977. Groundwater discharge into seas and the World ocean. Nauka, Moscow (in Russian).

Meskheteli, A.V., 1977. Submarine discharge of groundwater into the Black Sea: methods of its investigation. In: Otzenka i ratzionalnoe ispolzovanie resursov podzemnykh vod. Nauka, Moscow, 1980 (in Russian).

LABELLED SLUG TEST

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Abstract

The Labelled Slug Test is a flow log method using radioactive pulses and a gamma tool for determination of the flow velocity in a borehole. The method has been used to evaluate hydraulic conductivity of low - permeable formations. Equipment and procedure are described and discussed and comparison with results obtained from packer tests is made.

Resumé

Le "Labelled Slug Test" est une diagraphie de rayons gamma utilisant des traceurs radioactifs pour la détermination de la vitesse d'écoulement dans un trou de sondage. Cette méthode a été utilisée pour l'évaluation de la conductivité hydraulique de formations peu perméables.L'équipement et les procédés sont décrits et commentés. Les résultats obtenus par cette méthode sont comparés avec ceux obtenus au cours des essais packer.

1 Introduction

The knowledge of matrix and fissure permeability has great importance in connection with the investigation of groundwater utilization, waste disposal, geothermal energy, hot water storage, oil and gas production, etc.

If the investigated rock has a high fissure or matrix permeability, it is easy to measure the hydraulic conductivity of the rock, by means of conventional pumping or injection tests, impeller flow logs or a number of borehole logs (resistivity, neutron, sonic and caliper). In rocks with a very low hydraulic conductivity, the conventional pumping test will be time-consuming, the impeller flowmeter log can not be run due to a mechanical minimum speed of the impeller and the borehole logs do not give a direct determination of permeability.

The identification of fractures in low-permeable rocks by means of tracer tests has previously been studied by Marine (1979). The LST - test is a further development of this approach that will, at the same time, make it possible to calculate the distribution of the matrix permeability and the hydraulic conductivity of the fissures. Calculation of the hydraulic conductivity, based on a flow log type method, has been previously investigated by Schimscal (1981), Syms (1982) and Andersen et al. (1981).

2 The Labelled Slug Test method

During the LST, the distribution of water velocity in a pumped borehole is measured. This velocity depends mainly upon the hydraulic conductivities of the open section of the borehole, the applied drawdown and the variation in the cross-section of the borehole.

Assuming that the diameter of the investigated zone is constant, the inflow into the borehole within this zone, is expressed by the eq. (1).

$$Q_i = \pi \cdot r_w^2 \cdot (V_i - V_{i-1}) \tag{1}$$

where:

Q_i = the inflow from the interval l_i

r_w = the average radius in the interval

V_i and V_{i-1} = the fluid velocity at the end and at the beginning of the interval.

Assuming that the borehole may be divided into a number of intervals with homogeneous and isotropic hydraulic conditions, the drawdown, H, the flow rates, Q_i, the pumping time, t, the hydraulic conductivities, K_i, and the specific storage, S_s, are for every interval, l_i, related by eq. 2, Bear (1979):

$$H = \{Q_i/(4\cdot\pi\cdot K_i\cdot l_i)\}\cdot\ln\{2.25\cdot K_i\cdot t/(r_w^2\cdot S_s)\} \qquad (2)$$

The measured velocity profile can be related to the matrix permeability by combining eqs. (1) and (2):

$$H = \{r_w^2\cdot(V_i-V_{i-1})/(4\cdot K_i\cdot l_i)\}\cdot\ln\{2.25\cdot K_i\cdot t/(r_w^2\cdot S_s)\} \qquad (3)$$

With an estimation of the specific storage, S_s, it is possible to calculate the hydraulic conductivity from eq. 3.
The evaluation of the velocity (or inflow) profile in a borehole with a varying diameter, and procedures for the calculation of the matrix permeability and fissure conductivity are described by Gosk, G. (1983).

3 LST procedure

The LST-test consists of six steps:
1) A caliper log is run to determine the cross-section of the borehole.
2) A constant rate pumping or injection is carried out.
3) The radioactive tracer is placed in selected intervals.
4) A continuous logging of the time-dependent depth to the single tracer pulses is performed.
5) The distribution of the tracer pulse velocities is determined.
6) The distribution and type of hydraulic conductivity in the borehole is calculated.

The caliper data are used to eliminate the effect of a changing diameter on the pulse velocities. After this procedure, only the variation of the flow rate, which is related to the permeability, will influence the velocity of the pulses. The caliper tool should be equiped with a depth indicator of the same kind as that used with the LST-tool in order to ensure proper depth correlation for all logs.

Apart from naturally flowing wells, the LST-test has to be performed during injection or pumping. The choice of the method will depend on the depth to the water table, because pumping normally will not be possible if depths are greater than 200-300 meter. If possible, pumping will be preferred, because injection may cause mud invasion or hydraulic fracturing of the formation.
Pumping or injection should be started some hours before testing, to reduce the well storage effect and to give a preliminary estimation of the water velocity in the well. The recording of flow rates and drawdowns should be carried out in order to evaluate the transmissivity of the tested interval.

The LST-method requires a tracer that: (1) is easy to measure, (2) is perfectly soluble in the borehole water, (3) causes no change in the water behaviour nor (4) has any substantial tracer loss during the water movement. Only a radioactive tracer will fulfil these demands. If the tracer has to be used in a groundwater environment, it must have low radiotoxicity and a short half-life. Bromine-82 is a useful tracer that, in the form of sodium bromine, has a stable chemical bond, a low toxicity, a half-life of 36 hours and a gamma radiation energy of 0.556-1.478 MeV. The Bromine-82 is obtained by direct irradiation in a reactor and will therefore be available in most places. In order to secure high pulse recovery, injection of the radioactive tracer pulses should be done uphole, because the highest flow velocities will always be found in the upper part of the well.
Selection of the pulse interval should be done in such a way that every meter of the investigation well will be passed by at least one tracer during the test. A good preliminary setting of the pulses will be

obtained by injecting the tracers with a linear decreasing interval by depth. The life of the tracer will be limited by its radioactive decay, and it will therefore be important to save time by using a combination LST-tool with enough capacity to place additional pulses in "blank" intervals.

All available information from the well site (i.e. core tests, borehole logs, packer tests etc.), should be taken into account during design of the LST-test. This will improve testing results by placing pulses at the most suitable levels.

The evaluation of the water velocity distribution in the pumped well will be based on continuous measurement of the radioactive pulses by a gamma scintillation detector. The moving pulses will appear on the log record as discrete measurements of every single pulse. Due to this fact, it is of extreme importance to measure the single pulses with a time interval corresponding to the desired test resolution and to the actual pulse velocity.

The LST-method was designed to measure permeability in very low conductive rocks, which implies water velocities from about 60 m/hour down to 0.1 m/hour or even lower. It is therefore obvious that both depth and time identification have to be very accurate. Depth determination should be based on the mechanical depth indicator, log markers such as the casing collar locator and formation resistivity and formation gamma profiles. Time has to be recorded and displayed simultaneously with the pulse logging record.

If the investigated interval is long and the water velocity is high, it will not be possible to make continuous logging up and down through the entire interval without some loss of information in the high velocity intervals. For this reason, pulse velocities based on every two gamma logging runs have to be calculated during logging to optimize the further logging operations.

Every single pulse will have its own appearance, and the definition of pulse depth and time will be easy to establish based on well site hard copy. Calculation of real pulse velocity can be made on a desk calculator, when cross-section corrections, as described by Gosk, G. (1983), have been considered. Calculation of the hydraulic conductivities can be made with a simple desk calculator using the real velocity distribution and eq. 3., but because of the large amount of logging data and the considerable number of calculations, the use of a bigger computer is adviseable. For example, a 500 meter investigated interval, with 200 pulses, 36 hours logging operation and with a logging speed of 4 meter per minutes, will result in 3.500 pulse observations, consisting of three different parameters: pulse number, depth and time.

4 Field example

The results of a LST-test at Mors, Denmark, during January 1981, with a total pumping period of about 33 hours, are reported by Andersen et al. (1981). A 550 m deep well, Erslev no. 1S, which was drilled through Upper Cretaceous chalk and limestone, was used for the LST-test. The well, drilled in connection with the Danish Radioactive Waste Disposal Programme, was investigated by core inspection, a number of lithology/permeability borehole logs and pumping tests. The logging indicated numerous fractures in several intervals in the chalk/limestone formation, but pumping tests gave permeability values corresponding to permeability of the matrix determined on cores in the laboratory.
27 Bromine-82 pulses were released into the well in an interval from 550 m to 130 m below ground level (Fig. 1) and pumping was maintained by an ejector pump mounted 96 m below ground level. The injection and logging procedure was carried out with single parameter tools, which means that depth determination was based only on the mechanical depth indicator and cable marks.
The levels of the single tracer pulses were defined either by the pulse maximum or by an abrupt radiation change below or above the pulse maximum. The positions of the tracer pulses measured by 14 gamma logs during the 33 hours of pumping are illustrated in Fig. 1. The pulse velocities were corrected for variations in the cross-section of the borehole and these velocities were calibrated to a theoretical borehole diameter of 9 inches. A comparison of the measured and calibrated pulse velocity profiles shows that the intervals with an apparent upward

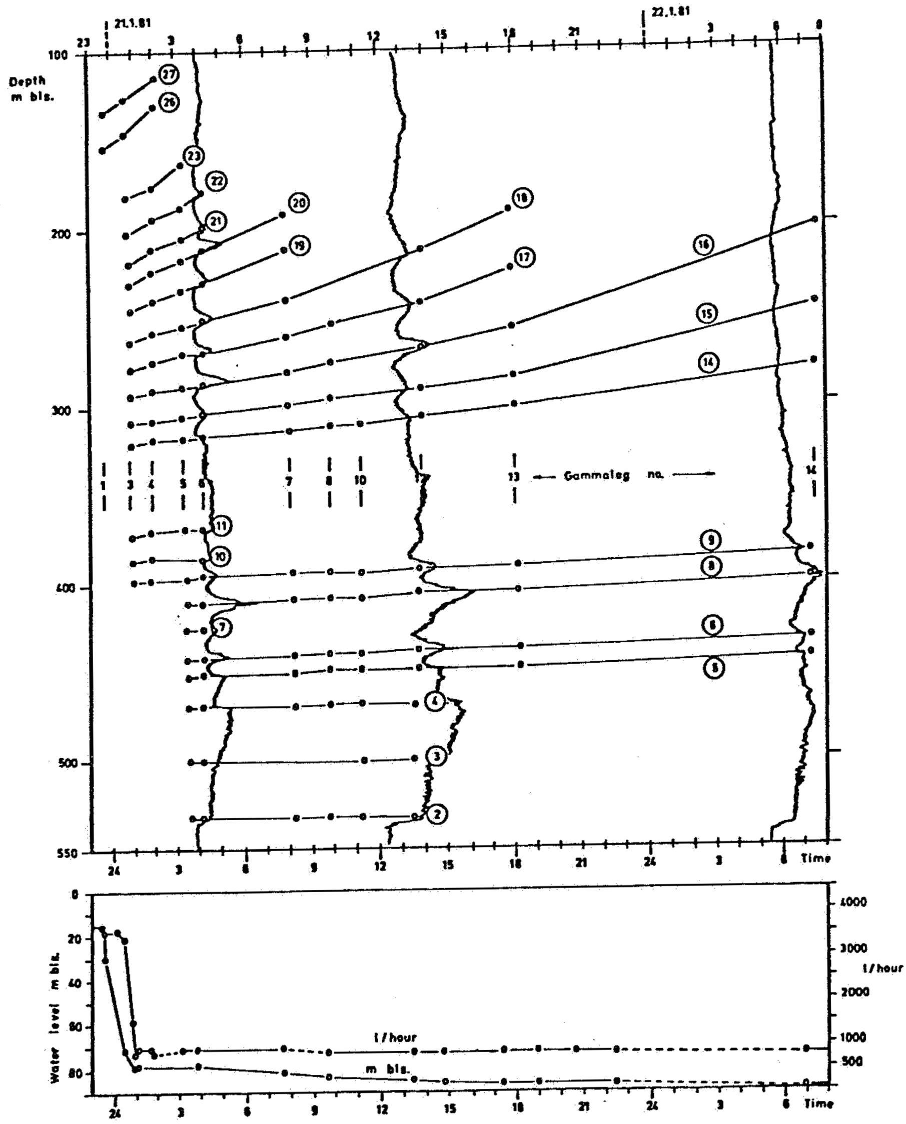

Figure 1 Labelled Slug Test data from ERSLEV-1 well
The positions of pulses, numbered 2-27, are plotted as a function of time. The flow rate and the drawdown are shown on the lower sketch

velocity decrease have been changed to an expected upward increase in velocity.

The agreement between hydraulic conductivities calculated from the LST and from the packer test proved to be satisfactory, Andersen et al. (1981) and Gosk (1981).

5 Summary

1) The LST-derived hydraulic conductivities are of the same order of magnitude as those derived from the packer pumping tests. The conventional logs used to identify open fractures or permeable zones may lead to ambiguous interpretations, if direct tests such as packer, pumping, spinner flowmeter and LST-tests are not made.
2) A considerable improvement in the resolution of the permeability profile may be obtained by using the LST. A resolution down to 1 m may be obtained if an appropriate number of tracer pulses are used.
3) The LST is always a wire-line test, and no rig is required.
4) In relation to the amount of information obtained, the duration of the LST may be regarded as very short, and the method therefore is also suitable during drilling.
5) The LST can be run prior to interval pumping or injection tests in order to select testing zones. This may be important for the evaluation of fissure permeability and may reduce the time necessary to obtain representative water samples.
6) Problems with the packer tests, such as leakage around the packers, do not exist for the LST.
7) The calculated velocities of the pulses can be influenced by the movement of the logging tool and by dispersion of the tracer. Laboratory tests are needed to quantify the effect of these factors.

References

Andersen, L.J., N. Bull and E. Gosk 1981. Results of a Labelled Slug Test in a Low-Permeability formation of White Chalk, Mors, Denmark. Proceedings of the Workshop on Siting of Radioactive Waste Repositories in Geological Formations, Paris, 19th-22nd May 1981. Nuclear Energy Agency Organisation for Economic Co-operation and Development.

Bear, J. 1979. Hydraulics of Groundwater , McGraw-Hill Series in Water Resources and Environmental Engineering, Jerusalem, 1979.

Gosk, E. 1981. Hydraulic Testing of Low-Permeable Chalk Formations, Mors, Denmark. Proceedings of the Workshop on Siting of Radioactive Waste Repositories in Geological Formations, Paris, 19th-22nd May 1981. Nuclear Energy Agency Organisation for Economic Co-operation and Development.

Gosk, G. 1983. Computer programs for processing of data from the Labelled Slug Test and the Spinner Flowmeter Test . Proceedings of the International Symposium on Methods and Instrumentation for the Investigation of Groundwater Systems, Noordwijkerhout, 2nd-6th May 1983.

Marine, I.V. 1979. Determination of the location and connectivity of fractures in metamorphic rock with inhole tracers . Proc. of NEA/ IAEA Workshop, Paris 19th-21st March, 1979. Low-flow, low-permeability, Paris 1979.

Schimschal, U. 1981. Flowmeter analysis at Raft River, Idaho. Ground Water. v. 19, no. 1, pp. 93-97.

Syms, M.C. 1982. Downhole flowmeter analysis using an associated caliper log. Groundwater, Vol. 20 No. 5, Sept.-Oct. 1982.

THE MEASUREMENT OF THE SOIL TEMPERATURE WITH THE TEMPERATURE CONE

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Abstract

In this paper the Fugro temperature cone will be discussed. The construction of the measuringdevice will be outlined, while also some test results will be evaluated in relation to the way of execution. Also the use of the temperature measurements will be indicated.

1 Introduction

A site investigation has been performed by Fugro B.V., where both cone penetration tests with local friction measurement and pore water-pressure- and temperature measurements were executed.
This site investigation took place as a part of a study initiated by Estel Hoogovens, IJmuiden and carried out by Nederhorst Grondtechniek B.V. The purpose of this study was to investigate the use of several groundworktechniques for the renovation of 'Warmbandwalserij I'. The techniques considered to have applicability are: chemical stabilised sand, reinforced augerbored piles, diaphragm walls and groutanchors. The application of these techniques must be considered in relation to the excavations immediately in the vicinity of matfoundations, the intolerance of driving vibrations, insufficient space for the slopes of a building pit and the necessity to fix retaining walls and to prevent the uplift of cellar- and tunnel floors.
This research was considered necessary as it was known that the groundwater temperature was higher than normal (5 - 10^{o} C.) in The Netherlands. This higher groundtemperature was generated by the

presence of ovens in cellars and the infiltration of hot water, originated from cooling of steel plates.
The temperature measurements taken provided the basis for research to f.e. the kipptime of the injection fluids, the possibility to reinforce the grout of the augerbored piles and the behaviour of the injection-fluid and cement grout.

2 Measuring element

In figure 1 a section of the temperature cone is shown. The drawing displays, that the measuring element, a thermal sensitive resistance, is built in a standard Fugro friction mantle cone. The temperature sensitive resistance is placed above the cone and isolated from the other part of the cone.
By means of a sounding truck, figure 2, the temperature cone is pushed

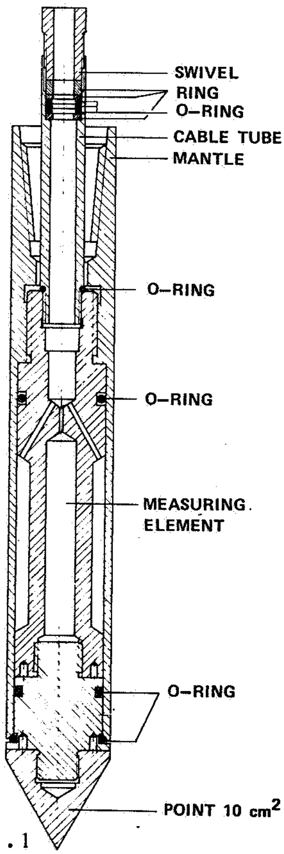


Fig.1

Fig.2

into the soil (within the trust capacity of the apparatus), which gives the possibility to perform temperature measurements at the required depths.
The measured signals of both temperature and cone resistance are transferred via an electrical measuringcable, which goes through the hollow sounding rods, to the measuring cabine of the sounding truck.
The temperature and cone resistance can directly and continuously be read of from the stripchartrecorder, placed in the cabine. The measuring range of the transducer is between -25^{o} to + 120^{o} C with an accuracy of 1^{o} C.
Further developments regarding the construction of the temperature cone are undertaken and are directed to decrease the measuring time (now being 10 - 15 min.) considerable to speed up the operation. Also the accuracy of the measuring system is being refined.

3 Test results

At the location investigated the soil consists from groundlevel up to the maximum penetrated depth of dense fine sand.
The reductions in the cone resistance are due to the presence of thin clay and/or peat layers.
At a depth of N.A.P. (Dutch Datum) -17.00 m. to N.A.P. -19.60 m a very silty sandlayer is present, with many reductions in cone resistance. These reductions indicate possibly the presence of thin peat and/or claylayers.
From a depth of N.A.P. -19.60 m to N.A.P. -20.10 m the socalled basis peatlayer is located. This peatlayer forms in the Western part of The Netherlands the separation between the Pleistocene- and Holocene deposits.
From a depth N.A.P. -20.10 m up to the maximum penetrated depth a very dense sand deposit with cone resistances in excess of 30 MN/m^2 was encountered.
Some results of the cone penetration tests with local friction measurement are presented in the Figures 3 and 4.
Besides the cone penetration tests, temperature measurements and pore waterpressure measurements were performed.

The temperature measurements and the pore waterpressure measurements are drawn on the cone penetration test results. As can be seen from these measurements the minimum recorded temperature is 17-18° C, while the maximum recorded temperature is 50° C.

Isotherms for 3 depths are presented in Figure 5. This map shows clearly the change in temperature and the areas influenced by the heatsources.

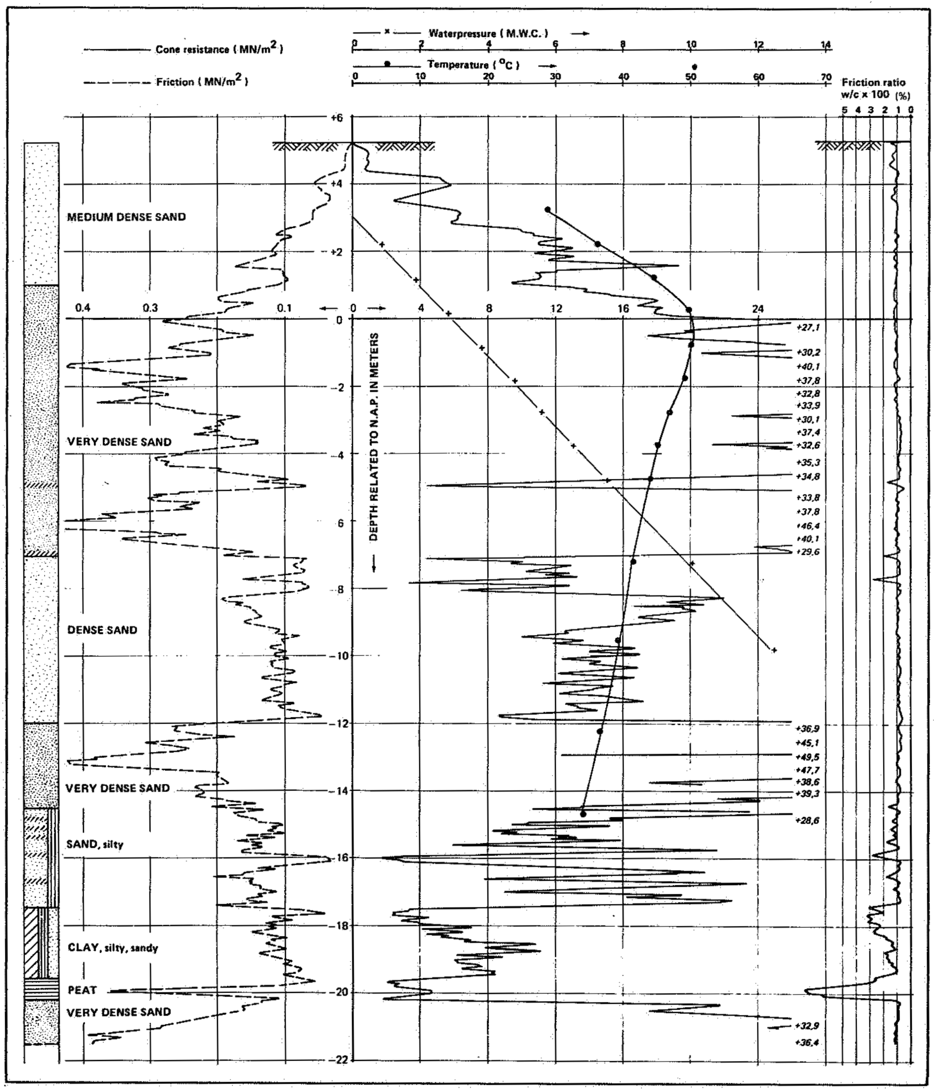


Figure 3 Cone penetration test with local friction

The pore waterpressure measurements show a hydrostatic pressure distribution.

The differences in hydraulic head between the locations is controlled by the infiltration of water for cooling the steel plates.

It appeared that the groundwater flow velocities were such that no washing out of grout was to be expected.

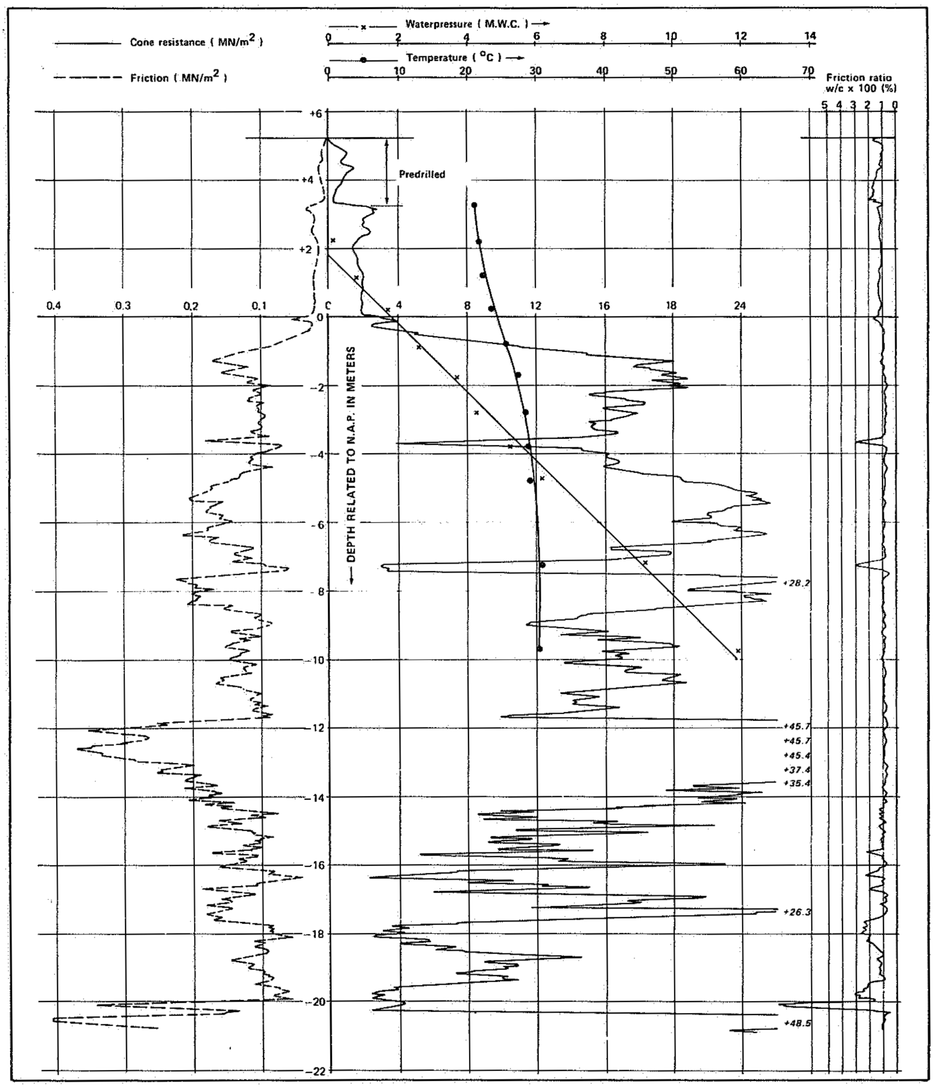
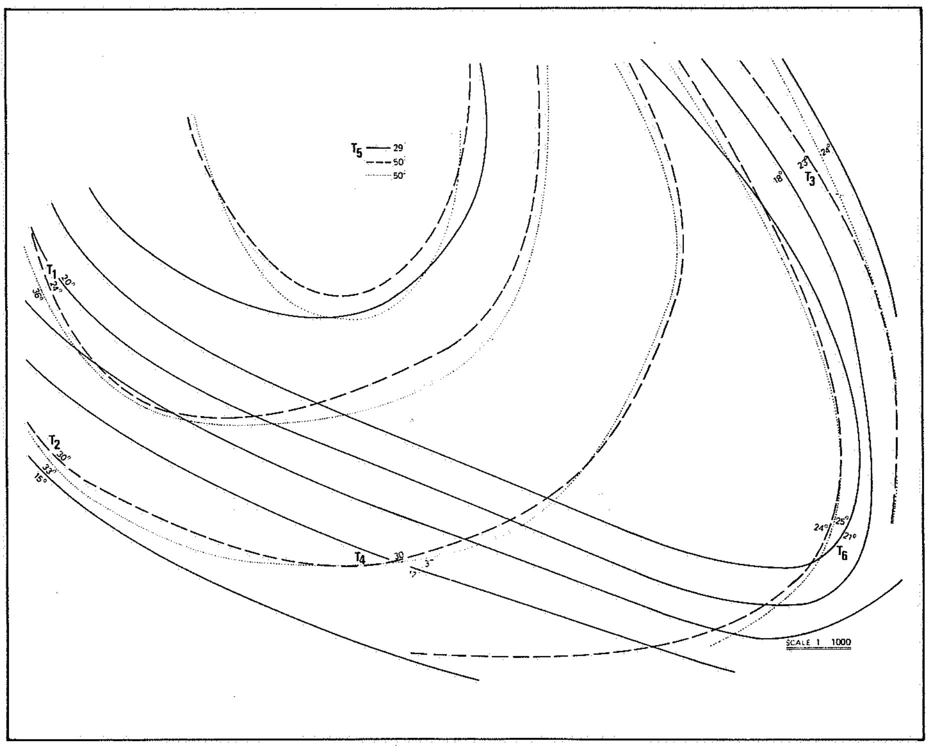


Figure 4 Cone penetration test with local friction

4 Influence of the testing method on the measurement results

Before carrying out the temperature measurements the measuring element must be calibrated. This can be achieved by placing the temperature cone in water and to check the reading obtained with that measured with a calibrated thermometer.
By repeating these actions before and after each completed temperature sounding the possibility exists to control the i.e. possible zerodrift. Bearing in mind the construction of the temperature cone it is clear, that some accommodation time is unavoidable before the exact temperature is measured. To investigate how fast the measurements can be performed especially in relation to the accomodation time three ways of execution



Legend:

——————— 3.5 m + N.A.P.

— — — — N.A.P.

·············· 1.0 m - N.A.P.

Figure 5 Isotherm map for 3 depths

were considered.

Method a: pushing the cone into the soil up to the desired depth and making a test to repeat this procedure until the maximum depth is reached.

Method b: pushing the cone in a standard way into the soil up to the maximum depth and carrying out the temperature measurements at each required depth when retracting the cone stepwise.

Method c: in addition to the first method, some measurements can be made when lifting the sounding rods.

The penetration speed for all methods is ca. 20 mm/s.
The results of the 3 above mentioned methods are presented in the Figures 6 and 7.
The execution time following method a and b does not show significant differences. In the case of using method c however the execution time is about one hour longer than for the other methods.
The accomodation time under these circumstances was 10 - 15 min.
The time necessary to carry out the above measurements was about 3 hours, incorporating 11 - 13 temperature measurements.
Summarizing the afore mentioned it is concluded that method b prevails, as it has the big advantage of performing a standard cone penetration test (Dutch Standard NEN 3680, U.S. Standard ASTM D3441), while by lifting the temperature cone an accurate test can be performed at each required depth.
Another advantage of this method is, that an indication of the soil strata is present, so one is able to decide at the site at which levels the soil temperature must be measured.

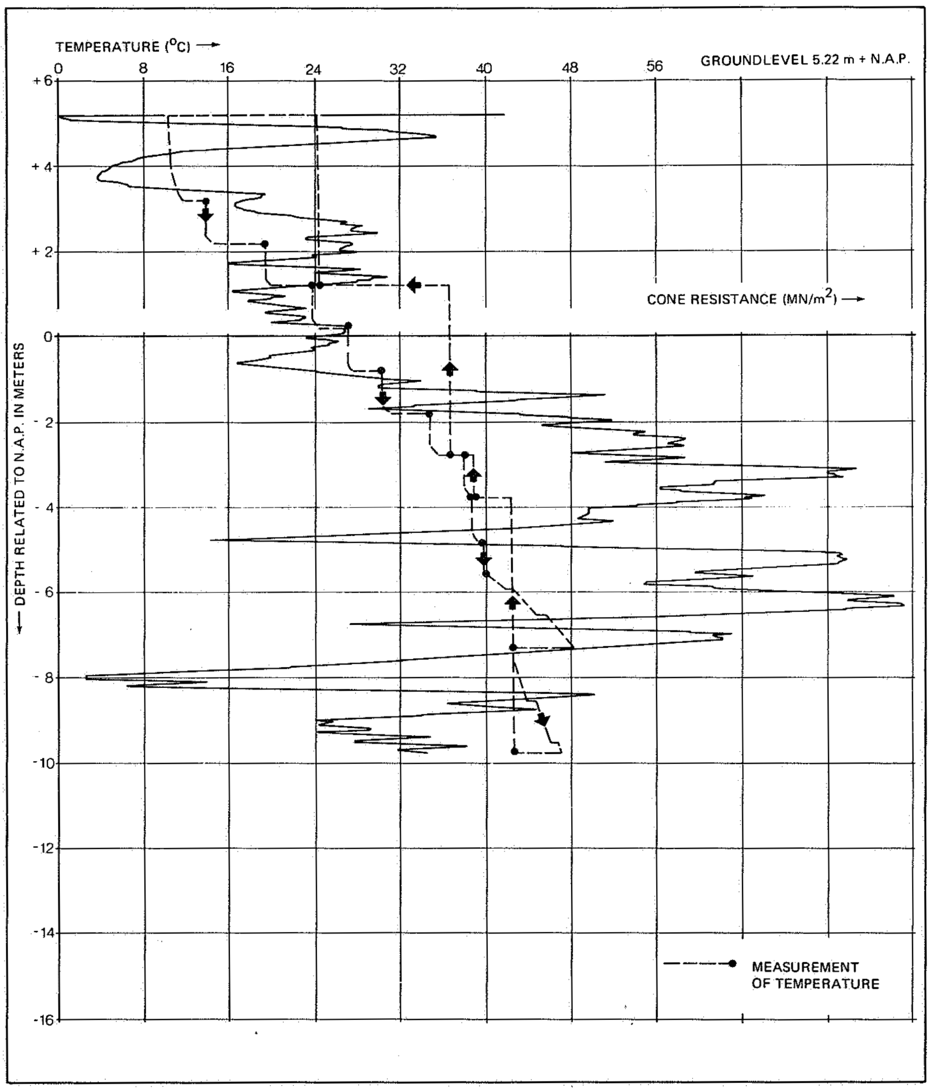


Figure 6 Cone penetration test

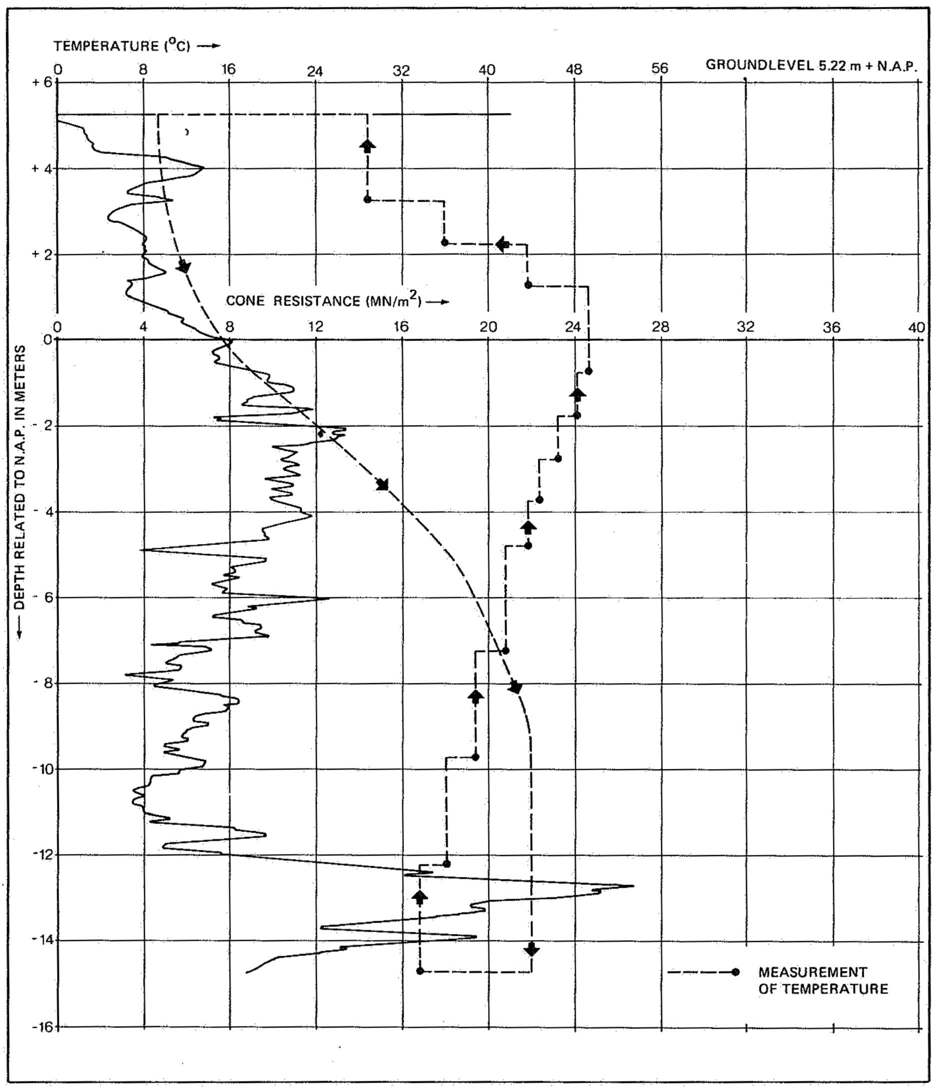


Figure 7 Cone penetration test

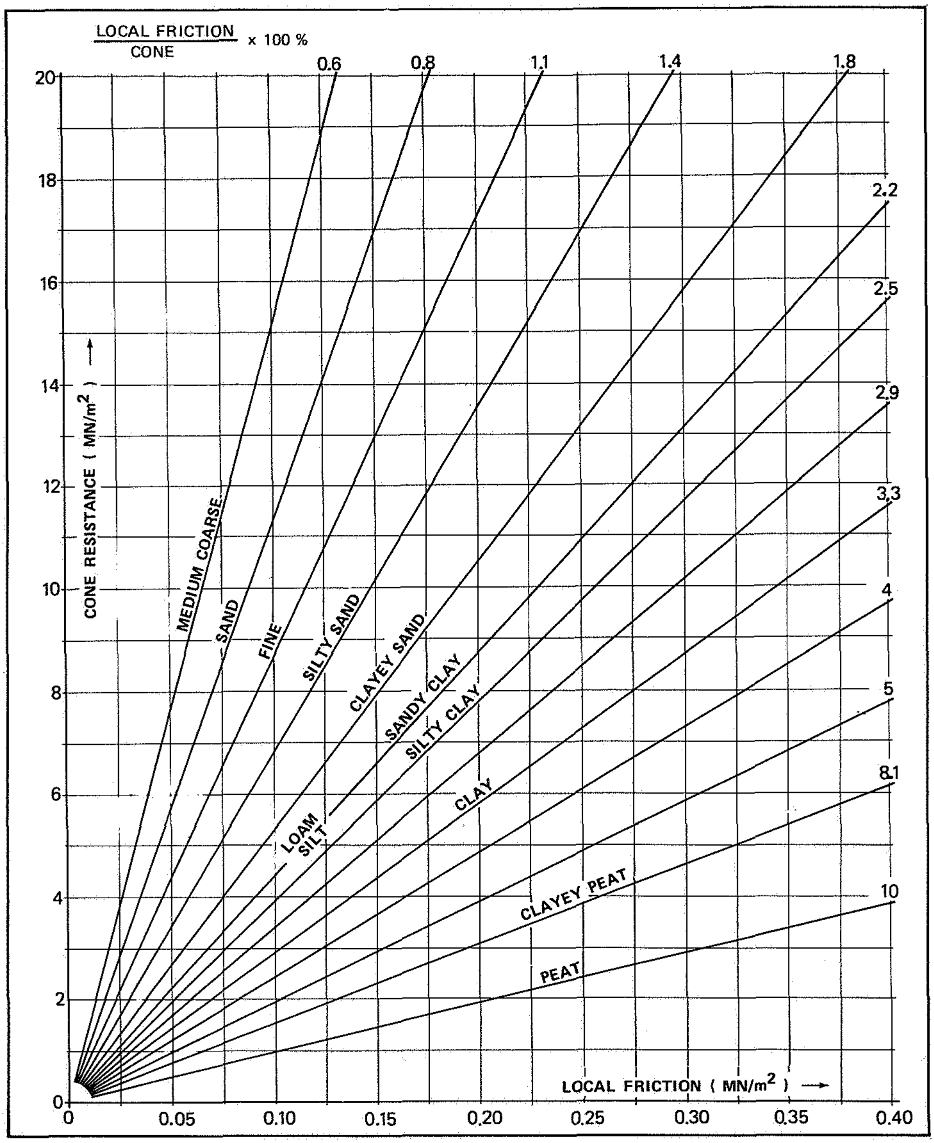


Figure 8 Correlation between cone and local friction to determine sub bottom layers

5. Applications

The temperature of the soil can be measured with the temperature cone up to the permissible sounding depth of the soil profile. With a 200 kN sounding truck and without special arrangements (extra load, friction reducers) it is possible to penetrate up to 40 m below groundlevel in The Netherlands.

The knowledge of the temperature distribution in the soil can be important for several kind of problems, such as:

- Civil engineering problems
  . selection of injection fluid
  . isolation problems (freese houses)
  . heat loss through pipelines (town heating)
  . frost penetration
- Thermal energy
  . investigation for hotwatersources
- Thermal investigation
  . ground waterflow (seepage, infiltration, temperature change)
  . influence of activities at groundlevel (heat conduction, infiltration, environmental pollution)
- Agriculture problems
  . influence of the temperature at harvest etc.

In the case of an extensive temperature investigation it is recommended to supplement the temperature cone soundings with cone penetration tests with local friction measurement.

From the latter tests the friction ratio, being the ratio of local friction and cone resistance can be computed. This friction ratio together with the cone resistance provides an insight in the soil strata.

The relation of cone resistance, local friction and friction ratio for the electric Fugro friction mantle cone is presented in Figure 8.

In certain circumstances it is recommended to perform a boring in the area investigated.

A NEW METHOD FOR IN SITU MEASUREMENTS OF PERMEABILITY

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Summary

The usual methods for determining the permeability in situ are the constant and falling head method and the pumping test. The first two are very often inaccurate and the last one is expensive.
Therefore, a new method has been developed: the dipole-probe. This probe is in principle analogous to the electrical porosity probe of our laboratory: water is pressed into the soil through the topfilter and sucked up through the lower filter or vice versa at a certain rate; by means of two other filters placed between the first two the potential difference is measured. From the rate of flow and potential difference the coefficient of horizontal permeability can then be calculated.

Field results

- With this probe the local coefficient of horizontal permeability of an anisotrope sandy soil can be measured in situ in a simple way.
- For determining the less needed coefficient of vertical permeability, two probes are needed. This method has proved to be less accurate and will therefore not be discussed.
- For a layered soil an average value is found.
- This probe cannot be used in clay.

Résumé

Les méthodes habituelles pour déterminer la perméabilité in situ sont l'essai d'infiltration à niveau constant, l'essai d'abaissement, et l'essai de pompâge. Les deux premiers sont souvent peu précis, et le

dernier est onéreux.
Pour cette raison une sonde nouvelle a été développée. En principe cette sonde est analogue à la sonde électrique de porosité de notre Laboratoire: à un certain débit d'eau est injectée dans le sol à travers le filtre supérieur et aspirée par le filtre inférieur, ou à l'envers; au moyen de deux autres filtres placés entre les premiers deux, la différence de potentiel est mesurée. Or, le coefficient de perméabilité horizontale peut-être calculé du débit d'eau et de la différence de potentiel.

Résultats practiques

- Avec cette sonde le coefficient local de la perméabilité horizontale d'un sol sableux anisotrope peut-être mesuré d'une façon simple.
- Pour déterminer le coefficient de perméabilité verticale, qui est rarement requis, deux sondes sont nécessaires.
  Cette méthode s'est montrée moins précise, et en conséquence ne sera pas traitée ici.
- Dans un sol stratifié on trouve une valeur moyenne.
- La sonde ne peut pas être utilisée en argille.

1 Introduction

The coefficient of permeability, especially of sandy soils, is among other things essential for calculating the flow of groundwater.
The coefficient of permeability is in situ usually measured as follows:

1) Constant head method } direct in situ
2) Falling head method } methods
3) Using an indirect method to determine permeability, the porosity of the sand is measured in situ with an electrical method. Afterwards, in the laboratory, the coefficient of permeability of a sample is determined as a function of the porosity. Indirectly by interpolation the coefficient of permeability is then known for the in situ measured porosity.

Disadvantages of the methods mentioned so far:

a) Around the probe a layer of disturbed and often crushed sand is formed. It is therefore impossible to measure the coefficient of permeability of the undisturbed soil.
b) The pressure is here measured in the probe before the filter, so its

resistance should be known. This resistance, however, is not constant, as the filter becomes gradually clogged.

c) Using method 3 one sample is prepared successively in two or more different porosities. At each porosity the coefficient of permeability is determined. The anisotropy or other irregularities, if any, are then lost.

With the new probe only drawback a) remains. This disadvantage can never be avoided when a probe is pressed into the soil. It will be shown that the influence of this disturbed layer on the measured coefficient of permeability, determined with our newly developed probe, is relatively small.

In conclusion we can say that the dipole-probe is a useful low-cost apparatus for determining the permeability in situ.

2 The principle of the new method

Most soils are anisotropic with coefficients of horizontal and vertical permeability, respectively k_h and k_v.

Defining in such soil q as the rate of discharge of a source or sink, r,z as horizontal resp. vertical distance from a certain point to source or sink and $h_{r,z}$ as change of potential in that point the following is obtained:

$$h_{r,z} = \frac{q}{4\pi\sqrt{k_h k_v}\sqrt{r^2 + \frac{k_h}{k_v} z^2}} \quad \text{hence } h_{r=0,z} = q/(4\pi k_h z) \qquad (1) \text{ and } (2)$$

On the line r = 0 the value of $h_{r,z}$ is only a function of k_h, as it happens the most important coefficient of permeability. Therefore, a permeability probe with the following characteristics was considered:

- source or sink with constant rate of discharge
- potential sensor on the vertical line through source or sink.

To limit the area around the probe in which the average permeability is determined and because potential differences are easier to measure, the probe was improved as follows:

- On the line r = 0 through source or sink a sink or source with the same rate of discharge was placed. Between source and sink a second potential sensor was placed and between both sensors the potential difference was measured.

With this probe the local coefficient of horizontal permeability is measured. Because of source and sink the new probe was called dipole-probe.

3 Description of the dipole-probe

Figure 1 is a schematic drawing of the probe. The piston pump presses water through filter A into the soil. Through filter B water enters the probe at the same rate. The filters A and B thus form a pair of source and sink. During measuring the rate of discharge of the pump is kept constant; it can be changed, however, with a variable-speed drive. The differential pressure gauge measures the potential difference between the filters C and D. The coefficient of horizontal permeability k_h is calculated as follows:

$$h_C = \frac{q}{4\pi k_h\ (L_1-L_2)} - \frac{q}{4\pi k_h\ (L_1+L_2)}\ ,\ h_D = -h_C$$

$$h_C-h_D = \frac{q}{2\pi k_h}\left(\frac{1}{L_1-L_2} - \frac{1}{L_1+L_2}\right) \text{ or } k_h = \frac{q}{C(h_C-h_D)} = \frac{q}{C\Delta h} \tag{3}$$

$$\text{with } C = \frac{2\pi}{\dfrac{1}{L_1-L_2} - \dfrac{1}{L_1+L_2}}\ ,\ \text{and } \Delta h = h_C - h_D \tag{4}$$

In reality the filters A, B, C and D are not points but cylindrical surfaces, so formula (3) is only an approximation and the separation of k_h and k_v is therefore not as exact as formula (2) would suggest.

The value C of formula (4) can experimentally be determined as follows: A dummy probe of the same dimensions, made of an isolator and metal filters, was made. The probe was tested in a swimming-pool with water of known specific resistance ρ_w. Between the electrodes A and B a known current I flows through the water, while between the electrodes C and D a potential difference U exists. Because of the analogy between the laws of Ohm and Darcy it follows that: $1/\rho_w = I/(CU)$. This value of C deviates only a few percent from that of formula (4).

During measuring the probe must be submerged in the groundwater. Special care should be taken that the pump and tubes are filled with water. When the probe is pressed into the soil the system is kept under slight over-pressure to avoid clogging of the filters.

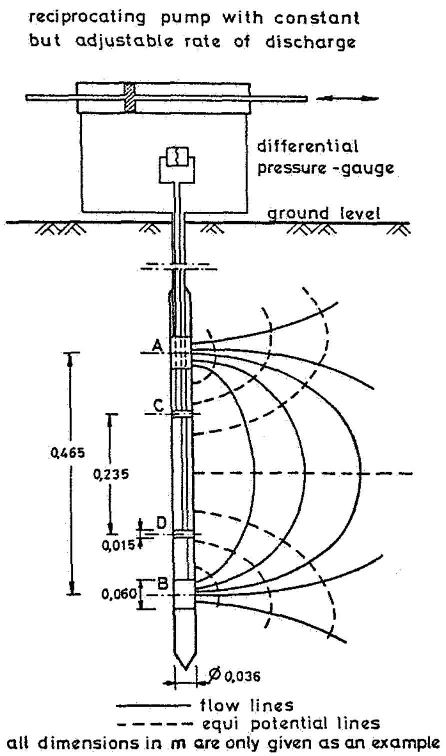


Figure 1 Schematic drawing of dipole-probe

The main data of the probe according to Figure 1 are:

10^{-3} m/s > k_h > 10^{-7} m/s

10^{-4} m³/s > rate of discharge of pump > 10^{-6} m³/s

C calculated with (4) 1,067 m

C measured in swimming-pool 1,119 m.

4 Verification of formula (3)

In practice not only anisotropy i.e. $k_h \pm k_v$ is found but also disturbances consisting of horizontal layers of different permeability. Also in that case only formula (3) or (6) is available. The determination of C cannot be improved either. In order to verify formula (3) a few such cases were checked by means of our program SEEP. The results are given in table 1.

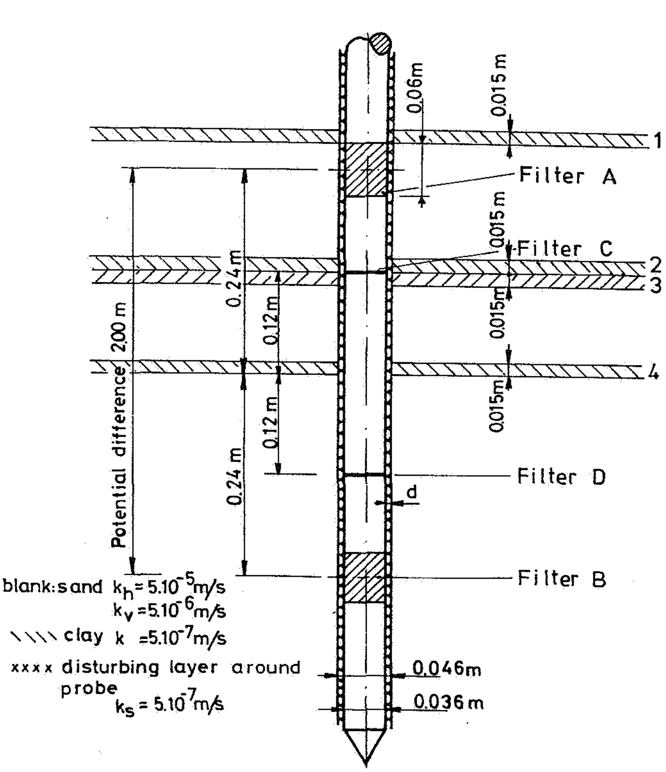
Table 1

	1	2	3	4	5	6	7	8	9	10	11
	fig. nr.	$k_h x10^6$ m/s	$k_v x10^6$ m/s	$dx10^3$ m	$k_s x10^6$ m/s	$kx10^6$ m/s	layer nr.	$qx10^3$ m^3/s	Δh m	C m	$k_h x10^6$ m/s
A	3	50	50	0	-	-	-	1,85	30,2	1,23	49,8
B	3	50	50	0	-	0,5	1	1,62	31,3	1,04	42,1
C	3	50	50	0	-	0,5	2	1,72	15,0	2,29	93,2
D	3	50	50	0	-	0,5	3	1,73	52,7	0,666	26,7
E	3	50	50	0	-	0,5	4	1,76	44,2	0,796	32,4
F	3	50	50	5	0,5	-	-	0,0943	1,49	1,26	51,5
G	3	50	5	5	0,5	-	-	0,0913	1,52	1,20	48,8
H	3	50	5	5	0,5	0,5	1.2 3.4	0,0903	2,12	0,852	34,6
I	4	50	5	5	0,5	0,5		0,0886	1,63	1,09	44,2

The columns should be read as follows:

Column	Description
1	Number of figure giving all data of soil and disturbing layers
2	k_h of undisturbed soil
3	k_v of undisturbed soil
4	Thickness of layer of disturbed soil around probe
5	Coefficient of permeability k_s of isotropic layer according to column 4
6	Coefficient of permeability k of the isotropic horizontal disturbing layers
7	Number of layers of Figure 2
8	Rate of discharge q. In all calculations the potential difference between the filters A and B was 200 m. This column shows how q varies with changing permeability
9	Potential difference Δh between the filters C and D
10	C calculated with formula (3), q of column 8, k_h of column 2 and Δh of column 9
11	k_h calculated with formula (3), q of column B, and C = 1,23 m, of row A.

In the rows A-H several variants of figure 2 have been calculated. The rows F and G clearly show that the anisotropy and the disturbing layers around the probe have only a slight influence. Much greater is the influence of the disturbing layers 1, 2, 3 and 4 separately, rows B-E,



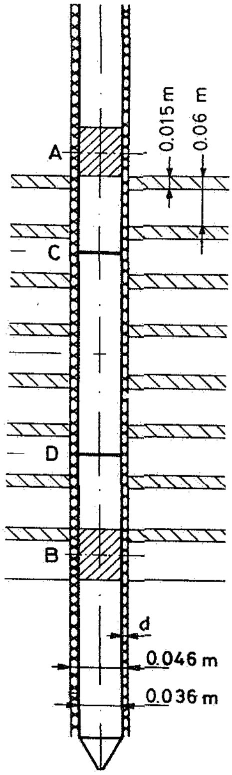


Figure 2 Dimensions of probe and disturbing layers of rows A ÷ H

Figure 3 Dimensions of probe and disturbing layers of row I

and together, row H. These disturbances influenced the flow to such a degree that the assumptions on which formula (3) was based are not valid any more. In practice, these disturbances are never known and therefore formula (3) has to be used all the same and deviations have to be accepted.

In Figure 3 several disturbing layers between the filters A and B are given. The influence on k_h is only slight, row I.

In practice there are always disturbing layers and the measured value of k_h will always deviate from the real value. The best solution is therefore to measure k_h at small intervals and to smoothe the function found in this way. The average value of k_h of the rows B, C, D, E, H and I, that is to say with disturbing layers, amounts to $45,5 \cdot 10^{-6}$ m/s. The error is now only 8,8% of the accurate value of $50 \cdot 10^{-6}$ m/s.

5 Measuring results

From a pontoon on a lake the following values of k_h (Table 2) were found.

Table 2

Depth m	Rate of discharge $x10^6$ m³/s	Δh m	$k_h x10^6$ m/s	kind of soil observations
7,00	4,559	0,106	38,4	sand
7,25	4,559	0,085	47,9	"
7,50	4,559	0,088	46,3	"
7,75	4,559	0,091	44,8	"
8,00	4,559	0,114	35,7	"
8,25	2,533	0,068	33,3	probe enters a less permeable layer consisting of sand, silt and humus
8,50	1,520	0,725	1,87	middle of layer
8,75	1,520	0,260	5,22	probe leaves this layer again
9,00	3,039	0,120	22,6	" " " " "
9,25	4,559	0,096	42,4	sand, less homogeneous than above - 8 m
9,50	4,559	0,063	64,7	sand
9,75	5,066	0,054	83,8	"
10,00	5,066	0,056	80,9	"
10,25	5,066	0,054	84,6	"
10,50	5,066	0,020 0,050	-	sand, h was not constant, therefore k_h not calculated
10,75	5,066	0,029	156	sand
11,00	7,09	0,019	333	"
11,25	7,09	0,017	373	"
11,50	7,09	0,019	333	"
11,75	8,11	0,017	439	"
12,00	8,11	0,031	233	"
12,25	8,11	0,036	201	"
12,50	6,08	0,120	45,3	"
12,75	6,08	0,028	194	"
13,00	6,08	0,018	302	"

In Figure 4 k_h is given as a function of the depth according to Table 2. A boring showed that the soil above the layer at 8,5 m depth was more homogeneous than underneath that layer. The curve us in concordance with this. The curve is smoothened by means of the average values above and underneath this layer: 41 . 10^{-6} resp. 272 . 10^{-6} m/s. It was supposed

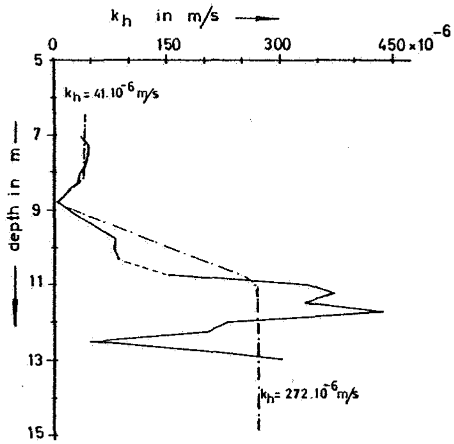


Figure 4 k_h as a function of depth

that the value at 12,5 m depth was too small. Because of that this value was not taken into account.

As other measurings also gave differences both plus and minus and as at trials in homogeneous sand of known permeability the differences were negligible, it may be supposed that the dipole-probe gives the right values.

6 Conclusion

It has been shown above that in homogeneous anisotropic soils the right local value of the coefficient of the horizontal permeability is measured with the dipole-probe. At in situ measurements a smoothening according to Figure 4 is always advisable.

In situations according to Figures 2 and 3 layers of sand and clay it lasts sometime very long before Δh reaches its right value, so that in such cases the use of the dipole-probe will be limited.

ANALYSES OF THE DATA OF THE NATIONAL GROUNDWATER MONITORING SYSTEM

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Abstract

In The Netherlands a network of 320 wells has been installed for monitoring the groundwaterquality in the overlaying- and first waterbearing aquifer. Normally, the concentration of nineteen parameters in the groundwater is determined, but the network is also used for special sampling.
This paper deals with the handling of the data and describes methods for statistical analysis and for the presentation of the geographical spreading of the groundwaterquality. Special attention is payed to the relation between groundwaterquality on one hand and soil use or soil type on the other.
Results for some parameters are shown.

1 Introduction

A large quantity of the Dutch drinking water is obtained from groundwater. During the past few years it has become evident that the quality of this groundwater is not as good as previously assumed. Due to local - and diffuse pollutionsources an increasing amount of groundwater is polluted, in some cases even directly threatening public health. Therefore the Ministry of Public Health and Environmental Protection has assigned the National Institute for Water Supply to establish a national groundwaterquality monitoring system.

The objectives of the monitoring system are:

- to make an inventory of the present quality, in addition to already available data;
- to distinguish long term changes in quality;
- to provide data for groundwaterquality control;
- to indicate the extent of human influence on groundwaterquality.

Therefore, the main goal of the system is not to find local sources of pollution, but to observe diffuse pollution and to locate sources of this pollution. (For local sources, in the context of the national soil sanitation program, local monitoring networks have been installed).

The construction of the network will be completed early 1983. Some 380 monitoring stations will be installed then. This is equivalent to an average monitoring ratio of one station per 80 square kilometers. The locations are carefully choosen in order to obtain a representative image of the groundwaterquality. For methods and considerations of the construction vide van Duijvenbooden (1979, 1981). Since the network is not finished yet, and only a few boreholes have been sampled more than twice, no attention has been payed up till now to the analysis of time series. The statistical research is at the moment focussed on the relation between soil type or soil use on one hand and on the other the groundwaterquality measured during the first sampling period of the monitoring system. Also, it will be shown that for some parameters it is useful to take into account the geographical spreading.

2 The installation of the monitoring system

Figure 1 gives a view of the locations of the sampled wells.
The installation of the network is finished for more than 80%. At the moment (sept. 1982) 105 wells were sampled once, 130 twice and 64 three times. The wells are equiped with three wellscreens at about 10, 15 and 25 meters below groundlevel. Due to local conditions these depths may vary. The frequency of sampling will be approximately once a year in the nearest future. The basic analysis package is given in table I. Incidently, a large number of trace elements are determined also.

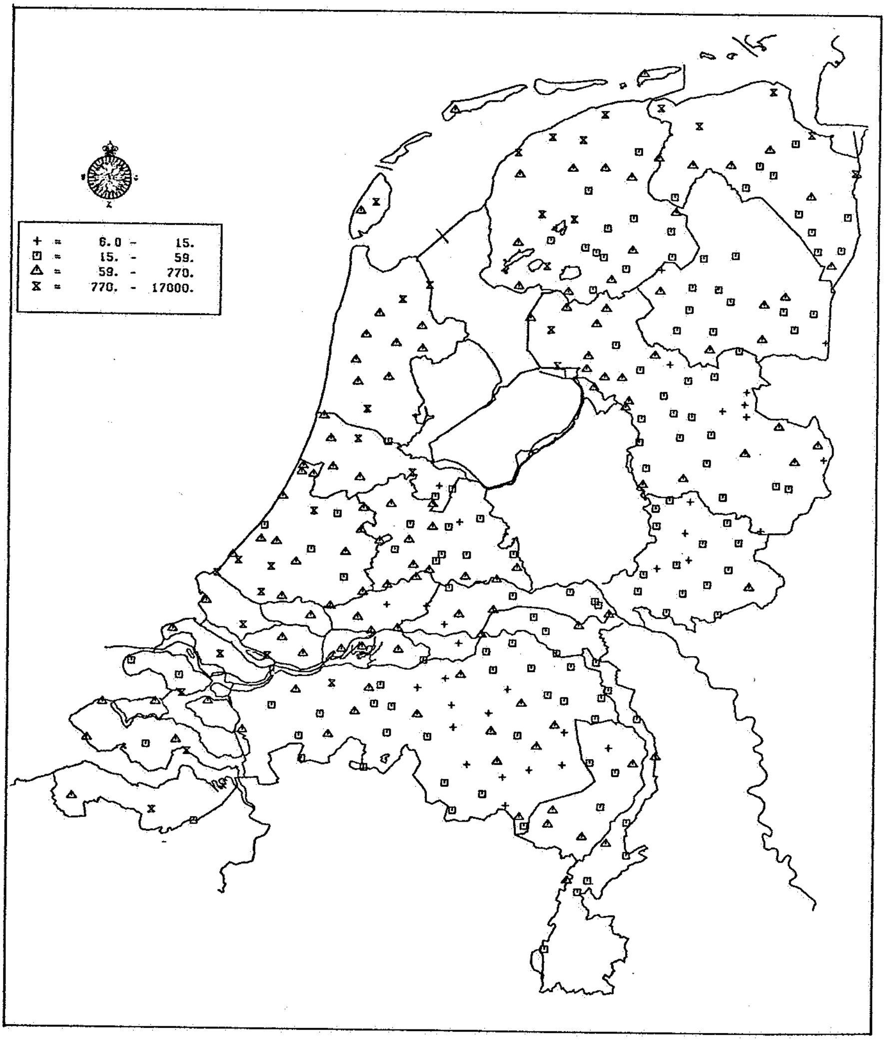


Figure 1

Cloride concentrations (mg Cl/l) in the first sampling of the filter of the monitoring network. Each symbol also represents a sampled well

Table 1 Basic chemical analyses program

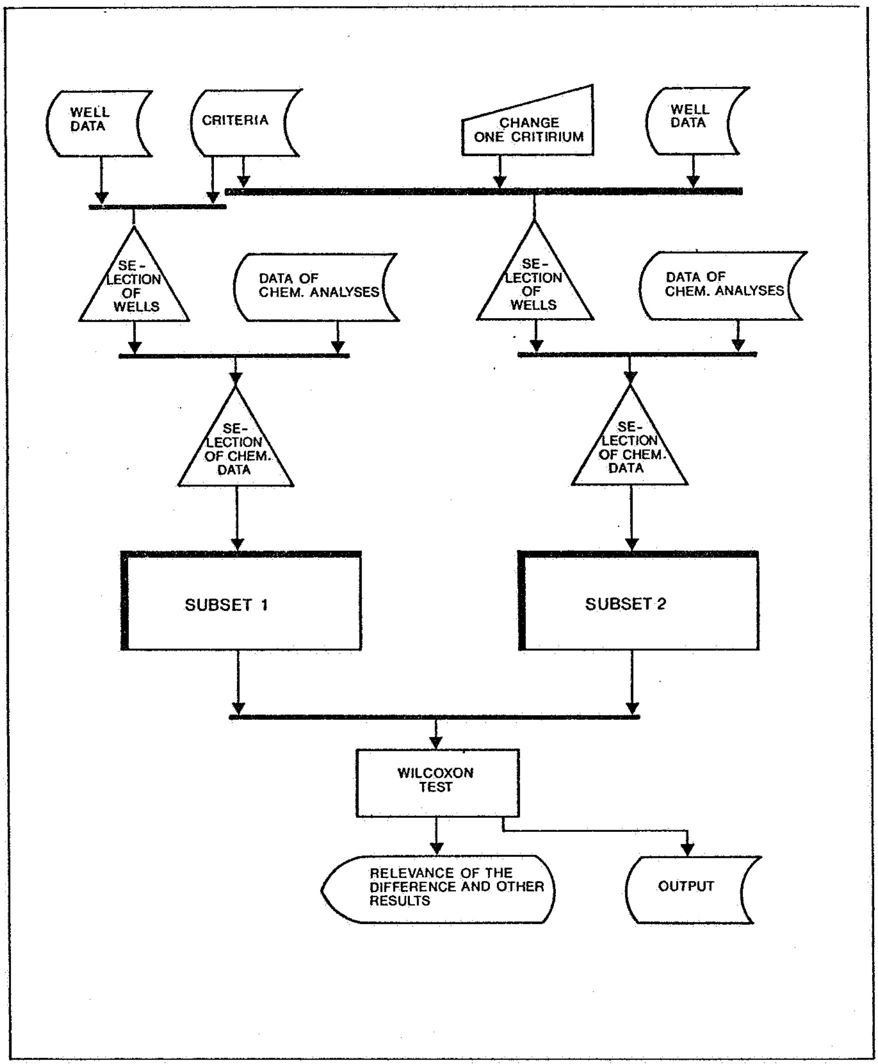
KjN	Ca^{2+}	Zn	$KMnO_4$
NO_3^-	Mg^{2+}	Ni	TOC
Cl^-	Na^+	As	VOCl
SO_4^{2-}	K^+	pH	EOCl
HCO_3^-	tot. P	Conductivity.	

The results of the chemical analysis are punched and stored on magnetic disk. (The analyse form is at the same time punch form). A PDP 11/34 minicomputer is used for processing these data. A number of interactive FORTRAN 77 programs has been developed for:

- the management of the network (progress control and output of the measured data for publication);
- statistical analyses;
- graphical presentation.

3 Statistical comparison of data

In order to distinguish the influence of soil type, soil use and geohydrological situation a statistical program MNSTA (Monitoring Network STAtistics) was written. This program creates two subsets of data, and compares these subsets with each other.
Scheme 1 shows a simplified chart of the information flow in the program.
In the file "well data" a number of properties of the wells are stored, such as: vertical direction of groundwaterflow; semi-pervious layers; fresh-, brackish- or salt water (depending on the measured chloride concentrations); soil use; soil type; etc. In the file "criteria" one defines what type of wells are to be statistically analysed. The program determines which wells satisfy these criteria by comparing them with the well data. From the file "data of chemical analysis" the relevant chemical data are searched and the first subset is formed. The program user can change one of the criteria, hence the program repeats the procedure as already described and a second subset is created. The Wilcoxon - test is applied to determine whether or not



Scheme 1

A simplified chart of the information streams in the statistical program MNSTA. (using ISO R1028 flow chart symbols)

SOIL USE - SOIL TYPE	H+ pH	Na+ mg/L	K+ mg/L	Mg2+ mg/L	Ca2+ mg/L	Ni ug/L	Zn ug/L	NO3- mgN/L	SO4= mg/L	PO4- mgP/L	KjN mgN/L	TOC mgC/L	EOCl ugCl/L	VOCl ugCl/L
RURAL LAND - SAND	6.0	410	25.	65.	110	37.	56	18.	130	1.1	11.	7.7	0.7	0.3
NUMBER OF OBSERVATIONS	31	35	35	35	35	35	35	35	35	35	35	34	33	35
RURAL LAND - CLAY	7.0	1600	58.	200.	250	29.	52	1.0	310	2.2	13.	28.	0.6	0.2
NUMBER OF OBSERVATIONS	20	20	20	20	20	20	20	20	20	20	20	20	19	20
RURAL LAND - PEAT	5.7	40	6.0	12.	56	7.5	24	2.8	42	0.1	3.6	16.	1.0	5.1
NUMBER OF OBSERVATIONS	8	8	8	8	8	8	8	8	8	8	8	8	7	8
GRASSLAND - SAND	6.4	33	8.2	8.4	74	21.	85	1.8	88	0.3	2.3	11.	0.5	0.1
NUMBER OF OBSERVATIONS	53	62	62	62	62	62	62	62	62	62	62	62	62	62
GRASSLAND - CLAY	7.1	220	21.	45.	160	8.8	12	0.1	78	1.1	8.0	11.	0.4	0.1
NUMBER OF OBSERVATIONS	30	30	30	30	30	30	30	30	30	30	30	30	30	30
GRASSLAND - PEAT	6.9	160	7.2	23.	89	7.1	13	0.1	17	1.2	9.6	17.	0.5	0.1
NUMBER OF OBSERVATIONS	21	21	21	21	21	21	21	21	21	21	21	21	21	21
NATURAL AREAS - SAND	5.8	29	3.5	5.6	23	21.	41	1.4	43	0.2	1.1	4.1	0.4	0.2
NUMBER OF OBSERVATIONS	31	36	36	36	36	36	36	36	36	36	36	35	34	36
NATURAL AREAS - CLAY	6.9	10	1.5	9.9	78	12.	69	0.1	37	0.8	2.2	7.0	0.2	0.1
NUMBER OF OBSERVATIONS	2	2	2	2	2	2	2	2	2	2	2	2	2	2
NATURAL AREAS - PEAT	6.5	25	3.5	10.	43	4.0	15	0.1	17	0.2	2.4	12.	0.4	0.1
NUMBER OF OBSERVATIONS	4	4	4	4	4	4	4	4	4	4	4	4	4	4
URBAN AREAS - SAND	6.7	67	20.	15.	110	12.	34	3.4	80	0.6	6.6	10.	0.5	0.2
NUMBER OF OBSERVATIONS	23	24	24	24	24	24	24	24	24	24	24	24	24	23
URBAN AREAS - CLAY	7.1	410	31.	150.	140	6.4	13	0.1	42	1.2	11.	11.	0.5	0.2
NUMBER OF OBSERVATIONS	7	7	7	7	7	7	7	7	7	7	7	7	7	7
URBAN AREAS - TOTAL	6.8	150	22.	48.	120	10.	28	2.4	73	0.8	7.8	11.	0.5	0.2
NUMBER OF OBSERVATIONS	33	34	34	34	34	34	34	34	34	34	34	34	34	33
OTHER SAMPLES	7.1	380	22.	61.	130	6.6	17	1.2	120	1.0	7.2	17.	0.8	0.6
NUMBER OF OBSERVATIONS	62	62	62	62	62	62	62	62	62	62	62	62	59	61

Table 2 Mean values of the first sampling of filter 1

Total number of samples 314

the change in the criteria influences the concentration distribution in the created subset. (For a description of the Wilcoxon test see Mann and Whitney (1947). The algorithm used is given by Wijvekate (1976)). The program lists the results of the Wilcoxontest, the mean values and standard deviations of the two subsets on the terminal. A more elaborate output (with the criteria and the concentrations in the wells) is written on an outputfile.

An example of the use of this program:
"Is there a significant difference in nitrate concentrations in agricultural areas with sandy soils and agricultural areas with clays?"

From the well data the program reads which wells are sited in agricultural areas with sandy soils and subset 1 is formed. The user changes the criterium of the type of soil from sand in clay, subset 2 is formed and the program compares subset 1 with subset 2. Result: "The chance of the nitrate concentration in agricultural areas with sandy soils being not larger than the nitrate concentration in agricultural areas with clay, is less than 0.5%". It is obvious that in agricultural areas with sandy soils the nitrate concentration is significantly higher than in agricultural areas with clayey soils.

An example of the output of mean values arranged to soil use and soil type is given in table 2.

4 Geographical spreading of the groundwaterquality

Another convenient way of presenting the chemical data is a geographical plot. Two examples are given in figure 1 and 2: the chloride and Kjeldahl nitrogen (ammonia + organic nitrogen) contents in filter 1 (circa 10 mtr. below surface). The concentrations are represented by four ranges, (but also individual analysis results can be plotted, on scales up to 1 : 400,000). If none of the samples has a concentration less than the detection limit, the ranges are determined as follows:

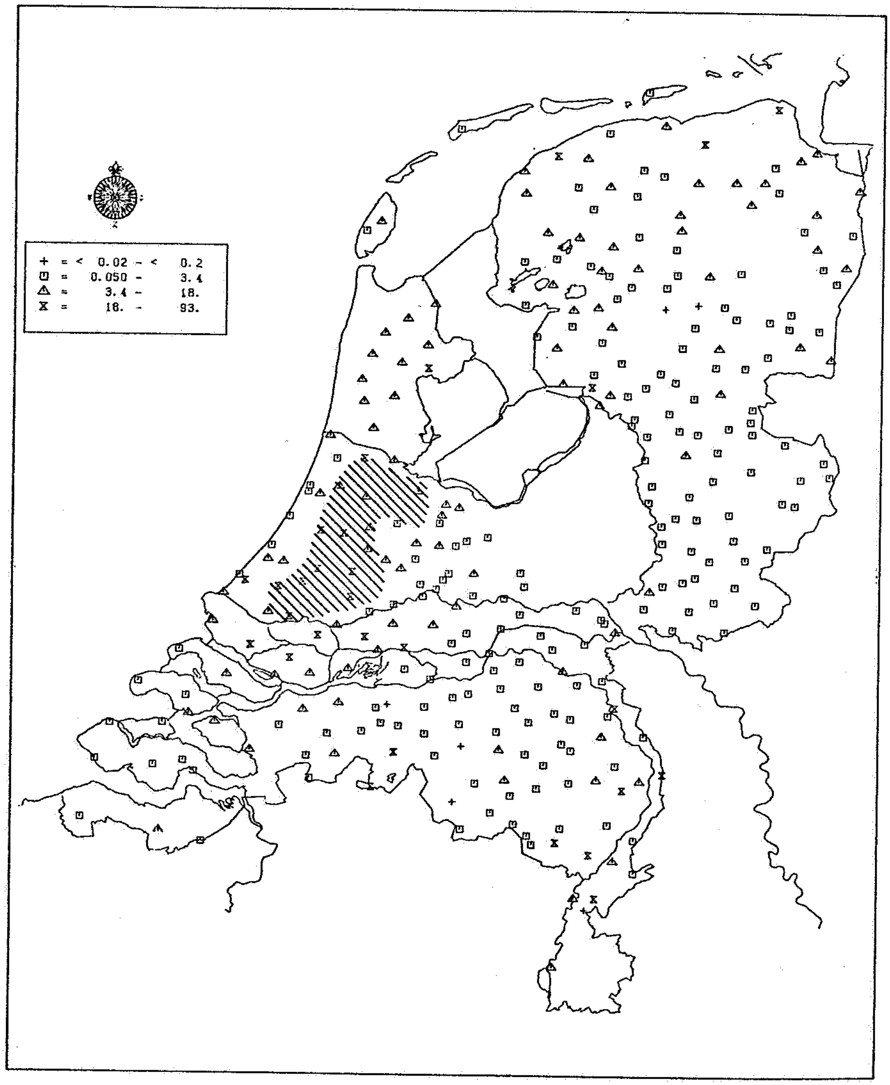


Figure 2
Kjeldahl nitrogen concentrations (mgN/l) in the first sampling of the first filter of the monitoring network. The hatched area represents peaty soils in South Holland

Range 1 lowest values	c. 10% of the total
Range 2	c. 40% of the total
Range 3	c. 40% of the total
Range 4 highest values	c. 10% of the total.

If some of the samples have a concentration less than the detection limit, this is indicated by the signs "<" in the legend of range 1 (see figure 2). Note that in figure 2 two different values of the detection limit are given: the detection limit varies between these values. The distribution is:

Range 1 detection limits		
Range 2 lowest values	c. 60%	of the total with concentration larger than the detection limit
Range 3	c. 30%	"
Range 4 highestvalues	c. 10%	"

The maps clearly show the regional distribution of the groundwater-quality: marine effects in the chloride concentration (fig. 1) and the clay-peat soils in the province South Holland (fig. 2) high Kjeldahl nitrogen content.
Another example of the use of the maps: In accordance with the results of the statistical analysis a very high nitrate concentration can be found in the sandy areas with a high factory farming activity.
It is of course also possible to substitute other limits for the ranges, for example to determine what wells exceed a certain norm.
Not only single values can be presented in these maps, but also diagrams such as stiff diagrams can be plotted.

5 Conclusion

Two methods of analysing the data of the national monitoring system have been presented:
a statistical approach to relate several properties of the locations of the wells with the measured concentrations and a geographical approach to distinguish regional effects. These two methods supplement eachother and make the monitoring system a powerfull instrument in

research of groundwaterquality.

References

van Duijvenbooden, W. 1979. Groundwaterquality monitoring network. Quaterly report 19. National Institute for Water Supply (RID) Leidschendam, The Netherlands 4 pp.

van Duijvenbooden, W. 1981. Groundwaterquality in The Netherlands - collection and interpretation of data in: Quality of Groundwater, W. van Duijvenbooden, P. Glasbergen, H. van Lelyveld (ed.). Elsevier Amsterdam, Oxford - New York pp. 617-628.

Mann, B.H. and D.R. Whitney 1947. On a test of whether one of two random variables is stochastically larger than the other. Annals of Math. Stat. 18 : 50-60.

Wijvekate, M.L. 1979. Verklarend Statistiek. Het Spectrum Utrecht/ Antwerpen fifteenth edition pp. 158-163.

PRE-PROCESSING AND STORAGE OF WATER LEVEL DATA RECORDED AT FIXED TIME INTERVALS

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Abstract

Water level data recorded by automatic equipment may contain errors of different kind and origin. Before such data can be processed safely in order to serve as a basis for conclusions, errors should be detected and -if possible- corrected. Only if no detectable and correctable errors remain, the data should be stored in a system of permanent files that can be accessed conveniently by programs to actually process the information. Those files should not be longer than necessary, should cause as little read time loss as possible, whereas data retrieval of either specified dates or specified recorder locations should be equally simple.

A system of programs and subprograms written for error detection, correction and producing efficiently organized permanent data files is described.

Resumé

Les observations de la nappe phréatique fournies par des appareils enregistreurs contiennent souvent des erreurs d'origine differente. Avant que ces observations soient utilisées pour en tirer des conclusions dignes de foi, il faut rattraper et, si possible, corriger les erreurs. Après qu'il ne reste plus des erreurs corrigibles, les observations doivent être stockées dans des fichiers permanents qui sont

facilement accessibles pour des programmes afin de fournir l'information nécessaire. Il faut que les fichiers soient d'une longueur réduite et n'exigent qu'un temps limité à lire, alors que retrouver des données des dates ou des lieux spécifiés doit être simple.
Un système de programmes et de sous-routines établis pour retrouver et corriger des erreurs et pour produire des fichiers permanents des données est présenté.

1 Introduction

A single channel digital recorder with recording time interval of 15 minutes and a recorded value of 4 decimal digits will produce some 175 kilobyte of data per year if standard ASCII code is used. The annual production of a small observation network of 10-20 recorders will therefore already be at megabyte level.
Manual handling of such a data stream is virtually impossible and the availability of a computer system of reasonable size and speed is essential. However, even if such a system is available a number of problems remains to be solved. The two major ones are

- detection and -if possible- correction of errors in the data before they are added to the actual data collection
- the accessibility of the data collection

The Dept. of Land and Water Use of Wageningen University has been operating a groundwater level observation network of about 150 observation wells since 1972. At present 14 wells have been equipped with punched tape recorders. The types used are Fischer & Porter and Leupold & Stevens, type 7000. The sensing system of both types is a float and counterweight, the analog to digital conversion is mechanical and the timing device is either an electromechanical clockwork or an electronic quartz-controlled timer. Registration is at fixed time intervals.
The data substrate is a 16-trace punched tape with four BCD-coded digits. The registered values are in millimeters and their range is 0 - 9999. Before this tape can be read, it is converted to a standard 1" punched paper tape,code ASCII or Elliott, or data cassette (ASCII). The latter is more reliable than the paper tape and has almost replaced

it.
From the beginning the need was felt of a good set of programs to overcome the problems mentioned above. At first, the system described by Herfst and Warmerdam (1972) and Herfst (1973) was used. Although this was an important step forward compared with previously available techniques, the system was not satisfactory because it only provided an error detection procedure but no filing system and the necessary corrections in the data had to be made manually on the paper tapes.
For this reason an own system was tried in 1977 and improved during later years. It includes
- an improved error detection and data listing procedure
- conversion of data to a very compact form
- an error correction program
- a data filing system with associated access software.

It should be emphasized, that no error recovery procedure can replace field precautions, field checks and maintenance of recording equipment. Making decisions on error corrections cannot (yet?) be left over to a machine. Good field check reports are of prime importance in judging computer produced error reports.
Data processing should only start after the data have been thoroughly checked and stored in an efficiently organized data collection. The latter activities will be referred to as pre-processing.

2 Error detection

Three types of error symptoms can be detected with certainty (Herfst and Warmerdam, 1972)
- the number of values (the correct number of values is computed using date/time of first and last punch and the recording time interval)
- code errors (only numbers and separation marks should occur; any other character must therefore be deformed code)
- word length errors (every "word" consists of 4 digits and a separation mark)

A detection program based on these symptoms will not reveal all errors in the data. For example, a lost bit in the 16-trace BCD-code will not cause a code error. It will still produce a numerical data value, in

most cases with an unexpectedly high numerical difference with its neighbours in the series. The same will happen if e.g. the float has got stuck temporarily.
For this reason a fourth test was added to the procedure. It is a test on difference between successive values. Differences exceeding preset limits will cause an error message to be issued by the error detection program. Such a test is effective because groundwater levels usually fluctuate slowly and groundwater level observation time series are highly autocorrelated if the registration interval is kept sufficiently short.
However, not every "jump" detected means an error. So detection of a jump should be seen as a warning rather than as a certain error.
The decision on which limits should be acceptable depends on both the conditions at the recording location (aquifer, climate) and the recording time interval: the shorter the interval, the smaller the differences. Such a decision means a trade-off between the accepted number of "false alarms" and allowable inaccuracies in the data. Increasing the limit will reduce the number of "false alarms", but will also reduce the sensitivity of the test. Reducing the registration time interval lenght will cause an increase of the efficiency of the test, but will also cause an increase of the data production. In most cases the latter should be preferred, because redundancy in the data may be very helpful in error recovery and it is always possible to reduce the number of data later. If some optimization of the registration time interval is done (the theoretical basis for this is given in several publications; a good example is Ogink (1974)),there should be no serious problems. For average Dutch conditions limits of -9 and + 9 mm in combination with 15-minute or 1 hour registration time intervals were found to give very satisfactory results.

During the error detection procedure the data are rewritten in a temporary storage file in a very compressed form in order to increase access speed and to reduce disk storage space. The full output produced by the error detection program consists of:

- an error detection report that includes the exact locations where errors in the data have been found (errors include "jump" warnings)
- a full data listing with error locations marked
- a temporary storage file that contains the data in a compressed form.

The full procedure if performed by one program named PTQ11. The report and the data listing, together with field instrument check reports are used in the error correction procedure. Before the correction procedure will be discussed, some explanation concerning the compressed data storage will be given.

3 Storage of data by program PTQ11

All our programs have been written in FORTRAN for use on a DEC-1090 system. This is a 36-bit system, which means that one word consists of 36 bits. Normally one data value is stored in one word, either as 4 ASCII characters (comprising 28 bits) or as its real numerical value in binary. However, all integer values from 0 - 9999 can be defined by 14 bits (a 14-digit binary number is in the range 0 - 16383).
PTQ11 stores 2 data values in one 36-bit word, leaving 8 bits unused for data storage. These 8 bits are not lost, but can be made very useful. The reason for this is, that groundwater levels often change very little during relatively long periods. If they are registered at fixed time intervals, this will result in data series with shorter or longer rows of equal values. The information contained in such a row can be described by 2 values: the observed value and the number of observations. The latter is stored in the remaining 8 bits. One 36-bit storage word contains therefore:

- 2 successive data values (2 x 14 bits)
- the number of successive occurrences of the combination of the two data values (8 bits)

At the beginning of the tempory file the following identification information is stored:

- station code
- data and time of first and last observation
- length of registration time interval

Space reduction obtained by this method depends on the the frequency of successions of equal data in the set. A well with rapid response to e.g. rainfall will produce longer files than a slowly responding well. Thus the file length is somewhat proportional to the actual information content of the data. In practice space saving was found to range from

60 to 95% and improvement in access speed was found to be up to 50%. However, three remarks should be made about this way of storing data:

- File access by WRITE or READ statements preceded or followed by the necessary coding/decoding in a program should be replaced by a call to a subroutine that performs all the necessary action. Such a subroutine is preferaby organized in such a way, that it can be used as if a simple one-value-per-word file were accessed.
- FORTRAN does not provide for direct bit- or bytewise access within words. If a byte-manipulating routine is written in FORTRAN it has to make use of arithmetic, making it far from efficient. FORTRAN-compatible routines written in assembler should be preferred.
- several variants on the storage system described are possible. It depends on data range, system word size and personal taste how it is worked out in individual cases.

4 Error correction

The basis for the error correction is the error detection report mentioned in 2. Decisions on which operations are to be performed should be based on that report, on field check reports about the recording instrument(s) an on a thorough knowledge of error sources and their possible effects on the data produced by the instrument(s). The program written for this purpose is called PTQ11A. Its input consists of

- the compressed data file produced by PTQ11
- commands from the user's terminal (the program is interactive)

The output is a disk file that contains the corrected data in the form of a simulated ASCII or Elliott coded punched tape. This file can be used only as input to PTQ11, so the user is forced to let the data go through the error detection procedure once again before they can be added to the data collection. With PTQ11A it is possible to

- remove data
- insert data
- modify data by addition, subtraction or replacement
- restore lost bits
- edit an output file

Together with a command that specifies the kind of action the program needs information as to where in the data it is expected to perform the action. This information is given in the way it is produced by PTQ11 as either value number (first value is nr. 1, second is 2, etc.) or date and time.
Operations on both individual values and series of successive values can be specified in one command set.
The combination of PTQ11 and PTQ11 has been found to be a powerful tool in tidying up recorder field data.

5 The data collection

Tidying up data is of relatively little use if the resulting data collection is not efficiently organized. A well organized data collection meets the following requirements:

- it is easily and quickly accessible
- its size is not larger than necessary
- it can contain all the available data
- future data of existing stations can be added
- new stations can be added

5.1 How the collection is organized

The collection consists of named data files, one per year, per station and length of timing interval.
The file name contains and is fully determined by year, station code and timing interval length.
The data in the file are arranged in normal chronological order and in the same compressed form as in the output file of PTQ11. Data in a file start on January 1st, one interval after 0:00 hrs. and end on December 31st, 24:00 hrs. The position of the data in the file is determined by month, day, time and timing interval. No user intervention is needed to find data if station, date, time and -preferably- timing interval length are known.
A data collection organized in this way meets the last four of the five

requirements mentioned above. The file names used by the author have

- 4 character positions for the station code (gives 1679616 possible alphanumeric combinations)
- 2 positions for the year (enough for a century if only numerical combinations are used)
- 3 positions for the timing interval, 2 numerical and 1 alphabetical. The alphabetical position indicates whether the interval is in hours or in minutes, the 2 numerical positions give the number of hours or minutes.

Example: the file 172080.M15 contains 15-minute data of the year 1980 from station 1720.

5.2 Access to the data collection

The file name is constructed using station code, date and timing interval length by a readily available function subprogram RCFNAM. This enables the calling program to open theproper file. The file is then read using another subprogram called RRC. Each call to RRC returns one data value to the calling program. This sequential access is the simplest way to use RRC.
A positioning entry POSRC in RRC can be used to position a file to any wanted value specified by observation date and time. This is done by decoding theword part that contains the number of successive occurrences of the data stored in the same word. A more random-like access has been made possible in this way.
RRC can handle as many simultaneously open files as the computer system permits, thus facilitating simultaneous data retrieval for several stations.
Write access is possible by a writing subroutine called WRC. After the file has been opened data are transferred to WRC one by one in chronological order. WRC takes care of proper file organization. It is the complement of RRC.
These subroutines can be called by FORTRAN programs in the normal way. In this way the first condition (easy and quick access) mentioned above has also been met.

5.3 Adding data to the collection

Data are added to the collection from the compressed output files written by PTQ11. The program that performs this kind of action (PTD12) has a built-in error check procedure similar to the one in PTQ11, except for the test on differences between successive data values which is not performed. Updating (or creation) of data files only takes place if no errors have shown up. Otherwise PTD12 will refuse to do its work. This is an additional safety measure to protect the data collection from being updated by poor quality data. PTD12 makes use of RCFNAM, WRC and RRC. It does not need any user intervention as it obtains all its relevant information from the PTQ11-produced file.
Another program (PTD13) writes long interval files using short interval files as input. This makes it easier to use data from long interval stations together with short interval stations. PTD13 uses all subprograms mentioned in 5.2.

References

Herfst, B.A. and P.M.M. Warmerdam 1972. Hydrologische gegevens op ponsband. Laboratorium voor Hydraulica en Afvoerhydrologie, rapport no. 2, Wageningen, 94 pp..

Herfst, B.A. 1973. Automatic collecting and processing of hydrologic data from catchment areas. Comm. Hydr. Res. T.N.O., Automatic Processing of Hydrological Data, Proceedings and informations nr. 17, pp. 11 - 25.

Ogink, H.J.M. 1974. Determination of an effective sampling interval for hydrologic time-series, VITUKI, Budapest, 100 pp.

THE TREATMENT OF STATISTICALLY NON-HOMOGENEOUS HYDROGEOLOGICAL DATA

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Abstract

Statistical methods are presented to treat statistically non-homogeneous hydrogeological data. These methods enable us to verify the hypothesis of homogeneity of data as well as to divide non-homogeneous data into statistically homogeneous groups. Some examples are given to illustrate the application of methods to treatment of hydrogeological data.

1 Introduction

The treatment of statistically non-homogeneous hydrogeological data is known to have a member of peculiarities. For instance, non-homogeneous hydrogeological data are usually divided into statistically homogeneous groups of data, each of them being treated separately. However, such division is not always possible to perform visually. If the data to be treated characterize non-uniform natural objects with indistinctly expressed hydrogeological boundaries, special statistical methods should be used to perform this division. To formalize this problem, assume that a certain natural object has been sampled in n points so that in each sampling point m of its geological characteristics ($m \geq 1$) have been measured. Denote the vector of these characteristics in k-th sampling point by X_k and consider a sequence $X^{(n)} = (X_k)$, $1 \leq k \leq n$ of the vectors obtained. If this natural object is non-uniform, then, according to statistical approach, the distribution functions of vectors X_k change.

If these changes may be represented as discontinuous changes of the parameter values of vector distribution functions, we arrive to the following statistical model.

Let $F(x,\theta)$ be a distribution function, which depends on the parameter θ, where θ belongs to an open set Θ from R^1. We consider the case, when the vectors $X_1,\dots,X_n$ are independent; they have independent components and their distribution functions

$$P(X_k \leq x) = \sum_{i=1}^{r+1} F(x,\theta_i)\ I\ (\sigma_{i-1} < k \leq \sigma_i) \qquad (1)$$

where $I(\cdot)$ = indicator of the set $(\cdot)$

σ_i = change-point of the distribution functions, $1 \leq i \leq r$

$\sigma_o = 0$

$\sigma_{r+1} = n$

In this model, the change-points of distribution functions $\sigma_1,\sigma_2,\dots,\sigma_r$ are unknown as well as the values $\theta_1,\dots,\theta_r$ of the parameter θ.

The hydrogeological data homogeneity testing in such a model is equivalent to the checking of the hypothesis H_o: $\{\theta_1 = \theta_2 = \dots = \theta_{r+1}\}$ against the alternative H_1: {there exists i, $1 \leq i \leq r$, such that $\theta_i \neq \theta_{i+1}$ }.

The division of the data on the homogeneous groups consists in the extimation of the parameters σ_1 where $1 \leq i \leq r$.

2 An investivation of statistical model

The model (1) was investigated both by mathematicians and geologists. The papers by Page (1955), Gardner (1969), Hinkley (1970), Sen et al. (1975), Pettitt (1970) should be pointed out among the mathematical papers. Among the geologists, a great contribution to investigation of this model has been made by Rodionov (1968). The results, which will be given, generalize the results of the above mentioned authors.

2.1 The case of only one change

Let only one change of the θ parameter value may occur, i.e. $r = 1$ in the model (1). Consider the tests to verify the hypothesis H_o and estimates of the change-point σ_1, which are based on the likelihood function of vectors $X_1, X_2, \ldots, X_n$. Let $f(x,\theta)$ be the density of the distribution function $F(x,\theta)$ with respect to any σ-finite measure μ. If the change-point $\sigma_1 = S$ and unknown parameters θ_1, θ_2 are substituted by maximum likelihood estimates of $\hat{\theta}_1(s)$, $\hat{\theta}_2(s)$, then the likelihood function of the vectors

$$L(s;X_1,\ldots,X_n) = \sum_{i=1}^{s} \ln\{f(x),\hat{\theta}_1(s))/f(x,\hat{\theta}_1(n))\} +$$

$$+ \left| \sum_{i=s+1}^{n} \ln\{f(x,\hat{\theta}_2(s))/f(x),\hat{\theta}_1(n))\} \right| \tag{2}$$

One of the following statistics may be used to verity the hypothesis $H_o: \{\theta_1 = \theta_2\}$:

$$T_1 = \sup_{\alpha n \leq s \leq (1-\alpha)} \{2L(s;X_1,\ldots,X_n)\}^{1/2}$$

$$T_2 = \sup_{\alpha n \leq s \leq (1-\alpha)} \{2s(n-s)/n^2\} L(s;X_1,\ldots,X_n)$$

$$T_3 = \sum_{i=1}^{n} 2L(s;X_1,\ldots,X_n)/n$$

$$T_4 = \sum_{i=1}^{n} (2s(n-s)/n^2) L(s;X_1,\ldots,X_n)$$

where α = any fixed number, $0 < \alpha < 1/2$.

The large values of the statistics mean that a change in parameter values takes place. To determine the critical values of these statistics, the results of proposition 1 may be used.

Denote the distribution functions of the m-dimensional analogs of Kolmogorov-Smirnov, Anderson-Darling and omega-squared statistics, $y \in R^1$ by $K_m(y)$, $A_m(y)$, $\omega_m^2(y)$.

PROPOSITION 1. Let the density $f(x,\theta)$ and $\delta f(x,\theta)d\mu$ be twice differentiated with recpect to θ; $\hat{\theta}_1(s)$, $\hat{\theta}_2(s)$ be maximum likelihood estimates consistent uniformly over $s \in [\alpha n,(1-\alpha)n]$; there exist the function $R(x)$ such that

$$\sup_{|\theta-\theta'|<\delta} |\partial^2 \ln f(x,\theta^1)/\partial\theta^2| \leq R(x), \ E_\mu R(x)f(x,\theta) < \infty$$

where E_μ = averaging with respect to measure μ

δ = any sufficiently small positive number

Then

$$P(T_1 \leq y) = P(\sup_{1\leq u\leq(1-\alpha)^2/\alpha^2} ||W_u/U^{1/2}|| \leq y)$$

$$P(T_2 \leq y) = K_m(y^{1/2})$$

$$P(T_3 \leq y) = A_m(y)$$

$$P(T_4 \leq y) = \omega_m^2(y)$$

where W_u = standard m - dimensional Wiener process

$||\cdot||$ = Euclidean norm in R^m

The value σ_1 of S, which maximizes over $S \in [1,n]$ the values of $L(s;X_1,\ldots,X_n)$ or $\{S(n-s)/n^2\}$ $L(s;X_1,\ldots,X_n)$, is often considered as estimates of the change-point σ_1. If the statistics T_1, T_3 are used, then it is convenient to apply $L(s;X_1,\ldots,X_n)$ to estimate σ_1. When T_2, T_4 are used, $\{s(n-s)/n\}$ $L(s;X_1,\ldots,X_n)$ is usually applied.

2.2 The case of more than one change

In this case, the following algorithm may be used for treatment of non-homogeneous data:

a) at first, we verify the hypothesis H_o by means of the results of 2.1;

b) if this hypothesis is rejected, we estimate one of the change-points with the help of the results of 2.1;

c) if a change-point σ_i, is estimated, we analize the first part of observations, i.e. $X_1,X_2, \ldots X_i$ where $_i$ is the estimate of $_i$, and the second part of observations $X_{i+1}, \ldots X_n$, according to a and b.

So, the sequence $X_1, X_2, \ldots, X_n$ of observations is separated into two parts, then into three parts, four parts and so on, until we get the series of statistically homogeneous groups of data.

3 Investigation of natural models

Treatment of hydrogeochemical data becomes increasingly important to hydrogeology. These data are widely used for solving numerous problems of the formation of the chemical composition of groundwater under natural and disturbed conditions. Hydrogeochemical data include information on total dissolved solids content, macrocomponents, microcomponents, and specific components (hardness, aggresssive character, of water etc.), which deteriorate groundwater quality.

It is imperative that hydrogeochemical data are treated together with geological and hydrogeological information, since the hydrogeochemical situation depends on geological and hydrogeological conditions. The chemical and mineralogical composition of water-bearing rocks and their permeability properties are the most important data. First, rocks are the sources of the matter composition of groundwater; second, they are factors influencing the kinetics of the physicochemical processes of mass exchange in the water-rock system.

The above data to be treated are largely non-homogeneous under natural and disturbed conditions. For instance, rocks are homogeneous as sources of the matter composition of groundwater only if they are: (a) quartz sand and sandstones, (b) pure limestones, (c) pure gypsums or anhydrites, (d) pure halites or other single salts. Permeability homogeneity of rocks is a rare phenomenon, while permeability non-homogeneity of rocks is often encountered in nature. It is equally characteristic of non-homogeneous and homogeneous rocks as far as their mineralogy and geochemistry are concerned. The effect of the rock permeability non-homogeneity on the processes in the water-rock system manifests itself through water seepage rates. The variety of seepage rates under rock permeability non-homogeneity conditions results in the sharp and frequent variation of the concentration of water components.

Homogeneous and non-homogeneous natural media are characterized by specific physicochemical processes and hydrogeochemical regularities.

A limited number of processes is typical of homogeneous media, they are: hydrolysis in the groundwater-quartz sands and sandstones system, carbonic acid leaching in the water-limestones system, dissolution in the gypsums, anhydrites, single mineral salts system. Hydrogeochemical regularities consist in the increase of the total dissolved solids content of water in the direction of flow at the expense of a limited number of constantly acting components.

In non-homogeneous media, some physicochemical processes commonly occur simultaneously. Hydrogeochemical regularities consist in: (a) an increase in the total dissolved solids content of water in the direction of flow at the expense of periodically or constantly acting components; (b) a change in the ratio of components as the total dissolved solids content increases.

4 Types of natural models

In nature, certain combinations of groundwater matter composition sources and factors activating these sources determine the formation of groundwater, related to some genetic hydrogeochemical types (or subtypes). The groundwater of practically each genetic hydrogeochemical type is noted for a complex composition and the considerable statistical non-homogeneity of the distribution of individual components. Water subtypes, distinguished within genetic hydrogeochemical types of water, are characterized by a complex composition, however, they often retain the statistical non-homogeneity of components distribution. The distribution of components depends on the temperatures, pressures, and depths of occurrence of the groundwater of a subtype in different areas and on the indistinctly expressed transitions of some lithologic-mineralogical varieties of rocks into others.

Thus, we face the problem of defining the boundaries of distribution of groundwater, related to various genetic hydrogeochemical types (or subtypes).

The examples of the most general genetic hydrogeochemical types are:

1) waters of complex physicochemical interaction with crystalline and sedimentary terrigeneous and carbonate rocks, occurring in the active water circulation zone of humid areas, with dissolved solids contents

ranging from less than 0.1 to 1 g/l and more, of mixed component composition, with dominating HCO_3 and CO_3 ions;

2) waters of sea influence, occurring in sea areas, with a dissolved solids content of over 0.5 g/l, with dominating chlorine and sodium;
3) waters of dissolution of sulphate rocks and sulfuric acid leaching of rocks, occurring under different hydrogeological and climatic conditions, with dissolved solids contents ranging from 2 to 100 g/l and more, with dominating SO_4 and Cl ions;
4) waters of dissolution of halogen rocks, occurring under different hydrogeological and climatic conditions, with a dissolved solids content of over 50 g/l, with dominating chlorine and various cations;
5) waters of complex physicochemical interaction with terrigeneous and carbonate rocks and of concentration, occurring in marginal parts of artesian basins, with dissolved solids contents ranging from over 5 g/l to 50 g/l and more, with dominating chlorine and sodium;
6) waters of metasomatic substitutions in terrigeneous rocks and of leaching by solutions of carbonate rocks with a higher ionic force, occurring in deep parts of artesian basins, with a dissolved solids content of over 20 g/l, with dominating chlorine and varying cations;
7) waters of mixing, occurring in zones of tectonic disturbances, affecting unconfined and confined aquifer systems in mountain-fold areas, crystalline massives, on platforms with dissolved solids contents ranging from less than 1 to 200 g/l and more, with varying cations;
8) waters of continental salinization, occurring in unconfined aquifer systems of arid areas, ranging from fresh to brackish, with a varying cation composition;
9) waters, forming under the influence of man-made factors.

The solution of the problem, involving the demarcation in respect of HCO_3, SO_4, Cl, is the most important to the groundwater of genetic types 1-5 and 8 and in respect of Na, Ca, Mg of types 2 and 6. The waters of types 7 and 9 occupy a special place, in these cases the problem should be solved in respect of all main anions and cations.

5 An example

The proposed method has been used for treatment of data on the chemical composition of the water of deep aquifers in the Southeastern Russian platform. The data might be non-homogeneous due to the presence of a fault crossing the area under study (the water type 9).
In treatment, the data were ordered along a profile crossing the fault. Let us consider an example of such treatment for Na. The Na content values in 29 sampling points are presented in the form of a curve in Figure 1.

Figure 1 The Na content in the groundwater of the aquifer system under study

Assuming that the Na distribution follows a normal law, the values of $L(s;X_1,\ldots,X_{29})$ were calculated, using the proposed method (Figure 2).

The value of $T_1 = 3.098$ was also estimated. The critical value of the statistics at the level of 0.1, determined from the results of Proposition 1, occurred to be equal to 2.98 and exceeded the critical value. Therefore, the initial totality of data is non-homogeneous, it consists of two homogeneous parts as a minimum, and the boundary between them is $\hat{\sigma}_1 = 9$. Then, the homogeneity of two successions $X_1,\ldots,X_{10}$ and $X_{11},\ldots,X_{29}$ were checked up; this check up showed that these totalities of data are homogeneous. Thus, the performed treatment showed the significant effect of the fault on the Na content values that seems to be due to interflow.

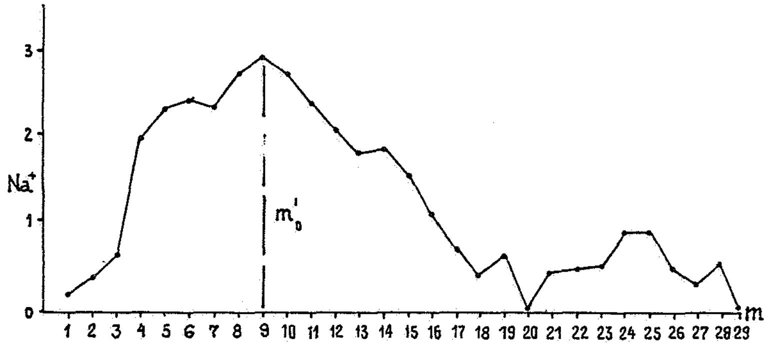

Figure 2. Values of $L(s;X_1,\ldots,X_{29})$ of the likelihood function

References

Gardner, L.A. 1969. On detecting changes in the mean of normal variates. Ann. Math. Statist., 40:116.

Hinkley, D.V. 1970. Inference about the change-point in a sequence of random variables. Biometrica, 57:1.

Pettitt, A.N. 1980. A simple cumulative some type statistics for the change-point problem with zero-one observations.

Rodionov, D.A. 1968. Statistical methods for demarcation of geological objects according to a complex of indicators. Nedra, Moscow (in Russian).

Sen, A.K. and M. Srivastava. 1975. On tests for detecting change in mean. Ann. Statist., 3:98.

COMPUTERIZED PROCESSING AND INTERPRETATION OF PUMPING TEST DATA

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Abstract

A computer program for analysing pumping test data has been developed at the Geological Survey of Denmark. The program is interactive and can be used with a minimum knowledge of computers. The programming language is BASIC and the program is specially designed for a Tektronix 4054, a graphic screen containing a 32 K-bytes micro-computer with a refresh (dynamic graphics) option. This option is very useful during interpretation of the data. It allows one to perform type curve matching directly on the screen by moving the chosen type curve to the position where it gives the best fit, in a manner similar to traditional manual type curve matching. Various forms of type curves based on analytical solutions are available for the interpretation. Plots may be conveniently reproduced as semilog and log-log plots either on the screen or printed by a small flat-bed plotter. The capacity of the machine is limited however, and in order to use the numerical models for the interpretation of data access to a computer-center may be desirable.

Resumé

Un programme informatique pour l'analyse de données obtenues lors des essais de pompage a été développé au DGU. Le programme est interactive et peut être utilisé avec très peu de connaissance en informatique. Le langage utilisé est le BASIC et le programme a été spécifiquement

développé pour le Tektronix 4054, un écran graphique lié à un 32K-bytes micro-ordinateur avec option de rafraîchissement. Cette option permet de faire, directement sur l'écran, la paire de courbes types en déplaçant la courbe choise à la position où elle donne le meilleur ajustement, comme dans la méthode d'ajustage manuel. Différentes courbes types basées sur des solutions analytiques sont disponibles pour l'interprétation. Les graphiques du type semilog et log-log peuvent être produits sur l'écran ou sur une copie en papier facilement reproductible.

1 Introduction

Due to the time-consuming prosesses of manual plotting and interpretation of pumping test data, we had for a rather long time studied the possibilities of computerizing this part of our work. The demand was primarily to develop a system that allowed for an easy interactive interpretation process and was able to reproduce the results quickly and with fairly good quality.

The computer facilities of the Geological Survey of Denmark include a graphic screen containing a 32 K-bytes micro-computer with a refresh option (Tektronix 4054). The programming language is BASIC extended with special graphic commands which facilitate the creation of various plotting routines. In combination with a small flat-bed plotter and a hardcopy unit, the entire system is a very powerful tool when solving minor computer and plotting tasks. The limitations in the application of the system are primarily associated with the rather low memory capacity and the reduced calculation effectivity due to the programming language and the lack of library programs.

As result of increases in the size of the program during its development, the capacity of the machine soon became insufficient to contain the program in its full length. It has therefore been necessary to divide the program in two main parts; the data storage part and the data preparation part, the latter is sub-divided into a semilog-plot section and a log-log section. Each part of the program is stored on tape as separate files and only the program-sections used are placed in the machine when required.

The program is interactive and asks for each command to be read in. This gives the user the advantage of being able to run the program with only a minimum knowledge of computers.

2 Data input

Data input presently takes place as a manual keyboard operation. Due to subsequent handling of the data, the program initially asks whether it is dealing with a pumping well or an observation well, and whether it is drawdown data or recovery data. In the case of drawdown data, one has the option of putting in variations in pumping capacity versus time. Data from an unspecified number of observation wells related to the same pumping test can also be stored, but due to the low machine capacity, there is a limit to the number of datasets that can be handled. In practice therefore, a maximum number of observation wells stored in the same file should not exceed six.
Datasets from the pumping well and the corresponding observation wells are stored in different files. Before storing the input data on tape, it is possible to list the drawdown/recovery data for each well and correct, delete or add data, if desired.

3 Data preparation

The data processing depends mainly on what kind of output form is wanted: 1) Semilog time/drawdown plots, preferably used in connection with pumping well data and 2) log-log time/drawdown plots, preferably used in connection with observation well data. Semilog distance/drawdown plots, used for comparison of different wells at a given time, are not yet available.
The program allows the data to be corrected for decreasing thickness of the aquifer or groundwater fluctuations caused by external circumstances i.e. barometric effects, change in infiltration rate etc. The fluctuations have to be stored as ordinary observation well data and can be subtracted from corresponding well data with a weight factor between 0 and 1. Both corrected and uncorrected data can be stored on

tape and later be retrieved for future studies.

3.1 Semilog plots

The program automatically computes the scale of the x, y - axes on the basis of the data set range. The drawdown/recovery axis will thus be divided into subintervals of 0.1 meter, 1.0 meter or 10 meter lengths depending on whether the difference between the maximum and minimum drawdown/recovery is less than 1 meter, between 1 and 10 meters or above 10 meters. In other words, if the drawdowns are in the range from 3.5 to 6.3 meters, the axis will be divided into 1 meter intervals from 3 to 7 meters. The time axis normally consists of 5 decades, ranging from 1 min to 100.000 min (10 weeks) or from 0.1 min to 10.000 min depending on the observation period. For special cases where there are data both below 1 min and above 10.000 min, the program is also able to choose a 6 decade plot. For pumping well data, drawdown and recovery data can be plotted and interpreted in the same diagram. In addition, recovery data can be plotted as a function of $t/(t+\Delta t)$ which is especially useful when the recovery time, Δt, is of the same order of magnitude as the pumping period, t.

3.2 Log-log plots

The x and y-axes are divided into 5 and 4 decades, respectively and as is the case for the semilog-plots, the scale of the axes are computed by the program. The drawdown/recovery axis is normally from 10^{-3} to 10^{1} meters and the x-axis can be chosen as an ordinary time axis ranging from 10^{0} to 10^{5} min, as a t/r^2 axis, or for recovery data, as a dimensionless $t/(t+\Delta t)$ axis. Data from op to 5 wells can be plotted in the same diagram. Each well has its own signature (dots, triangles etc.) and is successively listed with its DGU file no. in the corner of the diagram. These multi-well plots are very useful when the homogenity of the aquifer is to be examined. Type-curve interpretation is accomplished however, when data from only one well is plotted.

3.3 Interpretation of data

The most essential quality of the system is the refresh option: the ability to move figures all around the screen by means of two thumb-wheels, (one for the x and one for the y direction), and to fix the figure wherever desired. The refresh option is utilized in different ways, depending on whether the plot is semilog or log-log. For the semilog plots, it is possible, as described by the traditional Jacob method (the modified Theis equation), to let the program calculate the best straight line fit to the data-points and compute the transmissivity value. The straight line is calculated on the basis of the wellknown expression for logarithmic regression. The fixing of the data-interval to be analysed is denoted by a vertical line, generated by the program, when the machine is in refresh mode. This line can be moved (horizontally) and fixed at the extreme points of the specified interval. When the interval is fixed, the regression line is drawn and the transmissivity value computed. Several different intervals can successively be tested, for both drawdown and recovery data, and the last computed transmissivity value for each drawdown and recovery is always stored. If variations in the pumping rate have occured, the calculated mean capacity for the specified interval is used for the transmissivity calculation.

When plotting log-log diagrams, the determination of the line of best fit no longer takes place as an automatic regression calculation. Since the system allows objects generated on the screen to be moved however, a visual technique, just like the traditional, manual type-curve matching procedure was developed. The specified type curve is calculated by the program when the machine is in refresh mode. The curve can then be moved by the thumbwheels and fixed where it seems to form the best fit. Simultaneously, the hydraulic parameters, such as transmissivity, storage coefficient etc., are computed and listed and the next trial can be performed.

The type curves are computed on the basis of analytical solutions to the groundwater flow equation. The applied solutions include especially the general expression for drawdowns in confined aquifers with leakage, which in dimensionless form can be expressed as the "well function"

$$W(u,\beta) = \int_u^\infty \exp(-y-\beta^2/4y)dy/y$$

If the leakage factor, β, is set to zero, the "Theis well function" is obtained. Because no explicit analytical solution or series expansions exists for the general expression for the "well function", it has been necessary to solve the semi-infinite integral by numerical methods. A simple formula, the Simpson's rule, used for sub-intervals of increasing length and in connection with a converging criterion, has been found the most expedient in solving the problem.
The expression derived by Hantush (1964) is applicable for solving anisotropic aquifer conditions in partially penetrating wells. The dimensionless drawdowns can be written as:

$$s^* = W(u,\beta) + f$$

where

$$f = C_1 \cdot \sum_{n=1}^{\infty} 1/n\ (\sin nC_2 - \sin nC_3)\cos nC_4 \cdot W(u,\beta')$$

and C_1, C_2, C_3, C_4 are constants depending on the aquifer thickness and the position of the screened interval in the pumping well as well as in the observation well. β' is, among other things, dependent on the ratio between the radial and the vertical hydraulic conductivity, K_r/K_z.

The possibility of interpreting pumping test data by other type-curves than those discussed above is currently being studied. This especially concerns two expressions. One is the short time solution, $H(u,\beta)$, for the flow equation in leaky aquifers, which particularly takes the storage in semipervious layers into account. The other is the one - dimensional solution, $D(u,\beta)$, for the flow equation in leaky aquifers. It is especially useful when interpreting data from channel reservoirs such as esker aquifers.
The calculation time is surprisingly low even for a micro-computer like the Tektronix 4054, and does not appreciably affect the type-curve plotting when the $W(u,\beta)$-expression is regarded. The conditions are a

somewhat different for the partial penetration expression and under unfavorable circumstances, the summation term can converge so slowly that the number of summations may reach 30-40 steps. The calculation time under these conditions obviously affects the drawing time, but this problem is generally acceptable.

The method of interpreting data described above gives more awareness of how the data is used than an exact regression calculation permits. Furthermore, a regression calculation would tend to become very complicated if the expressions are on the level of the partial penetration expression. The calculation time, and especially the programming work, would be considerable.

3.4 Data output

In connection with each data-plot, it is also possible to generate a text-plot which can list a number of specifications which may be important for a later identification and investigation of the pumping test. The following information is listed: Name of the project, geographical location of the test, DGU file no. of the pumped well, date and time of the pumping period, pumping capacity, DGU file no. of the specified wells, distance from the pumped well and computed hydraulic parameters. If there is any change in the capacity of the pumped well, there will also be a plot made showing the pumping capacity versus time. Different types of plots are shown in the figures 1-3.

These plots are executed by a small flat-bed plotter that is directly connected with the machine. Because of the size of the plotter, it is necessary to perform a minor manual exercise if a "total" plot is desired. A text-plot and a data-plot have to be plotted separately but by means of automatically fixed corner-marks on each plot, the parts can be cut together to the size of A3 and easily transformed to A4-size when copied. As seen in the examples, figures 1-3, the plots are of rather good quality and can be incorporated in reports and publications.

PUMPING TEST

SOLRÖD VANDVÄRK - SOLRÖD

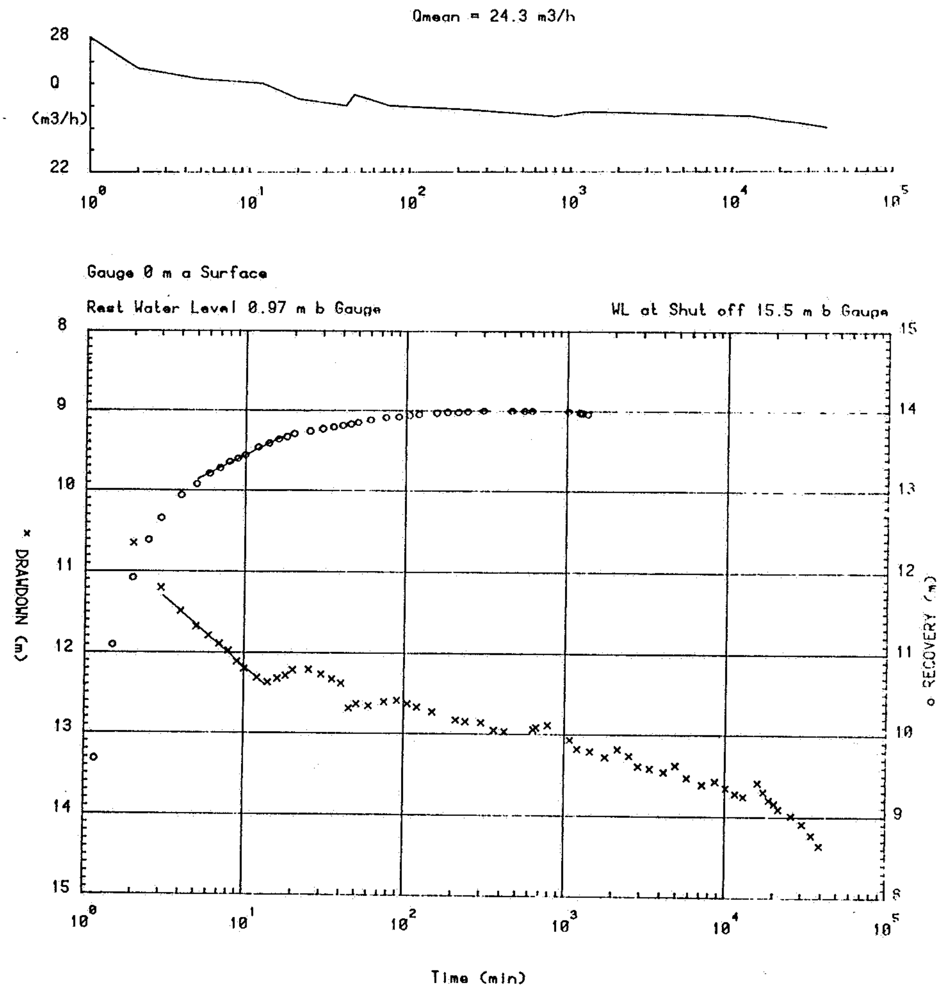
Pumping Well Data DGU file no.207.2656

Set up, date 31.03.81 time 12.35 Shut off, date 04.05.81 time 13.00

Capacity at the Shut off, Q = 24 m3/h

Drawdown: T = 7.7E-4 m2/s

Recovery: T = 0.0012 m2/s



18.11.82 BM

GEOLOGICAL SURVEY OF DENMARK

Figure 1 Pumping test : Solröd Vandvärk - Solröd

PUMPING TEST

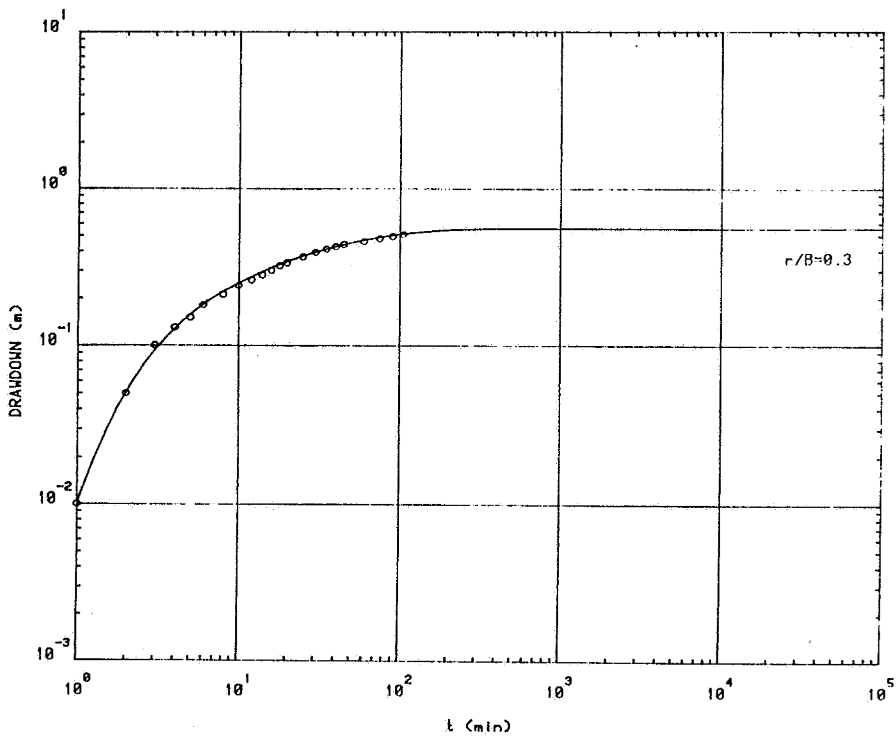
VARMTVANDSLAGRING - HØRSHOLM

Observation Well Data, Pumped Well - DGU file no.194.665

Set up, date 13.05.82 time 13.13

Pumping Capacity, Q = 10.4 m3/h

DGU file no.	Distance (m)	T (m2/s)	S	P'/m' (s-1)	Kr/Kz
194.667	35.44	0.0013	6.1E-4	9.31E-8	15



18.11.82 BM

GEOLOGICAL SURVEY OF DENMARK

Figure 2 Pumping test : Varmtvandslagring - Hørsholm

PUMPING TEST

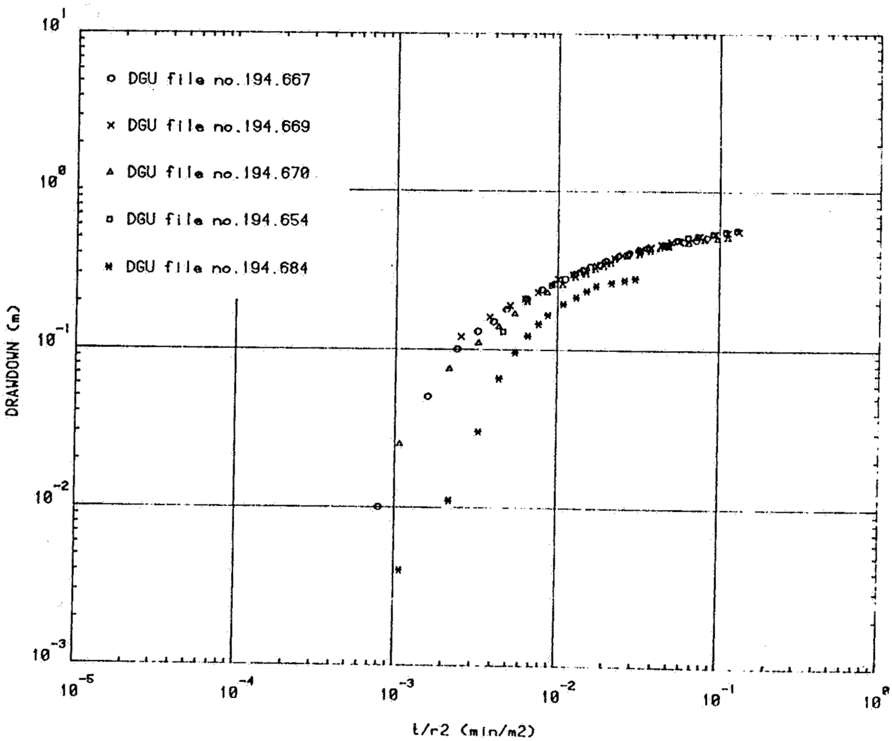
VARMTVANDSLAGRING - HØRSHOLM

Observation Well Data, Pumped Well - DGU file no 194.665

Set up, date 13.05.82 time 13.13

Pumping Capacity, Q = 10.4 m3/h

DGU file no.	Distance (m)	T (m2/s)	S	P'/m' (s-1)	Kr/Kz
194.667	35.44	0.0013	6.1E-4	9.31E-8	15
194.669	28.3	0.0014	5.0E-4	6.99E-8	10
194.670	30.55	0.0013	3.0E-4	6.74E-8	15
194.654	28.68	0.0013	3.9E-4	6.32E-8	15
194.684	58.78				



18.11.82 BM

GEOLOGICAL SURVEY OF DENMARK

Figure 3 Pumping test : Varmtvandslagring - Hørsholm

4 Future aspects

The input process is generally the most time-consuming part of the program. Most of our pumping test data are recorded graphically, either by water level recorders or by transducers. These data curves are presently digitized manually and this is rather laborous work. Faster digitizing and easier data input from recorded graphs will soon be possibel when digitizing equipment is installed at our computer-center.
The present means of interpreting data are sufficient for computing most of the essential hydraulic parameters. However, more comple aquifer-systems exist, for example, one with 2 or more reservoirs with separating semipervious layers and it could be of importance to be able to calculate the hydraulic properties of this entire system. To solve this and similar complicated problems it would be a tremendous advantage to have a numerical model, which can easily generate the type-curves desired for a particular purpose. This numerical method will probably demand much larger computation/curve-drawing times and this could involve rewriting the program in order to utilize our computer facilities.

References

Abramowitz, M. and I.A. Stegun 1970. Handbook of Mathematical Functions. Dover Publications, Inc., New York.

Busk, T. 1973. Numerisk Integration. Hæfte 25. 2. udgave. Numerisk Institut, DTH, Copenhagen.

Hantush, M.S. 1964. Hydraulics of wells, in: V.T. Chow (editor). Advances in Hydroscience vol. I. pp. 281-432. Academic Press, New York and London.

Kruseman, G.P. and N.A. de Ridder 1976. Analysis and Evaluation of Pumping Test data. International Institute for Land Reclamation and Improvement, Wageningen, The Netherlands.

STATISTICAL ANALYSIS OF PARAMETERS AFFECTING WATER TABLE RISE IN IRRIGATED AREAS OF WESTERN UTTAR PRADESH

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Abstract

The multiple regression analysis has been used for development of statistical relationships between water table rise as dependent variable and rainfall, canal irrigation, groundwater extraction, ground slope and premonsoon depth to water table as independent variables, for ground water level data of 175 stations located in alluvial tracts of North India for the period 1971 to 79 for predominantly canal irrigated areas and also for areas only under ground-water irrigation.

1 Introduction

In the present study geohydrological data from the western part of Ganga basin (77^{o}E-88^{o}E and 24^{o}N-30^{o}N) have been analyzed to investigate the influence of various factors viz., seasonal (June to October)rainfall, canal irrigation etc., on the net seasonal water table rise in the alluvial water table aquifers of western U.P. (North India).

2 General characteristics of the area

2.1 Climate

The climate is subtropic monsoonic. The mean annual rainfall varies from 700 mm in the western part to 1400 mm in the eastern part of the Ganga plain. Nearly 87% of rainfall is confined to the monsoon period of June to October. The temperature of the plain ranges from 3^{o}-$4^{o}C$ (January) to 43^{o}-$45^{o}C$ (June).

2.2 Physiography and geology

The area of study is very fertile level plain country constituted mainly of Pleistocene and Subrecent alluvium. On the average the ground slope is of the order of 0.4 m/km from north to south and the principal lithological units are clay, sand and kankar.

There are two aquifer horizons. The water in the shallow aquifer (occuring usually at a depth of 4-10 m) could be considered as under water table conditions while that in the deeper aquifer as under semi-confined to confined conditions.

The area has both natural stream and artificial canal-systems, the former being mainly effluent in nature.

3 Methodology and results

3.1 Data analysed

Following is the list of data analyzed:

a) Y=Seasonal (June-October) rise in water table (m)

b) X1=Seasonal rainfall depth in mm

c) X2=Seasonal canal irrigation depth in mm

d) X3=Groundwater extraction in mm by very shallow devices

e) X4=Groundwater extraction in mm by all devices

f) X5=Ground slope at observation site $X10^{4}$

g) X6=Specific yield of water table fluctuation zone (%)

h) X7=Sand percentage in top 12.20 metres at observation site

i) X8= Premonsoon depth (m) to water table

3.2 Categorisation

To deal systematically with the problem two principal categories of the

Table 1. Explanation of subcategories

Nature of irrigation	Lithological information		
	Specific yield of water table fluctuation zone is known	Sand percentage in top 12.20 meters near observation well calculated	No lithological information available
Hydrograph stations located in predominantly canal irrigation areas	Subcategory I(i)	Subcategory I(ii)	Subcategory I(iii)
Hydrograph stations located in exclusively groundwater irrigated areas	Subcategory II(i)	Subcategory II(ii)	Subcategory II(iii)

Table 2. Sample size of different subcategories

Subcategory	Total sample size	Number of negative Y values	Number of Y values for which groundwater extraction data is not available
I(i)	73	19	43
I(ii)	145	17	72
I(iii)	331	36	153
II(i)	145	30	80
II(ii)	100	18	59
II(iii)	262	41	135

data have been considered. Each of these is further subdivided in three subcategories as explained in Table 1.

3.3 Computer programme

A multiple regression programme was used to analyse the data and consequently to evolve relationships between dependent variable Y and independent variables X1, X2 ... X8 in the form given below:

$$Y=B0+B1.X1+B2.X2+ \ldots +B8.X8$$

BI(I=1 to 8) being the multiple regression coefficient of Y on XI.
The principal results of execution of this programme are - Simple or multiple regression coefficients (BI). Simple or multiple correlation coefficient (R) as well as the correlation coefficient of dependent variable Y with each of the independent variable.
Shahin (1976) has presented tables giving the values of simple/multiple correlation coefficients for different numbers of variables treated and for different sample sizes. The computed values of simple/multiple correlation coefficient can be compared with values of correlation coefficient given by Shahin for corresponding number of variables and sample size. If the computed simple/multiple correlation coefficient is equal to or greater than the corresponding value given by Shahin then the variables involved are significantly correlated.

3.4 Results

Results are discussed category-wise. In the following discussion R and N have been used respectively for multiple correlation coefficient and sample size. Table 2 gives sample sizes for different subcategories. Category I deals with those hydrograph stations which are located in predominantly canal irrigated areas and therefore the results of analysis of data from category I could be used to evolve empirical relationships relating significant geohydrological variables to water table rise in predominantly canal irrigated areas.

Table 3. Results of runs 1 to 5-subcategory I(i)

	Run number				
	1	2	3	4	5
R	0.395085	0.426575	0.432216	0.491654	0.493008
B0	-0.317133	-0.903975	-0.772293	-0.713040	1.065758
B1	0.001840	0.001940	0.001818	0.001974	0.002004
B2			-0.000644	-0.000894	-0.000829
B5				-0.058077	-0.059690
B6					-0.086461
B8		0.084701	0.103603	0.173736	0.179059

Table 4. Results of runs 6 and 7-subcategory I(ii)

	Run number	
	6	7
R	0.458825	0.484904
B0	0.412231	-0.313716
B1	0.001232	0.001268
B2		-0.000079
B5		-0.005426
B7		0.009458
B8	-0.048605	-0.029257

Table 5. Results of runs 8 and 9-subcategory I(iii)

	Run number	
	8	9
R	0.505387	0.528537
B0	-0.051109	0.197412
B1	0.001682	0.001608
B2		-0.000514
B5		-0.012311
B8	-0.010021	-0.002043

Five computer runs of programme were made for subcategory I(i) of this category starting with one independent variable. Each next run included one more independent variable than the preceding one. The resulting multiple correlation coefficient(R) improved on introduction of a new variable to the variable set, thus proving its bearing on recharge characteristics (Kamal,1982) as shown in Table 3.

Following is a list of computer runs for subcategory I(i).Run 1-Y versus X1; Run 2-Y versus X1 and X8; Run 3-Y versus X1,X2 and X8; Run 4-Y versus X1,X2,X5 and X8 and Run 5-Y versus X1,X2,X5,X6 and X8. In each of these runs N=73.

The list of computer runs for subcategory I(ii) is as follows: Run 6-Y versus X1 and X8 (N=145); Run 7-Y versus X1,X2,X5,X7 and X8 (N=145) and list for subcategory I(iii) is:Run 8-Y versus X1 and X8 (N=331); Run 9-Y versus X1,X2,X5 and X8(N=331). The results of these runs are given in (Tables 4 and 5).

The category II deals with data of hydrograph stations located in areas served exclusively by groundwater. The list of computer runs for subcategories of this category is as follows: Run 10-Y versus X1 and X8 (N=145); Run 11-Y versus X1,X5,X6 and X8 (N=145); Run 12-Y versus X1 and X8 (N=100); Run 13-Y versus X1,X5,X7 and X8(N=100);Run 14-Y versus X1 and X8(N=262) and Run 15-Y versus X1,X5 and X8(N=262) (Tables 6 to 8).

The programme was run also for combinations made out of subcategories. All subcategories of category I were combined in order to have large sample size. In some cases the water table rise (Y) values were negative. Data corresponding to these values were removed from the data set to see if the value of R improves. But a comparison of runs 16 and 17 (corresponding) shows that R value deteriorates. This can be seen from Table 9 giving results of following runs:Run 16-Y versus X1,X2,X5 and X8 (N=549) and Run 17-Y versus X1,X2,X5 and X8 (excluding those data for which Y was negative, N=477).

Various subcategories of category II were also combined. Following is list of runs for this combination. Run 18-Y versus X1,X5 and X8(N=507); Run 19-Y versus X1,X3,X5 and X8(N=233) and Run 20-Y versus X1,X4,X5 and X8 (N=233) (Table 10).

Data from all subcategories could be combined with sequel restriction on number of variables which can be treated together. The computer runs using all data are listed below and their results given (Table 11); Run

Table 6. Results of runs 10 and 11-subcategory II(i)

	Run number	
	10	11
R	0.522190	0.554780
B0	-0.048643	-0.351233
B1	0.002117	0.002036
B5		-0.136568
B6		0.047304
B8	-0.058548	-0.062689

Table 7. Results of runs 12 and 13-subcategory II(ii)

	Run number	
	12	13
R	0.580308	0.611872
B0	0.117611	0.185096
B1	0.001664	0.001751
B5		-0.021216
B7		-0.004372
B8	-0.065283	-0.031308

Table 8. Results of runs 14 and 15-subcategory II(iii)

	Run number	
	14	15
R	0.456469	0.502322
B0	-0.188966	0.079364
B1	0.001670	0.001853
B5		-0.041845
B8	0.000248	-0.022521

Table 9. Results of runs 16 and 17-category I

	Run number	
	16	17
R	0.483188	0.405784
B0	0.216873	0.896274
B1	0.001452	0.000941
B2	-0.000249	-0.000445
B5	-0.011839	-0.013516
B8	-0.006338	-0.010766

Table 10. Results of runs 18 to 20-category II

	Run number		
	18	19	20
R	0.416086	0.431506	0.453165
B0	0.772589	0.763535	1.177166
B1	0.001180	0.001308	0.001257
B3		0.011062	
B4			-0.014822
B5	-0.041419	-0.049878	-0.050589
B8	-0.018193	-0.013819	-0.003365

Table 11. Results of runs 21 to 24-all data

	Run number			
	21	22	23	24
R	0.477108	0.483208	0.498407	0.394919
B0	-0.165232	-0.004774	0.081320	0.744437
B1	0.001632	0.001632	0.001653	0.001054
B5			-0.018274	-0.022265
B8		-0.026038	-0.020808	-0.012475

21-Y versus X1 (N=1056);Run 22-Y versus X1 and X8 (N=1056); Run 23-Y versus X1,X5 and X8(N=1056) and Run 24-Y versus X1,X5 and X8 (those data excluded for which Y was negative, N=895). As earlier, the value of R is good so as to imply the significant correlation of various variables involved.

4 Discussion of results and conclusion

As expected positive value of correlation coefficient was found for pair (Y,X1) and negative correlation coefficients for pairs (Y,X4; Y,X5 and Y,X8) as shown in Tables 12 and 13. No conclusive correlation is observed between Y and lithological variables. As against to the expectation the correlation coefficient between Y and X2 (canal irrigation) is negative. But this apparent anamoly could be due to the canal irrigation being an artificial & not the natural source of water. A higher amount of canal water is supplied in seasons of low rainfall & vice-versa. As the main source of water in the area is rainfall,therefore higher the rainfall (lower the canal irrigation) higher will be the rise in water table & vice-versa.

In Table 14 the computed values of R and those specified by Shahin are compared. As seen from Table 14 the computed R values are very superior to values specified by Shahin.

The results of run 16 (Table 9) have been recommended for prediction of Y if other values are known in predominantly canal irrigated areas while results of run 20(Table 10) to be used in exclusively groundwater served areas. The results of run 23 (Table 11) are recommended for areas of western U.P. where irrigation characteristics are unknown.

Table 12. Correlation coefficients of different variables with Y-for category I

Subcategory	Correlation coefficients between Y and X1	X2	X5	X6	X7	X8
I(i)	0.395	-0.134	-0.086	0.047		0.107
I(ii)	0.442	-0.163	-0.006		0.109	-0.049
I(iii)	0.504	-0.238	-0.154			-0.051
Combined I(i), I(ii) & I(iii)	0.469	-0.196	-0.118			-0.039

Table 13. Correlation coefficients of different variables with Y-for category II

Sub-category	Correlation coefficients between Y and X1	X3	X4	X5	X6	X7	X8
II(i)	0.507			-0.202	0.175		-0.127
II(ii)	0.548			-0.181		-0.045	-0.224
II(iii)	0.456			-0.109			0.030
Combined II(i), II(ii)& II(iii)	0.356	0.077	-0.148	-0.225			-0.028

Table 14. Comparison of multiple correlation coefficients (R)

Number of variables	Obtained value of R (Table 11)	Value of R specified by Shahin
2	0.477	0.081
3	0.483	0.096
4	0.498	0.106

References

Kamal, P. 1982. M.Techn. thesis. Statistical analysis of important parameters affecting water table rise in western U.P., UOR, India.

Shahin, M. 1976. Statistical analysis in hydrology, Lecture notes; International course in hydrology, Delft, The Netherlands.

COMPUTER PROGRAMS FOR PROCESSING OF DATA FROM THE LABELLED SLUG TEST AND THE SPINNER FLOWMETER TEST

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Abstract

During the investigation of the Mors salt dome, a site considered as a possible repository for Danish high level radioactive wastes, a new method for testing low-permeable formations - The Labelled Slug Test - was developed. The large amount of data obtained during this test makes manual evaluation both difficult and time consuming. Principles of computerized procedure for the evaluation of the results are given and problems arising during calculation are discussed.
Spinner flowmeter data is normally used to give a qualitative estimate of permeability distribution. Formulas and procedures are proposed which make direct calculations of permeability from spinner readings possible.

Resumé

Pendant l'investigation du dôme de sel de Mors, qui est parmi les localisations possibles pour le dépôt de résidus radioactifs danois, une nouvelle technique pour l'essai de formations peu perméables - le "Labelled Slug Test" - a été développée. La grande quantité de données obtenues pendant ce test rend son traitement à la main difficile et de longue durée. Les principes mis en oeuvre pour le traitement des informations au moyen d'un ordinateur sont indiqués, et les problèmes qui se présentent au cours du traitement sont commentés. Les données obtenues par le "Spinner Flowmeter Test" sont

d'habitude utilisées pour une estimation qualitative de la distribution de perméabilité. Des formules et des procédés qui rendent possible une évaluation directe de perméabilité à partir de ce test sont également proposés.

1 Introduction

In January 1981, during hydrological testing of the Chalk formation overlying the Mors salt dome, it became clear that a quick and inexpensive determination of inflow zones, mainly to point out water sampling intervals, was needed. In order to detect significant changes of water velocity in the borehole, several radioactive pulses were placed in its open section. The movement of the pulses during pumping gave information about water velocity as a function of depth.
Later, this qualitative measure of inflow distribution was supplied with a mathematical description of the problem and a profile of the hydraulic conductivity (permeability) was calculated.
This testing method, called The Labelled Slug Test (LST, was introduced by Andersen et al. (1981).

The Spinner Flowmeter Test (SFT) gives similar information - water velocity in the borehole as a function of depth - and permeability calculations are done in a way similar to LST. Due to the limited sensitivity of the spinner, only relatively high velocities of water (>1 m/min) can be detected.

Variation in the cross-section of the borehole and the large amount of data obtained during a LST/SFT-survey make evaluation of the permeability profile time consuming. In the following, the principles of a computer program for evaluation of a LST/SFT-survey, whose development was supported by IAEA, are presented.

2 Principles of permeability evaluation

Under the assumption that the rate of inflow, $Q_i(X)$, is known for a number of intervals in the open section of the borehole, a calculation of the corresponding permeability profile, K(X), may be performed if the flow in the formation can be described by some known formulas.
In the computer program, there is a choice between the classical Theiss' solution for radial flow in a homogeneous, isotropic medium, Bear (1979), and a formula for fissure flow, Barker (1981).

2.1 Matrix flow

For the purpose of calculation, it is assumed that the hydraulic properties within each interval, defined by two successive measurements, are constant, and that the Theiss' equation gives an adequate description of the flow. In cases where sufficient information is available, any other suitable analytical solution can be used.
Theiss' solution (eq. 2.1) combines the inflow from the i-th interval, Q_i, drawdown, H, transmissivity, T_i, storage coefficient, S_i, pumping time, t, and well radius, R_w.

$$H = \{Q_i/(4\cdot\pi\cdot T_i)\}\cdot \ln\{2.25\cdot T_i\cdot t/(R_w^2\cdot S_i)\} \tag{2.1}$$

Furthermore, it is assumed that:

H = constant through the whole depth of the borehole
$T = K_i\cdot\Delta X_i$, where: K_i = permeability (or hydraulic conductivity)
ΔX_i = the length of the interval considered
$S_i = S_s\cdot\Delta X_i$, where: S_s = specific storage
R_w = average borehole radius

This equation may be expressed in terms of hydraulic conductivity, K_i, as:

$$K_i = \{Q_i/(4\cdot\pi\cdot H\cdot\Delta X_i)\}\cdot \ln\{2.25\cdot K_i\cdot t/(R_w^2\cdot S_s)\} \tag{2.2}$$

or

$$K_i = A_1 \cdot \ln(A_2 \cdot K_i) \qquad (2.3)$$

where A_1 and A_2 are functions of the measured values, ΔX_i, and t, the calculated values, Q_i and R_w and the estimated/calculated value, S_s. This equation is solved by a rapidly converging iterative procedure; there are 3-5 iterative steps, if the convergence criterion is chosen to be 1% of the change between two successive iterations.
Schimschal (1981), proposed a similar formula for permeability calculations from flow-log data obtained during an injection test.

2.2 Fissure flow

During the analysis of fissure and matrix flow, the main problem consists in establishing common references for the presentation of the results, as the matrix is better characterized by the hydraulic conductivity, while the fissures are better characterized by the transmissivity.
For the calculation of the fissure transmissivity, T_f, the equation developed by Barker (1981), is used:

$$T_f = \{Q_i/(2 \cdot \pi \cdot H)\} \cdot \ln\{T_f/(1.781 \cdot R_w (K_h \cdot K_v)^{\frac{1}{2}})\} \qquad (2.4)$$

where:

K_h = horizontal hydraulic conductivity of the matrix surrounding the fissure

K_v = vertical hydraulic conductivity of the matrix surrounding the fissure

1.781 = exp (γ)

γ = Eulers' constant

Q_i = inflow from the fissure

H = drawdown

R_w = average borehole radius

or

$$T_f = B_1 \cdot \ln(T_f \cdot B_2) \tag{2.5}$$

This equation is similar to equation (2.3) and the method of solution is the same. The constants B_1 and B_2 are defined by eq. (2.4).
The "hydraulic conductivity of the fissure" is obtained by dividing the calculated fissure transmissivity by the interval length, a procedure which is only acceptable for the purpose of comparing fissure and matrix properties, but lacks physical meaning.

3 Determination of inflow rate, Q_i

The Q_i-values, which are necessary for the evaluation of the matrix permeability (eq. (2.2)) or fissure transmissivity (eq. (2.4)), are obtained from SFT or LST data.
Changes of water velocity in the borehole may be caused by inflow from the formation or by changes in the cross-section of the borehole. It is convenient to eliminate the influence of the changing diameter by transforming the data obtained in a real borehole into data corresponding to a "converted borehole". The constant diameter of the converted borehole is calculated under the assumption that its length and volume equals the length and volume of the real borehole.
The depth scale of the converted borehole becomes non-linear, but every velocity change after this transformation reflects water inflow from the formation.
One of the first steps in the calculation procedure is the evaluation of the converted borehole and caliper data is used for this purpose.
All depth coordinates hereafter refer to the converted borehole.
The determination of Q_i-values is done differently for SFT and LST.

3.1 Q_i from the Spinner Flowmeter Test (SFT)

The angular velocity of the spinner is a measure of the water velocity (or flow rate) in the borehole, and after the effect of a variable diameter is removed, the changes in spinner velocity express inflow rates for the respective intervals.
A reliable estimate of flow from flowmeter readings requires quite extensive calibration of the tool and this calibration should be performed for the expected range of diameters and flow rates in the field situation.
The calibration curves for different logging procedures: running up, running down and stationary readings, have to be fed into the computer and the field data may then be converted into the required variable-Q_i.

3.2 Q_i form the Labelled Slug Test (LST)

The LST consists in measuring the movement of a number of radioactive pulses in the borehole during water withdrawal at a (preferably) constant rate.
Similarly to SFT, the effect of the changing diameter is removed when calculations are performed for the converted borehole, but unlike a flowmeter-survey, where spinner velocity is directly related to the water velocity and flow rate, the LST data expresses flow in the borehole in a less direct way.

The determination of inflow rates, Q_i, for the intervals defined by the successive positions of the pulses require:
- an exact determination of the pulse positions and
- an assumption about the type of flow, i.e., matrix or fissure flow.

3.2.1 *Determination of pulse positions*

Pulse positions during the LST are evaluated from the readings of the gamma tool, and their accuracy can be influenced by:

- flow pattern in the borehole
- dispersion
- disturbance caused by the logging tool
- speed of the tool, and
- radioactive decay.

At present, laboratory tests are carried out in order to investigate the effect of the above-mentioned factors.

3.2.2 *Fissure/matrix flow*

In order to solve equations (2.3) and (2.5) for the permeability, the knowledge of inflow distribution, $Q_i(X)$, is required. The data provided by the LST consists of corresponding levels and times for the radioactive pulses and the calculation of the Q_i-values has to be based on an assumption about the type of flow for the considered interval: fissure or matrix flow.

When the fissure flow model is used, it is assumed that only fissures contribute to the flow and the velocity of the water is calculated by dividing the distances travelled by the pulses by the corresponding travel times. These calculations are performed for each pulse and the resulting velocity profile is used to calculate $Q_i(X)$. The distribution of the permeability can then be obtained as described in chapter 2.2.

The analysis of matrix flow is more complicated. Under the assumptions valid for equation (2.1), the distribution of the flow rate and the average velocity ($Q(X)$ and $V(X)$) are represented by a continuous function, consisting of a number of straight lines where the slopes are proportional to the permeabilities of the intervals defined by the

detected positions of the pulses ($X_1, X_2, \ldots, X_i, X_{i+1}, \ldots$).
For each interval, $X_i \leqq X \leqq X_{i+1}$, the water velocity, $V(X)$, is expressed by the equation (3.1).

$$V(X) = \frac{dX}{dt} = V(X_i) + f_i \cdot (X - X_i) \tag{3.1}$$

where:

f_i = a proportionality factor which has to be determined from the LST-data.

V_i = the velocity corresponding to X_i.

An integration of equation (3.1) over $\Delta X_i = X_{i+1} - X_i$ and $\Delta t_i = t_{i+1} - t_i$, which are the travelled distance and corresponding time for two successive measurements, leads to equation (3.2)

$$\ln(1 + f_i \cdot \Delta X_i / V_i) = f_i \cdot \Delta t_i \tag{3.2}$$

To solve this equation, it is necessary to know the water velocity at the beginning of the considered interval, and the only unknown in the equation becomes the f_i-value as the positions and times are obtained from the LST-data.
Equation (3.2) represents a system of equations for every pulse. These equations are solved with respect to f_i-values and the velocity profiles one for each pulse, are obtained inserting the solution from equation (3.2) into equation (3.1).
The inflow rates calculated from the velocity profile are then used to calculate the permeability distribution as described in chapter 2.1.

4 Discussion

The combination of the LST and the SFT is a powerful tool for the determination of a permeability profile for a wide range of K-values. The possibility of testing long intervals within a relatively short time and obtaining a high resolution makes this method attractive when compared to the traditional permeability testing methods.

The reliability of the results depends on:

- how well the assumptions and formulas used describe the physical system;
- the accuracy of the measurements.

The first factor, the discrepancy between the model and the real geological system, is a common problem for all evaluation procedures. The second factor, the accuracy of the measurements, allows room for improvement. The understanding of the flow pattern in an irregular shaped borehole, the dispersion of the radioactive pulses and the influence of the logging procedure on the appearance of the pulses, are fields where the description of the LST can be improved.
It is however, a favorable feature of the method that some of its uncertainties can be minimized by a high measurement density. The further development of the LST/SFT-survey will include a theoretical analysis and laboratory testing of the above-mentioned processes.

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References

Andersen, L.J., N. Bull and E. Gosk 1981. Results of a Labelled Slug Test in a Low-Permeability Formation of White Chalk, Mors, Denmark. Proceedings of the Workshop on Siting of Radioactive Waste Repositories in Geological Formations, Paris, 19th-22nd May 1981. Nuclear Energy Agency Organisation for Economic Co-operation and Development.

Barker, J.A. 1981. A Formula for Estimating Fissure Transmissivities from Steady-State Injection-Test Data. Journal of Hydrology, No. 52 (1981) pp. 337-346.

Bear, J. 1979. Hydraulics of Groundwater , McGraw-Hill Series in Water Resources and Environmental Engineering, Jerusalem, 1979.

Schimschal, U. 1981. Flowmeter analysis at Raft River, Idaho. Ground Water. v. 19, no. 1, pp. 93-97.

RESUME'S DE TOUTES LES COMMUNICATIONS EN FRANCAIS

THEME 1:

L'IMPORTANCE ET LES PRINCIPAUX CARACTERISTIQUES DES EAUX SOUTERRAINES

G. Kovács *(pp. 3-22)*

Il y a beaucoup d'endroits dans le monde ou les plus importantes ressources en eaux sont formées par les eaux souterraines. La relation réciproque entre les eaux de surface et souterraines est inévitable, ainsi leur utilization conjonctive est le plus raisonnable approche de l'utilization des eaux. La planification et le fonctionnement des exploitations d'eaux souterraines nécessitent la simulation des systèmes d'eaux souterraines. Que la simulation utilisée pour décrire le régime des eaux souterraines soit faite par le modelage hydrodynamique du système ou par d'autres approximations, des informations sont nécessaires sur:

- la structure des systèmes (leur échelle dans l'espace et le temps);
- les paramètres caractéristiques de la géométrie, du comportement hydrologique et des conditions internes des systèmes, et
- la connexion des systèmes avec les autres parts du cycle hydrologique (conditions aux limites).

L'objectif du symposium est d'évaluer le récent développement des méthodes et moyens adéquats pour déterminer les paramètres caractérisant les systèmes d'eaux souterraines. Ce rapport introductoire fait le point de certains principes qu'il faut considérer, et pose quelques questions lesquelles doivent être répondues après que les moyens les plus appropriés pour l'étude des eaux souterraines aient été étudiés.

- OBJECTIFS ET CRITERES POUR LE DEVELOPPEMENT DES PROGRAMMES DANS LE TERRAIN
  A.C. Skinner *(pp 23-39)*

  La structure d'ensemble ayant trait à la planification des programmes de travail sur le terrain est commentée en détail. Il est fait tout spécialement référence aux différents facteurs d'ordre financier, technique, organisationnel et socio-environnemental, lesquels ont leur rôle dans l'élaboration et la mise à exécution de ces programmes.

THEME 2:

- LA STRATIFICATION DANS LA QUALITE DES EAUX SOUTERRAINES; SON IMPORTANCE POUR LA STRATEGIE DE L'ECHANTILLONNAGE
  J.M. Parker, M.A. Perkins et S.S.D. Foster *(pp. 43-54)*

  La caracterisation suffisante de la qualité des eaux souterraines, a besoin de plus que les programmes d'échantillonnage pour définer les variations régionales et temporelles. A quelque instant spécifique, les variations de la qualité, souvent d'une échelle tres grande, existent problement avec la profondeur dans la zone saturée, surtout dans les nappes phréatiques. Cette stratification de l'eau sera controllé par, et réfléchera, les régimes de la débit des eaux souterraines, régle et naturellement et artificiellement.
  Cette thème sera illustré avec des résultats de plusieurs études détaillées de la pollution diffuse des eaux souterraines dans les zones phréatiques des aquifères les plus importantes de la Grande Brétagne. Un niveau d'investigation, recherche plûtot que routine, donne une vue plus claire de la distribution des solutes dans les systèmes d'eaux souterraines; ce pose aussi de nombreuses questions sur la suffisance des approches traditionelles pour mésurer la qualité des eaux.

- TRAITEMENT STATISTIQUE DE STRUCTURES SEDIMENTAIRES DANS LE SOUS-SOL ET SON EMPLOI POUR DES PROCEDURES D'ECHANTILLONNAGE
  A.N.M. Obdam *(pp. 55-72)*

Dans la reconnaissance géohydrologique on peut appliquer des schèmes divers d'échantillonnage.
Pour l'adaptation optimal de la procédure d'échantillonnage à un certain souterrain sédimentair, c'est recommendable de discerner des structures sédimentaires sur des échelles différentes. La connaissance des conditions sédimentaires peut être serviable pour cet analyse géologique. Excepté des descriptions qualitatives des hétérogénéités, c'est possible d'obtenir une analyse quantitative au moyen de l'application des instruments statistiques divers, comme le semi-variogramme.
Une procédure améliorée d'échantillonnage est proposé par l'application des tests statistiques à plusieurs reprises, d'une hypothèse structurelle à moyen d'une analyse de variance et ensuite de l'adaptation du schème d'échantillonnage.
Aussi quelques observations sont fait, touchant la relation de l'évaluation de frais et des bénéfits des procédures d'échantillonnage.

- APPLICATION DE LA THEORIE DE L'OBSERVATEUR DANS L'OPTIMISATION D'UN RESEAU DE SURVEILLANCE DES EAUX SOUTERRAINES
M. Nawalany *(pp. 73-85)*

La méthode proposée est basée sur trois concepts déjà inclus dans la théorie de la maîtrise: équations de l'état, possibilité d'observation du système et systèmes d'observateur. Une forme canonique des équations de l'état pour un système arbitraire d'eaux souterraines est dérivée et il est proposé un simple critère pour la possibilité d'observation. Pour un tel système d'eaux souterraines le système d'observateur a été construit. L'erreur de la reconstruction de l'état est employée comme un critère dans l'optimalisation d'un réseau de mesure des eaux souterraines.

- PROJET ET OPTIMALISATION DES RESEAUX DE SURVEILLANCE DE LA QUALITE DES EAUX
T. Schilperoort et S. Groot *(pp. 86-100)*

Dans ce document, on présente une approche générale concernante la planification et l'optimisation des réseaux d'échantillonnage. Celle-ci souligne l'importance des rapports entre les objectifs du réseau d'échantillonnage, la dynamique des processus et les techniques d'optimisation. Cette approche a été élaborée pour quelques techniques specifiques fondées sur l'analyse des séries chronologiques, Kriging et le filtre de Kalman. Il parait que la connaissance de la structure de corrélation des processus est essentielle. Finalement quelques applications pratiques des techniques d'optimisation sont présentées.

- REDUCTION D'UN RESEAU DE PUITS D'OBSERVATION DES NIVEAUX DES EAUX SOUTERRAINES

  G.K. Brouwer *(pp. 101-117)*

Un algorithme pratique de prévision a été réalisé pour séparer des puits d'observation primaires et secondaires. Le critère sélectionné est une mesure objective: quand l'erreur quadratique moyenne de l'erreur de prévision dépasse les 15 cm le puit d'observation est considéré comme étant primaire si non secondaire. La méthode utilisée - le filtre de Kalman - est introduite et ensuite 4 exemples sont donnés. Des éléments spécifiques pour la planification des réseaux sont considérés: l'interval des observations et espacement entre les puits. Un article "Incertitude spatiale du niveau piézomètrique" traitera de l'espacement optimale.

- INCERTITUDE SPATIALE DU NIVEAU PIEZOMETRIQUE

  G.K. Brouwer et P.R. Defize *(pp. 118-128)*

L'erreur de l'interpolation spatiale dépend de la variation de la surface piézométrique et des conditions géohydrologiques à la limite. Des publications récentes suggestent que krigeage est une méthode propre, particulièrement convenable pour une répartition optimale des locations d'observation. Pour un réseau national avec 15000 puits une méthode avec une préparation des données minimales est également importante. Dans cette étude le fonctionnement de krigeage pur et d'une combinaison d'interpolation déterministe et krigeage sont comparés. L'aire de l'étude est une nappe aquifère à fuites dans une région septentrionale des Pays-Bas. Des calculs suggèrent que les paramètres du système de krigeage ne dépendent pas du temps. C'est pourquoi les calculs sont faits sur le niveau annuel moyen. Autrement le système géohydrologique est modélé avec une équation des débits à quasi trois dimensions. L'équation différentielle d'un système stationnaire est résolue avec la méthode des éléments finis. Krigeage est appliqué, mais cette fois sur les différences de niveaux observés et calculés. La précision d'estimation est comparée à 9 locations de contrôle et on ne pouvait pas discriminer entre les deux modèles. La sûreté du modèle déterministe-stochastique est considérée comme étant la meilleure.

- LE MODELE CONCEPTUEL HYDROGEOLOGIQUE INDENTIFICATION NUMERIQUE DU SYSTEME ACUIFERE. ACQUISITION ET SYNTHESE DES DONNEES

G. Castany · *(pp. 129-136)*

Le développement du traitement informatique pour l'identification du système aquifère et l'emploi de modèles numériques distribués de simulation, obligent l'hydrogéologue à recueillir des données quantitatives, précises et nombreuses. La synthèse des informations est exprimée par un modèle conceptuel hydrogéologique, base de la modélisation.

- PRINCIPES FONDAMENTAUX DE L'ORGANISATION ET DE LA MISE EN OEUVRE D'ETUDES HYDROGEOLOGIQUES DANS LE TERRAIN POUR L'EXPLORATION DES EAUX SOUTERRAINES

G.V. Kulikov et L.S. Yazvin *(pp. 137-144)*

Dans cette communication les principes fondamentaux sont discutés de l'organisation et de la mise en oeuvre de l'exploration de l'eau souterraine et de leur application dans des conditions hydrogéologiques variées.

UN PROGRAMME DE RECHERCHES DANS LE TERRAIN POUR LA DETERMINATION DE LA QUANTITE ET DE LA QUALITE DES RESSOURCES EN EAUX SOUTERRAINES DANS BASSIN CRETACE EN BOHEME

J. Balek, V. Jiřele, L. Kalábová et J. Vrba *(pp. 145-154)*

Le bassin crétacé en Bohême a une superficie de presque 16000 km^2 et on y exploite à présent plus de 6 m^3s^{-1} d'eaux souterraines. On s'attend au cours de la prochaine décennie à une augmentation du puisage de plus de 100%. Cependant une telle augmentation exige des investigations approfondies.

La structure géologique du bassin est très compliquée. La tectonique et une combinaison de limites hydrologiques/hydrogéologiques divisent ce bassin en neuf sous-bassins. Deux nappes aquifères principales peuvent être distinguées: une nappe libre dans le Turonien Moyen et une nappe captive dans le Cénomanien. Les variations considérables des facteurs climatiques, hydrologiques, tectoniques, géologiques et pédologiques et la couverture végétale variée indiquent que les ressources en eaux souterraines ne sont pas uniformément réparties sur le bassin et sont, du fait de la fluctuation de la précipitation et l'écoulement, aussi très variables d'une année à l'autre.

En vue de l'étendue du périmètre examiné et le grand nombre de données collectées (1000 points d'observation) une approche par un modèle conceptuel s'est avérée comme la plus appropriée pour l'évaluation des ressources en eaux souterraines et de leur qualité sous les conditions ambiantes. Comme entrée du modèle une banque de données a été établie constituée par les données journalières et hebdomadaires provenant de réseaux conventionnels climatiques, hydrologiques, hydrochimiques et hydrogéologiques.

Il importe toutefois en ce qui concerne les paramètres de modèle caractérisant la structure et les formules décrivant les diverses composantes du cycle hydrologique dans le bassin de les vérifier par une système de surveillance plus fréquent et plus complexe dans une petite aire représentative sélectionnée dans le bassin. Une telle approche a mené à l'établissement d'un programme de surveillance dans le bassin versant modèle à Nedamov.

- SURVEILLANCE ECONOMIQUE DE LA QUALITE DES EAUX SOUTERRAINES
K.J. Edworthy *(pp 155-168)*

Dans la mesure où les buts du contrôle sont clairement compris par toutes les parties et que l'hydrogéologie est suffisamment bien connue et comprise, le coût et la valeur d'un réseau doivent être examinés en fonction de la valeur économique de la ressource en eau souterraine. Il est coûteux de faire des forages, le premier objectif doit donc être de limiter le nombre de forages au minimum suffisant pour permettre de couvrir la nappe en superficie et en profondeur; contrôler un même forage à plusieurs niveaux peut résulter en des économies importantes, mais pour ce faire il est important que l'hydrologie de base soit bien connue. Même lorsque l'emplacement et l'intervalle des échantillons sont bien définis, des erreurs d'échantillonnage sont inévitables, bien qu'elles puissent être évaluées. Bien sûr, il est essentiel de comprendre ces erreurs si l'on veut éviter le coût inutile de spécifier une précision fausse de l'analyse.

- CONTROLE DE LA RECHARGE ARTIFICIELLE DANS UN AQUIFERE SUREXPLOITE
A. Aureli *(pp. 169-182)*

Le long de la côte orientale de la Sicile, dans le département de Syracuse, l'exploitation des nappes d'eau souterraine, à la suite de la construction d'importantes industries pétrochimiques, a atteint des pointes très élévées, nettement supérieures à la remarquable capacité de recharge des aquifères, ce qui a entrainé la destruction des réserves, l'abaissement de plus de 100 m du niveau piézométrique original, en portant ce niveau à plus de 70 m au-dessous du niveau de la mer. Dans ces conditions on a commencé à enregistrer une intrusion d'eau de mer. Puisqu'il était nécessaire de préserver les caractéristiques des eaux souterraines qui, à part l'usage industriel, dans la zone sont aussi employées comme eau potable et pour des usages agricoles, on a réalisé une station expérimentale de réalimentation artificielle capable d'injecter dans les aquifères une portée de 350 l/s d'eaux provenant d'un fleuve qui coule plus au nord, ces eaux étant opportunément traitées au préalable.
Pour observer les effects provoqués par l'exécution de la réalimentation

artificielle, on a mis sous observation plus de 60 puits, sur beaucoup desquels on a installé des appareils de contrôle permanent tels que piézographes, salinographes, thermographes. La relation illustre les conditions locales, les exigeances d'acquisition des données pour la reconstruction tridimensionnelle des aquifères et pour l'exécution d'un modèle mathématique et les problèmes pratiques qu'on a dû affronter pour le choix, la mise en oeuvre et la gestion des instruments de contrôle.

THEME 3:

- APPLICATION PRATIQUE DE L'APPROCHE CONTINUE POUR CARACTERISER LA POROSITE DE ROCHES CARBONATEES

J. Kovács *(pp. 185-193)*

La distribution statistique de la taille des fractures est nécessaire pour déterminer le comportement hydrologique des roches solides. L'application de l'approche continue demande aussi quelques connaissances sur la variabilité des fractures. Pour cette raison, une investigation détaillée a été faite pour décrire la porosité linéaire et la distribution de la taille de fracture dans quatre formations de carbonate (deux dolomites et deux pierres calcaires).
Le rapport fait le point des résultats numériques du champ de travail, lequel est l'unique application pratique de l'approche continue connue de la littérature.

- ESTIMATION DE PARAMETRES HYDROGEOLOGIQUES PAR L'EMPLOI DE DONNEES PROVENANT DE L'EXPLOITATION DE CHAMPS DE PUITS

F. Székely *(pp. 194-198)*

Les paramètres hydrogéologiques dans le périmètre de champs de puits en cours d'exploitation peuvent être estimés à l'aide de données des niveaux piézométriques rétablies en permanence dans des puits arrêtés séparément ainsi que de données provenant de puits d'observation. Le niveau piézométrique initial, et les paramètres de transmissivité et de réalimentation, dans le cas d'infiltration dans les lits de fleuve et de fuites verticales, peuvent être déterminés en minimisant la déviation moyenne absolue entre le niveau piézométrique mesuré et calculé. Pour ceci on a utilisé le programme PAROP de FORTRAN IV.

- ESSAI DE POMPAGE A PUITS UNIQUE

L. Nilsson *(pp. 199-209)*

On doit souvent baser les relevés hydrogéologiques et les essais de pompage dans les zones à roches ignées sur des puits uniques.
Une théorie a été élaborée de nappes aquifères fracturées basée sur des données obtenues avec un seul puits. On a construit également à cette fin un appareil de mesure avec une haute résolution dans l'espace et dans le temps.
Le rapport décrit cette technique et donne des exemples de différents types de relevés hydrogéologiques. A part de l'emploi pour l'analyse de nappes aquifères cette technique a été utilisée aussi pour la surveillance de nouveaux et d'anciens puits d'eau à la fois dans les roches ignées et dans les dépôts sédimentaires.

- ESTIMATION DES PARAMETRES DES NAPPES AQUIFERES DANS DES PUITS A LARGE DIAMETRE

V.N. Nair et B.B.S. Singhal *(pp. 210-214)*

Dans la plupart des pays en voie de développement d'Asie et d'Afrique, l'Inde inclue, l'exploitation d'eau souterraine se fait principalement au moyen de puits creusés à large diamètre. Environs les deux-tiers de la surface de terre en Inde est couverte de roches dures et presque 50 pour-cent des ressources d'eau souterraine renouvelable se fait dans ces roches. On manque de solutions mathématiques pour calculer les propriétés hydrauliques de puits peu profonds à large diamètre. Papadopulos et Cooper ont suggéré une méthode d'analyse des données des essais de pompage de puits à large diamètre dans un aquifère captif. Mais ceci n'est applicable qu'à une couche aquifère entièrement pénétrée. Boulton et Streltsova (1976) formulaient des équations pour rabattement dans des

puits à pénétration partielle sous des conditions d'aquifères à nappe libre. En Inde, la méthode de Papadopulos et Cooper a été employée pour l'évaluation de paramètres aquifères (T et S).
Dans cette communication la méthode suggérée par Boulton et Streltsova a été utilisée pour déterminer les valeurs de transmissivité pour des puits à large diamètre captant des formations Miliolite et Gaj du Quaternaire et Tertiaire. Les résultats obtenus sont parfaitement comparables et cette méthode pourrait donc être essayée dans d'autres régions.

- APPLICATION DE LA METHODE DES FILETS LIQUIDES POUR L'ESTIMATION DES PARAMETRES GEOHYDROLOGIQUES AUX NAPPES AQUIFERES DU DECCAN EN MAHARASHTRA
S.B. Deolankar *(pp. 215-222)*

Le terrain basaltique de la région Rahuri, Maharashtra, Inde, est une région irriguée. La variation des valeurs de transmissivité obtenue d'essais de pompage indique la nature anisotropique des couches aquifères. Une carte de contours de la nappe phréatique de la région a été faite et dont on présume qu'elle représente le système hydrologique. Des conditions isotropiques idéalisées sont supposées et les lignes de courant de l'eau souterraine sont commencées là où la transmissivité est connue. Les tubes de courant sont construits en allongeant des deux côtés les lignes de courant. En supposant le même débit dans un tube de courant et en appliquant la loi de Darcy, la variation spatiale dans la transmissivité est calculée. On indique l'anisotropie de couches aquifères par la variation spatiale de la transmissivité de 23 à 123 m^2/jour.

- DETERMINATION DU COEFFICIENT D'EMMAGASINEMENT PAR L'EFFICACITE BAROMETRIQUE
G.J.M. Uffink *(pp. 223-231)*

Dans les dunes sur la côte de Hollande la pression atmosphérique et les variations de la charge hydraulique ont été enregistrées dans dix-sept puits d'observation pendant une période de trois semaines. Les données ont été analysées par l'analyse de série chronologique pour déterminer l'efficacité barométrique. De l'efficacité barométrique on peut dériver le coefficient d'emmagasinement d'une nappe aquifère. Les valeurs du coefficient d'emmagasinement qui sont déterminées utilisant l'expression de Jacob, sont inférieures valeurs obtenues par un essai de pompage. Selon Jacob cette différence peut être causée par l'infiltration des couches adjacentes. Une autre explication résulte de la théorie sur l'élasticité des formations aquifères par Verruijt. Verruijt a montré que la compressibilité du squelette des grains est différente pour une déformation verticale (pression atmosphérique) en comparaison du cas de l'écoulement radial à un puits (essai de pompage). Une relation modifiée entre le coefficient d'emmagasinement et l'efficacité barométrique a été donnée, où la théorie de Verruijt est prise en compte.

- INVESTIGATIONS DES EAUX SOUTERRAINES DANS LE PARAIBA SEMI-ARIDE
  S.V.K. Sarma et J.W.G. de Figueiredo *(pp. 232-242)*

Les résultats d'essais de pompage faits dans la région semi-aride de CAGEPA, Catolé do Rocha, de Paraiba, ont été analysés avec la méthode Jacob (temps-rabattement et distance-rabattement). Des limites imperméables de l'aquifère et des sources d'alimentation ont été localisées avec la méthode des images de puits. Transmissivité et coefficients d'emmagasinement des couches aquifères étaient déterminés pour trouver la valeur de ces aquifères pour des schémas d'utilisation de l'eau dans cette région restrainte. Les désavantages des méthodes actuelles ont été discutés. L'analyse des résultats d'essais montre que les aquifères situés dans les sédiments alluviaux de la rivière Agon, au nord de la ville donnent de l'espoir.

- INTERPRETATION DES RESULTATS DE MESURE IN-SITU
  F.B.J. Barends, K.A. Brink et E.O.F. Calle *(pp. 243-252)*

L'interprétation des résultats de mesures in-situ pour la détermination des caractéristiques géohydrologiques est basée sur l'étalonnage d'un modèle qui répond le mieux possible au comportement observé. Le choix du modèle est cependant plutôt arbitraire. Afin d'éviter une certaine complexité on se sert souvent d'une modèle mathématique simple tout en acceptant que le caractère transitoire des phénomènes ne soit pas traîté rigoureusement. Heureusement, il apparaît que la grandeur considérée, notammant la pression interstitielle, n'est pas influencée outre mesure par cette approximation. Toutefois le comportement d'autres phénomènes, comme le débit sortant en surface et la dilatation du sol, n'est pas aussi insensible au choix du modèle. Cet aspect est traîté dans cet article.

- VIABILITE ET PRECISION DE LA DETERMINATION DES TRANSMISSIVITES A L'AIDE DE PUITS D'ESSAI

J.A. Boswinkel *(pp. 253-266)*

Comparés avec des essais de pompage les puits d'essai ne donnent que de l'information sur la transmissivité d'une nappe aquifère. Une méthode bien connue pour obtenir la valeur de la transmissivité à partir d'essais de pompage pour des écoulements non-permanents dans les nappes captives et celle de Theis-Jacob.
Avec quelques suppositions - exprimées en critères temporels - cette méthode se prête aussi à résoudre des problèmes de nappes semi-captives et libres avec des puits d'essai.
Ces critères sont déterminés aussi pour l'application de la méthode de Theis-Jacob bienque les circonstances ne s'accordent pas avec les hypothèses qui sont à la base de la méthode de Theis-Jacob.
La méthode de Theis-Jacob appliquée avec les critères temporels exigés a été testée sur des données d'essais de pompage pour comparer la viabilité et la précision de tests de puits d'essai estimés d'après cette méthode avec les valeurs des transmissivités obtenues par des essais de pompage.

- APPLICATION D'UN MODELE DE BILAN D'EAU A UN SOL TRES SALIN DANS LES REGIONS SEMI-ARIDES

A.F. Eloubaidy et H.K. Al-Hamdyney *(pp. 267-272)*

Des efforts sont faits pour examiner les données disponibles, à l'aide d'une équation de bilan d'eau pour estimer les coefficients décrivant les propriétés hydrauliques des couches aquifères dans la région du

Dujailah Project, située dans la plaine alluviale du fleuve Tigre (à environs 200 km au sud de Baghdad). La région du projet connaît une condition de salinité grave, qui est dûe à une élévation de niveau excessive du niveau phréatique qui est le résultat de l'irrigation et est augmenté par un drainage naturel insuffisant. Les données rassemblées des niveaux de l'eau souterraine, d'un système de 194 puits peu profonds et de 14 piézomètres, ensemble avec d'autres données du terrain et de laboratoire signifiantes, sont utilisées pour étudier les caractéristiques hydrogéologiques d'aquifères.

- DONNEES DE BASE HYDROGEOLOGIQUES POUR L'INVESTIGATION DE SYSTEMES D'EAUX SOUTERRAINES DANS LES ZONES AVEC DES ROCHES DURES EN SUEDE
  T. Olsson *(pp. 273-278)*

Jadis les investigations des eaux souterraines en Suède ont été concentrées sur les dépôts quaternaires. Ces dépôts possèdent en général une plus grande capacité d'emmagasinement et plus grande perméabilité comparées avec les roches dures qui forment l'imperméable cristallin. Toutefois la dernière décennie a fait ressortir de nouvelles tâches hydrogéologiques avec des demandes d'une connaissance plus approfondie de l'imperméable. Les principaux objectifs de ces investigations sont: caractérisation d'un site pour des constructions souterraines et développement de la méthodologie, investigations du site et caractérisation des eaux souterraines pour le rejet de déchets radioactifs. A l'égard de ces deux objectifs, la capacité aquifère de la masse rocheuse est d'une importance cruciale.

- DETECTION EN CONTINU DES PARAMETRES PHYSICO-CHIMIQUES DE L'EAU (TEMPERATURE, CONDUCTIBILITE, SODIUM) DANS LES SOURCES KARSTIQUES DU JURA SUISSE. TECHNOLOGIE DES MESURES ET INTERPRETATIONS HYDROGEOLOGIQUES
  I. Muller, C. Wacker et C. Wittwer *(pp. 279-288)*

Durant plusieurs années, à l'exutoire de quatre bassins karstiques du Jura suisse, la détection en continu de la température ($\pm$ 0.1^{o}C), de la conductibilité électrique ($\pm$ 3 μS/cm) et de l'activité de l'ion sodium ($\pm$ 0.1 mg/l) est réalisée à l'aide de sondes appropriées.
Comparées à l'hydrogramme des sources, les fluctuations de ces paramètres permettent de les considérer comme traceurs naturels et comme indicateurs des différents composantes de l'écoulement des aquifères karstiques.
Ces enregistrements en continu permettent de cerner des événements limités dans le temps qui ne seraient pas perceptibles par un échantillonnage horaire.
Des résultats importants sont ainsi obtenus sur:
- le mécanisme de crue des sources karstiques
- la vitesse de transfert dans le réseau très perméable (perméabilité de fractures)
- le tarissement de l'infiltration rapide provenant de la surface
- l'alimentation de l'exutoire à partir des "blocs" peu perméables (perméabilité primaire)

Certains traceurs artificiels et pollutions anthropogènes sont aussi mis en évidence à l'aide d'électrodes sélectives appropriées.

- LA TRITIUM COMME UN TRACEUR DES EAUX SOUTERRAINES AU ZIMBABWE
P. Wurzel *(pp. 289-300)*

La distribution et la vitesse des eaux souterraines dans la vallée alluviale du Sabi au Zimbabwe ont été élucidées par l'emploi de séries de mesures du tritium sur une période de 15 ans. La vallée alluviale du Sabi constitue l'aire la plus importante d'alluvium du Zimbabwe. L'étude du tritium a démontré:

a) que le fleuve Sabi ne contribue que faiblement à la réalimentation du bassin aquifère le plus important,
b) que la vitesse des eaux à travers la nappe aquifère perchée était plus grande qu'à travers la nappe aquifère principale et plus profonde, et
c) que la vitesse calculée d'après l'étude du tritium était en bonne circonstance avec celle calculée avec les techniques hydrogéologiques orthodoxes.

- TECHNIQUES DE TELEDETECTION POUR LES RECHERCHES DE SOURCES SOUS-MARINES D'EAU DOUCE: APPROCHE QUALITATIVE ET QUANTITATIVE

A. Gandino et A.M. Tonelli *(pp. 301-310)*

Grâce à ses caractéristiques physiques une surface d'eau se laisse aisément prospecter par des recherches aeriennes de la thermique de l'infrarouge, plus particulièrement la bande de 9 ÷ 11 microns. Pendant plus de huit années d'activités une méthodologie opérationnelle a été développée à fin de localiser les sources sous-marines d'eau douce et d'évaluer leurs débits utilisant l'élaboration électronique de données et des modèles mathématiques. Plus de 700 sources réparties sur 1500 km des lignes côtières italiennes avec un débit total de 100 m^3/sec environ ont été identifiées. Une information spéciale est obtenue par le procédé de tranchage du niveau thermique et l'analyse de la texture. Une telle méthode est également utile pour l'estimation d'un bilan hydrologique régional d'une nappe aquifère et pour la protection des eaux souterraines le long des côtes.

- UNE NOUVELLE APPROCHE GEOPHYSIQUE PAR L'UTILISATION DE TECHNIQUES DE TELEDETECTION POUR L'ETUDE DES PROFONDEURS DES NIVEAUX DES EAUX SOUTERRAINES ET DE L'EVAPORATION DANS LES DESERTS

M. Menenti *(pp. 311-325)*

On présente deux nouvelles méthodes pour déterminer des variables importantes pour l'évaluation des ressources en eau souterraine. On montre d'abord qu'on peut dériver du comportement thermique de la surface les propriétés et l'épaisseur des différentes couches du sol. On présente une application de télédétection à l'infrarouge thermique pour évaluer le niveau de la nappe phréatique. La deuxième méthode s'agit de la détermination de l'évaporation actuelle des régions vastes à l'aide d'une combinaison de télémesures à l'infrarouge thermique aussi que visible avec des mesures prises à la surface.

- APPLICATION DES TECHNIQUES DE TELEDETECTION POUR L'EVALUATION DES RESSOURCES EN EAUX SOUTERRAINES

  G.T. Marathe, T.K. Ghosh et M.G. Srinivas *(pp. 326-333)*

  La Sironcha tehsil dans la province de Chandrapur de l'état de Maharashtra en Inde est un pays couvert de fôrets denses. Il n'a pas été exploré à cause d'un manque de routes praticables en toutes saisons et de moyens de communications. Lorsque le Gouvernement de Maharashtra prenait la décision d'ouvrir cette région retardée à l'industrialisation, il devenait impératif d'évaluer ses ressources en eau. Cette région est enfermée par trois fleuves contenant de l'eau pendant toute l'année mais la partie centrale ne dispose pas d'eau de surface. Pour cette raison on doit évaluer avec urgence les ressouces en eau souterraine. Dans cette communication est décrit comment des techniques de télédétection ont été utilisées ensemble avec des méthodes conventionnelles pour évaluer les ressources en eau souterraine d'environs 6000 km^2 de cette région.

- ESTIMATION DU STOCKAGE NATUREL D'EAU SOUTERRAINE DANS DES DEPRESSIONS INTERMONTAGNEUX PAR DES TECHNIQUES DE TELEDETECTION

  Yu.L. Obyedkov *(pp. 334-342)*

  Une méthode pour estimer les ressources naturelles en eau souterraine des bassins intramontagneux, basée sur l'utilisation de la télédétection et de connaissances des conditions géologiques et physiographiques générales de la région d'investigation, est discutée. Cette méthode permet aux investigateurs d'obtenir une estimation préliminaire de la profondeur de la couche sédimentaire des bassins intramontagneux et de leurs réserves d'eau souterraine.

- LES MESURES IN-SITU DE LA CONDUCTIBILITE HYDRAULIQUE DE MATERIAUX EFFRITES OU PEU CONSOLIDES A FAIBLES PROFONDEURS

  D.J. Allen et M. Price *(pp. 343-353)*

  Connaissance de la conductibilité hydraulique de matériaux effrités à faible profondeur ou peu consolidés importe pour la compréhension de l'infiltration et le mouvement de polluants. Les difficultés que présente l'échantillonnage de tels matériaux rendent nécessairement l'emploi d'essais in-situ. Ainsi a-t-on développé des techniques permettant d'exécuter in-situ des mesures reproductibles de la conductibilité hydraulique sur des materiaux meubles en utilisant des installations qui peuvent être mises en place entre 1.5 et 20 mètres en dessous du niveau du sol. Les installations comprennent une série de piézomètres modifiés qui sont disponibles dans le commerce ou qu'on peut facilement construire. Une description est donnée de l'équipement pour des essais de surface employant l'injection et si possible des essais de pompage à charge constante ou variable. Sont également discutés les résultats de tel essais à deux emplacements.

- L'UTILISATION DE CERTAINS METHODES GEOPHYSIQUES A L'INVESTIGATION DES AQUIFERES DANS LES FORMATIONS CARBONATEES

S. Simionas et C.I. Simionas (pp. 354-364)

Les forages exécutés à travers des formations aquifères carbonatées, à l'aide des installations de forage hydraulique à circulation inverse, n'offrent pas d'informations complètes sur le collecteur par lequel l'eau circule. L'utilisation des combinaisons de méthodes géophysiques d'investigation adéquates pour l'étude des formations carbonatées nous a permis la détermination assez exacte de ses traits physiques, de la lithologie et la structure géologique, de la minéralisation du fluide; ces données-là sont utiles pour l'étude de la dynamique complexe des eaux souterraines qu'on rencontre dans une zone de côte de la mer Noire, à structure géologique compliquée, où la couche aquifère est intensément exploitée.

- DETERMINATION DE PARAMETRES HYDROGEOLOGIQUES ET DE CARACTERISTIQUES AQUAPHYSIQUES DE ROCHES PAR LES RESULTATS D'OBSERVATIONS GEOPHYSIQUES

G.V. Kulikov et N.N. Sharapanov (pp. 365-374)

Pour la réalisation d'une cartographie de paramètres hydrogéologiques et des caractères des roches il est nécessaire d'avoir une information continue de leurs variations spatiales. Cela conduit à des dépenses considérables, même en utilisant des méthodes traditionnelles d'étude (forage et échantillonnage de puits, essais de pompage). L'investigation par des méthodes géophysiques est considérée à présenter plus de perspectives. Une combinaison efficace de méthodes géophysiques, formant une partie essentielle d'études d'hydrogéologie et de géologie d'ingénieur conduites dans le but d'améliorer et de mettre en valeur des terres et pour explorer l'eau souterraine, a été développée et est largement utilisée en U.S.S.R. Une combinaison de méthodes géophysiques permet de faire une estimation des phénomènes suivants (à base de corrélations empiriques):

a) perméabilité des aquifères productifs;
b) la perméabilité des sables et des argiles dans la zone non-saturée;
c) porosité efficace des sables et graviers des aquifères à nappe libre;
d) minéralisation d'eau souterraine et le degré de salinasation des roches dans la zone d'aération;
e) profondeur de l'aquifère à nappe libre; vélocité de l'eau souterraine;
f) composition granulaire des sables et graviers, aussi bien qu'un nombre de propriétés géologiques d'ingénieur.

Les limites d'erreur de l'évaluation quantitative des paramètres varie de 10 à 30%.

- IDENTIFICATION DE L'ECOULEMENT DE L'EAU SOUTERRAINE ET DE LA HETEROGENEITE AQUIFERE PAR LA GEOTHERMOMETRIE

W. van Dalfsen *(pp. 375-384)*

Les dernières années, beaucoup de travail a été fait dans le domaine des simulations numériques de transports souterrains de polluants et de chaleur, liés aux mouvements des eaux souterraines. Plusieurs simulateurs ont été proposés. On peut les évaluer d'après la validité numérique et d'après la mesure dont ils tiennent compte des phénomènes physiques et chimiques, liés aux transports dans les milieux poreux. Pourtant, si les paramètres physiques et chimiques sont insuffisamment connus, on ne peut pas faire une prédiction sur les effects de transport, même pas avec les simulateurs les plus élaborés. A fin de comprendre et d'évaluer le transport dans un milieu poreux souterrain, il est absolument nécessaire d'identifier les variations dans la conductivité hydraulique. Ces variations peuvent être détectées en mesurant des profils de températures dans des puits d'observations, à condition que l'eau passant par ces points d'observation, soit thermiquement signée. De telles situations se produisent autour des puits d'injections où l'eau injectée a une température différente que la température ambiante souterraine, autour de voiries où l'eau percolante a été chauffée et autour des puits d'extraction de chaleur. Plusieurs profiles de températures mesurés, indiquent que beaucoup de variations dans la conductivité hydraulique peuvent exister selon l'axe vertical. Vu ces observations, on peut se douter de la validité de l'emploi de modèles mégascopiques de dispersion hydraulique ou thermique, dans les simulations numériques des phénomènes de transport dans des milieux poreux souterrains.

- ISOTOPES DE MILIEU, REALIMENTATION, ZONES DE DECHARGE ET PRESENCE D'EAUX FOSSILES DANS LE SYSTEME DE NAPPES AQUIFERES DE MADRID

I. Herráez, M.R. Llamas et J.Ch. Fontes *(pp. 385-394)*

Le bassin, vaste (6000 km^2), tectonique de Madrid est comblé par un puissant (près de 4000 m) complexe détritique, tertiaire, continental, aquifère. On présente une étude des teneurs en isotopes du milieu: oxygène 18 (99 mesures), deutérium (26), tritium (18), carbone 13 (14) et carbone 14 (21) qui inclut des résultats inédits et réexamine les données de la littérature. Les teneurs en oxygène 18 varient peu mais de façon significative entre -7.0 et -9.2%. Les eaux souterraines des zones de recharge sont systématiquement enrichies en isotopes lourds par rapport à celles des zones de décharge. Ceci confirme le confinement des circulations et suggère la présence d'eaux anciennes dans la décharge. Cette hypothèse est en accord avec la corrélation qui est mise en évidence entre teneurs en oxygène 18 et en carbone 14. Toutefois, on ne relève aucune variation significative des excès en deutérium entre eaux anciennes de décharge et eaux récentes (contenant du tritium) de recharge ce qui supposerait que les circulations atmosphériques sont restées similaires sur la plage de temps correspondant au transit des eaux.

- DETECTION ELECTROMAGNETIQUE POUR LES EAUX SOUTERRAINES DANS LES ROCHES DURES A FAIBLES PROFONDEURS

H.P. Patra et N.L. Shastri *(pp. 395-404)*

Les limitations de sondages conventionnels de résistivité dans l'étude des conditions substratum dans les formations résistantes, appellent des techniques de sondage multi-fréquence. Une telle méthode "central frequency sounding" a été développée théoriquement et comprends le mesurage des components verticaux du domaine magnétique induit au centre du ruban circulaire ou carré de dimension limitée et convenable. L'analyse de caractéristiques de réponse pour le domaine de fréquence ainsi que le domaine de temps CFS en termes de résolution, de détection et d'équivalence confirme l'application de l'approche pour "shallow groundwater sensing" dans les formations difficiles. La résolution se trouve contrôlée en grande partie par l'épaisseur et la conductibilité de couches individuelles. On remarque que l'équivalence est plus fort dans le type-K que dans le type-H du modile terre. La détection de couches conductibles intermédiaires est bonne.

- ESTIMATION DES TRANSMISSIVITES DE NAPPES AQUIFERES PAR LES MESURES GEOELECTRIQUES

D.C. Singhal et Sri Niwas *(pp. 405-414)*

Une relation analitique entre transmissivité d'aquifères et une résistance électrique transversale normalisée d'aquifères homogènes et isotropiques a été présentée dans cette communication, après la modification d'une relation plus ancienne établie par les auteurs présents, entre transmissivité d'aquifère et résistance transversale. En normalisant la résistivité d'aquifère à l'aide de la résistivité de l'eau souterraine, une relation modifiée est obtenue entre la transmissivité d'aquifère et une résistance transversale normalisée. La relation obtenue est linéaire comme le produit de conductivité hydraulique d'aquifère et la conductivité normalisée d'aquifère est constante pour un bassin particulier et paraît être valable pour le calcul de la transmissivité d'aquifère dans des régions avec une eau souterraine de qualité variable. Les transmissivités obtenues par cette voie sont plus proches de la vraie transmissivité comme devient évident d'un exemple d'aquifères alluviaux de la région du Southern Banda, Uttar Pradesh, India.

- APPLICATION DU "SLUG TEST" ET D'UN ESSAI A PRESSION SINUSOIDALE A LA RECHERCHE DES SYSTEMES AQUIFERES FISSURES
  J.H. Black, J.A. Barker et D.C. Holmes *(pp. 415-424)*

Cet exposé présente une nouvelle interprétation de l'essai classique dit "slug test" et décrit un essai innovateur à pression variable en fonction sinusoidale du temps. On met en évidence que l'analyse classique des slug tests faits dans roches cristallines fracturées amène à des erreurs importantes sur le stockage spécifique; de plus il y a des erreurs moins importantes sur la conductibilité hydraulique. On présente une méthode de correction de ces analyses classiques. L'essai à pression sinusoidale est brèvement décrit y compris une première épreuve de ce principe. L'analyse des résultats à partir d'un milieu poreux homogène s'avérant inapte, et on reprend le modèle du milieu poreux fissuré. On décrit à grands traits quelques possibilités et avantages pratiques de cette technique.

- LA RESISTIVITE ELECTROMAGNETIQUE: UN PUISSANT OUTIL POUR LA DETERMINATION DES VARIATIONS LATERALES EN LITHOLOGIE OU EN QUALITE D'EAU SOUTERRAINE EN PAYS-BAS
  I.L. Ritsema *(pp. 425-438)*

Les variations hydrogéologiques spatiales, par exemple les transitions latérales en lithologie ou en qualité de l'eau souterraine, sont souvent reliées à des variations latérales de la résistivité électrique souterraine. Les variations peuvent être détectées à la surface en mesurant la résistivité apparente souterraine ou bien par la méthode à courant continu (DC) ou bien par la méthode électromagnétique (EM). En général la méthode EM est plus rapide et moins couteuse que la méthode DC.
Dans cette étude, deux appareillages différents pour mesurer la résistivité EM ont été employés: le EM34-3 à boucles d'induction horizontale ou verticale (HL/VL) et le EM16R à ondes radio de très basse fréquence (VLF).

Les résultats obtenus avec ces deux méthodes s'accordent bien avec les résultats obtenus avec la méthode DC et en rapport avec les informations obtenues dans des puits de forage.
Quatre exemples d'application de ces méthodes de résistivité HL/VL et VLF dans les Pays-Bas seront discutés. Les méthodes ont été appliquées afin de détecter une lentille, peu profonde, d'eau douce incluse dans des milieux salins, afin de localiser une couche d'argile conductive et afin de dresser des cartes de l'étendue des pollutions des eaux souterraines autour de deux voiries.
Dans les cas mentionnés, l'application de la méthode de résistivité EM s'est montrée efficace.

- PROFILS ELECTROMAGNETIQUES POUR LES INVESTIGATIONS DE SYSTEMES A PETITE ECHELLE D'ECOULEMENT D'EAU SOUTERRAINE
F. Dirks, W. Geirnaert et M. Groen *(pp. 439-448)*

L'application hydrogéologique de mesures électromagnétiques dans des roches cristallines en Afrique de l'Ouest et dans une zone sédimentaire des Pays-Bas est discutée.
Dans le socle Pré Cambrien de Niger et de Haute Volta, on trouve de l'eau souterraine en quantité exploitable dans des roches cristallines liées à des zones de fracture.
La localisation des fractures sur le terrain par des profils électromagnétiques a énormément amélioré le pourcentage de succès des nouveaux sondages.
Aux Pays-Bas, la méthode électromagnétique est appliquée à l'étude de la distribution spatiale d'eau souterraine contaminée. La méthode donne des bons résultats quand le contraste en conductivité entre l'eau contaminée et l'eau naturelle est suffisant.
Cette méthode présente de grands avantages sur les mesures de résistivité conventionnelles en ce qui concerne la rapidité d'exécution.

- ESTIMATION DE PARAMETRES DANS LES PROBLEMES DE L'ECOULEMENT DES EAUX SOUTERRAINES PAR L'UTILISATION D'UN ALGORITHME DE FILTRE KALMAN
F.C. van Geer et P. van der Kloet *(pp. 449-462)*

Les modèles mathématiques pour calculer l'écoulement d'une nappe souterraine contiennent un certain nombre de paramètres. Le plus souvent il est nécessaire de connaître ces paramètres à obtenir une solution des équations du modèle. Il est possible d'estimer les paramètres et une solution des équations du modèle en même temps avec le Kalman Filtre. Le Kalman Filtre donne un estimateur optimal et linéaire (Sage et Melsa, 1971). Dans ce papier deux manières sont présentées de formuler le Kalman Filtre pour estimer les paramètres. Un problème théorique d'écoulement d'une nappe souterraine est développé comme une application.

- ESTIMATION PAR L'ANALYSE DES VARIABLES MULTIPLES DE L'INFLUENCE DES FACTEURS CLIMATIQUES SUR LES EAUX MINERALES A CARLSBAD

I. Verhoef-Opavská *(pp. 463-475)*

Les données provenant d'observations des sources minérales à Carlsbad/Karlovy Vary, Tchécoslovaquie, et celles des facteurs climatiques sont considérées comme une matrice de données caractérisée par n objets avec p caractéristiques.

Le but de l'analyse des variables multiples est de décrire les particularités du régime des objets en les mettant comme des points dans une espace.

Les résultats de l'analyse indiquent les tendances principales du régime des sources en les confrontant avec les facteurs climatiques. En effet la plupart des sources montre un plus grande variation de la température - reflétant ainsi les changements saisonniers de la température de l'air - que la variation du débit qui n'est plus prononcée que pour une seule source.

Bienque les facteurs climatiques exercent leur influence, la configuration générale des groupes de concentration témoigne de la stabilité interne des paramètres quantitatifs des sources.

THEME 4:

- UTILISATION D'UN NOUVEAU DISPOSITIF POUR DEBITS FAIBLES ET CONSTANTS DANS LES ESSAIS DE POMPAGE DANS DES PUITS CREUSES
R.N. Athavale, V.S. Singh et K. Subrahmanyam *(pp. 479-488)*

En Inde, des puits creusés forment la source première d'eau souterraine pour l'irrigation et pour usage domestique. La détermination de paramètres d'aquifères obtenue de puits creusés est ainsi importante pour une exploitation des ressouces d'eau souterraine. Des essais hydrologiques, faits à l'aide des pompes installées par les fermiers dans les puits creusés, montrent une diminution importante du débit dans le temps. Les puits sont aussi vidés en quelques heures à cause de la grande capacité des pompes. Des essais hydrologiques satisfaisants sur des puits creusés, continuant pendant plusieurs heures et montrant une réaction importante de l'aquifère, sont seulement possibles si le débit est tenu faible et constant. Cet objectif a été atteint au moyen d'une nouvelle invention qui implique le retour au puits d'une quantité variable de l'eau obtenue. Des détails de l'invention sont décrits. La solution analytique de Papadopulos et Cooper (1967) a été utilisée pour interpréter les données.

- DETERMINATION DE NIVEAUX D'EAU DANS DES PUITS D'OBSERVATION PAR MESURE DU TEMPS DE TRANSIT D'IMPULSIONS ULTRASONIQUES ET CALCUL DES PARAMETRES HYDRAULIQUES
R. Kohlmeier, G. Strayle et W. Giesel *(pp. 489-501)*

Les rabattements du niveau d'eau lors des essais de pompage sont déterminés à l'aide de la mesure du parcours d'une impulsion ultrasonique. Dans chacun des piézomètres un vibrateur ultrasonique est pendu quelques mètres au-dessous du niveau de la nappe. L'impulsion émise est réflectée au niveau d'eau et reçue par le vibrateur. Les détails techniques suivants sont traités comme suit: construction de l'appareillage de mesure, nombre et distance des piézomètres qui sont observés simultanément, exactitude de la mesure du niveau d'eau, enregistrement d'autres données (p.ex. débit pompé, précipitation, pression atmosphérique, température,

conductivité électrique de l'eau extraite, enregistrement et traitement des données).
Le résultat des travaux dépend de façon décisive de l'exactitude des données obtenues, comme le montre l'exemple d'un essai de pompage dans l'aquifère très perméable d'un gravier grossier situé dans le haut de la vallée de "Iller" (avant-pays alpin dans le district de Ravenburg, Baden-Württemberg). A cause d'une géométrie compliquée une séquence rapide des données est indispensable. Alors que les influences du bord de sections rectangulaires simples se laissent décrire analytiquement avec relativement peu des données, les bords des sections naturellement arrondies des vallées ne se présentent géohydrauliquement qu'au cours d'une longue phase de transition.

- MESURE AUTOMATIQUE DE LA SUCCION DANS UN SOL

D.M. Borneuf *(pp. 502-512)*

Une station de mesure automatique de la succion dans un sol a été construite et installée dans un bassin versant de l'Alberta. La mesure de la succion se fait à l'aide de tensiomètres reliés à un robinet à voies multiples. Un capteur de pression réagit à la succion de chaque tensiomètre à travers le robinet à voies multiples; les valeurs de la succion aux tensiomètres sont enregistrées sur bande de papier. Un micro-ordinateur commande les fonctions de la station. Un panneau solaire permet le fonctionnement de la station de façon continue. On envisage d'équiper la station de telle sorte que les données puissent être transmises à un ordinateur central qui traitera les données. Le micro-ordinateur utilisé peut l'être pour commander la mesure d'autres paramètres tels que niveau de l'eau dans un puits ou un piézomètre, températures, pression, conductivités etc...

- EQUIPEMENT POUR DES ESSAIS DE PACKERS ATTACHES A UN CABLE DANS DES FORMATIONS DE FAIBLE PERMEABILITE

E. Gosk et L.J. Andersen *(pp. 513-520)*

Le choix de l'installation technique à utiliser pour les essais de

perméabilité, en particulier dans les études de couches peu perméables à grande profondeur, est un facteur essentiel pour calculer les coûts du projet et la crédibilité des résultats.
La mise en place et la fonction d'un seul packer et d'un "straddle-packer" attaché au câble, le dispositif de pompage et le système d'acquisition de données sont décrits.
Ce matériel a été employé à des profondeurs de 550 m au-dessous du niveau de la mer, dans des puits de 9", sans tubage.

- TECHNIQUES D'ECHANTILLONNAGES D'EAU SOUTERRAINE PROVENANT DE PUITS D'EAU
L.J. Andersen *(pp. 521-527)*

De nombreux techniques employés pour les prises d'échantillons sont décrits et commentés. Des échantillonneurs packer qui peuvent prendre des échantillons d'eau souterraine à des profondeurs arbitraires d'une section ouverte ou protégée d'un puits sont utilisés. Le premier modèle utilise un triple-zone-packer, tandis que les deux autres modèles prélèvent les échantillons automatiquement. Les échantillonneurs sont vidés par l'azote ou par la pression de l'eau pompée. Le prélèvement d'échantillons d'eau souterraine est aussi important que son analyse postérieure. Les échantillons de puits à pénétration complète dans des aquifères à charge variable ne représentent pas toujours l'eau originelle de l'aquifère.

- SYSTEME DIGITAL D'ACQUISITION DE DONNEES HYDROLOGIQUES
P.J. Dillon *(pp. 528-537)*

Une description d'un système d'acquisition des données qui contrôle les niveaux d'eaux souterraines et d'eaux de surface, et qui alimente un ordinateur en données enregistrées. Il y a un nouveau transducteur de niveaux d'eau souterraine et une mémoire portative et multi-usages pour fournir des données. L'équipement est relativement bon marché, adaptable à d'autres applications, et s'est révélé fidèle et précis au cours des essais sur le terrain. L'alimentation et la vérification des données demandent peu de temps dans ce système.

- SYSTEME DE SURVEILLANCE DES EAUX SOUTERRAINES

V.J. Latkovich, J.I. Rorabaugh, K.V. Sharp, E.H. Cordes et J.C. Jelinski *(pp. 538-545)*

Les programmes du U.S. Geological Survey (USGS - Recherches Géologiques des Etats Unis) exigent des instruments pour l'acquisition de données sur les niveaux des eaux souterraines de manière continue et sans inspection pendant de longues périodes. Il n'existe à présent aucun système économique pour diriger les processus de la collecte et de la régistration des niveaux des eaux souterraines à long terme ou pour des essais dans les nappes aquifères soit à court soit à long terme où des observations sont nécessaires à des intervalles de temps très courtes (quelques secondes). Le "Geological Survey" est en train de développer un système de surveillance (GWMS) pour répondre à ces besoins spécifiques. Le GWMS est un système d'instruments complémentaires y compris un micro-ordinateur programmable. Le GWMS sera flexible en ce sens que des modifications ultérieures peuvent facilement être appliquées telles que une capacité pour la télémétrie satellite.

- COMBINAISON D'ECHANTILLONNAGES ET D'ESSAIS HYDRAULIQUES DANS LES ROCHES PEU PERMEABLES

R.L.F. Kay et D.C. Holmes *(pp. 546-555)*

Un système à câble avec doubles packers a été développé qui peut mesurer la conductivité et la pression hydrauliques ainsi que le stockage spécifique d'une zone définie dans un forage. Le système a la capacité de prendre des échantillons d'eau dans cette zone. Le système, qui fonctionne jusqu'à un profondeur d'environ 1000 m, a été utilisé pour la prise d'échantillons d'eau souterraine sans contamination à un profondeur de 300 m dans une roche cristalline fracturée. Pour donner un exemple, un échantillon montrait un niveau de ^{14}C en dessous des limites de détection avec presque aucun ^{3}H. Le système permet également la prise d'échantillons pour l'analyse de gaz dissous. Le système comprend une pompe, sous le forme d'un tuyau qui monte de la zone sous examen puis bifurque immédiatement au-dessus d'un soupape pneumatique, qui se situe normalement 10 m environ en-dessous de la surface d'eau dans le forage. A soupape fermé, l'eau auprès de la surface peut être expulsée en introduisant du gaz comprimé. Une fois vidée le soupape est réouverte, ce qui mène à un changement instantané de la pression dans la zone sous examen. Ceci veut dire que lors de la prise d'eau de la zone un "slug test" est exécuté, ce qui donne des informations sur les caractéristiques hydrauliques et la pression à équilibre dans cette zone. En traçant les variations de ces caractéristiques selon le profondeur on peut identifier l'endroit où l'eau entre et sort du forage et, de là, les endroits qui conviennent le mieux pour une prise d'échantillons géochimique. Pour la prise d'échantillons chimique le procédé "slug test" est répété jusqu'au constat de paramètres chimiques stables: normalement 3-5 fois le volume entre les packers.

- UN DISPOSITIF GEOPHYSIQUE MARIN POUR L'ETUDE DE LA RESURGENCE SOUS-MARINE D'EAU SOUTERRAINE
  A.V. Meskheteli et D.V. Kuznetsov *(pp. 556-564)*

Dans cette communication les méthodes et les techniques utilisées pour l'investigation des sources sous-marines de la côte du Caucase de la Mer Noire sont considérées.

- LE "LABELLED SLUG TEST"
  N. Bull, E. Gosk et L.J. Andersen *(pp. 565-573)*

Le "Labelled Slug Test" est une diagraphie de rayons gamma utilisant des traceurs radioactifs pour la détermination de la vitesse d'écoulement dans un trou de sondage. Cette méthode a été utilisée pour l'évaluation de la conductivité hydraulique de formations peu perméables. L'équipement et les procédés sont décrits et commentés. Les résultats obtenus par cette méthode sont comparés avec ceux obtenus au cours des essais packer.

- MESURE DE LA TEMPERATURE DU SOL A L'AIDE D'UN CONE DE TEMPERATURE
  J.G. de Gijt *(pp. 574-584)*

Dans cette article le Fugro température cône sera discuté.
La construction de l'appareil mesuré sera traité en évaluant aussi quelques résultats d'essais en relation avec la méthode de l'exécution.
L'usage de la détermination de la température sera aussi indiquée.

- UNE NOUVELLE METHODE POUR LA MESURE IN-SITU DE LA PERMEABILITE

R.A. Rietsema *(pp. 585-593)*

Les méthodes habituelles pour déterminer la perméabilité in-situ sont l'essai d'infiltration à niveau constant, l'essai d'abaissement, et l'essai de pompage. Les deux premiers sont souvent peu précis, et le dernier est onéreux.
Pour cette raison une sonde nouvelle a été développée. En principe cette sonde est analogue à la sonde électrique de porosité de notre Laboratoire: à un certain débit d'eau est injectée dans le sol à travers le filtre supérieur et aspirée par le filtre inférieur, ou à l'envers; au moyen de deux autres filtres placés entre les premiers deux, la différence de potentiel est mesurée. Or, le coefficient de perméabilité horizontale peut-être calculé du débit d'eau et de la différence de potentiel.
Résultats pratiques:

- Avec cette sonde le coefficient local de la perméabilité horizontale d'un sol sableux anisotrope peut-être mesuré d'une façon simple.
- Pour déterminer le coefficient de perméabilité verticale, qui est rarement requis, deux sondes sont nécessaires. Cette méthode s'est montrée moins précise, et en conséquence ne sera pas traitée ici.
- Dans un sol stratifié on trouve une valeur moyenne.
- La sonde ne peut pas être utilisée en argille.

- ANALYSES DES DONNEES DU RESEAU NATIONAL DE SURVEILLANCE DE L'EAU SOUTERRAINE

  J. Taat *(pp. 594-603)*

  Aux Pays-Bas un réseau de 320 puits a été installé pour surveiller la qualité de l'eau souterraine dans la nappe phreatique et la première nappe captive. Régulièrement la concentration d'une vingtaine de paramètres est déterminée, mais le réseau est aussi utilisé pour des échantillonnages spéciaux.

  Le présent article s'occupe du traitement des données et décrit des méthodes d'analyse statistique et de présentation de la variation géographique de la qualité de l'eau souterraine. On y prête une attention spéciale à la relation eventuelle entre d'une part la qualité de l'eau souterraines et d'autre part l'usage ou le type de sol.

  Les resultats pour quelques paramètres sont présentés.

- ELABORATION PRELIMINAIRE ET STOCKAGE DE DONNEES SUR LES NIVEAUX DE L'EAU A DES INTERVALLES FIXEES DE TEMPS

  S. van der Schaaf *(pp. 604-612)*

  Les observations de la nappe phréatique fournies par des appareils enregistreurs contiennent souvent des erreurs d'origine differente. Avant que ces observations soient utilisées pour en tirer des conclusions dignes de foi, il faut rattraper et, si possible, corriger les erreurs. Après qu'il ne reste plus des erreurs corrigibles, les observations doivent être stockées dans des fichiers permanents qui sont facilement accessibles pour des programmes afin de fournir l'information nécessaire. Il faut que les fichiers soient d'une longueur réduite et n'exigent qu'un temps limité à lire, alors que retrouver des données des dates ou des lieux spécifiés doit être simple.

  Un système de programmes et de sous-routines établis pour retrouver et corriger des erreurs et pour produire des fichiers permanents des données est présenté.

- LE TRAITEMENT STATISTIQUE DE DONNEES HYDROGEOLOGIQUES NON-HOMOGENES
L.Ju. Vostrikova et K.E. Pityova (pp. 613-621)

Des méthodes statistiques sont présentées pour traiter statistiquement des données hydrogéologiques non-homogènes. Ces méthodes nous permettent de vérifier l'hypothèse d'homogénéité de données aussi bien que de diviser des données non-homogènes dans des groupes homogènes du point de vue statistique. Quelques exemples sont donnés pour illustrer l'application des méthodes du traitement de données hydrogéologiques.

- ELABORATION ET INTERPRETATION PAR L'INFORMATIQUE DE DONNEES SUR LES ESSAIS DE POMPAGE
B. Madsen (pp. 622-632)

Un programme informatique pour l'analyse de données obtenues lors des essais de pompage a été développé au DGU. Le programme est interactive et peut être utilisé avec très peu de connaissance en informatique. Le langage utilisé est le BASIC et le programme a été spécifiquement développé pour le Tektronix 4054, un écran graphique lié à un 32K-bytes micro-ordinateur avec option de rafraîchissement. Cette option permet de faire, directement sur l'écran, la paire de courbes types en déplaçant la courbe choise à la position où elle donne le meilleur ajustement, comme dans la méthode d'ajustage manuel. Différentes courbes types basées sur des solutions analytiques sont disponibles pour l'interprétation. Les graphiques du type semilog et log-log peuvent être produits sur l'écran ou sur une copie en papier facilement reproductible.

- ANALYSE STATISTIQUE DES PARAMETRES INFLUENÇANT LES REMONTEES DE LA NAPPE PHREATIQUES DANS DES PERIMETRES IRRIGUES EN UTTAR PRADESH OCCIDENTAL
S.M. Seth, P. Kamal et B.B.S. Singhal (pp. 633-643)

L'analyse multiple de régression était employée à développer des relations statistiques entre l'hausse de la nappe aquifère comme un variable dépendant et la pluie, l'irrigation par canal, l'extraction d'eau souterraine, la plongée de terrain et la profondeur de la nappe aquifère avant la mousson comme des variables indépendants pour des niveaux de la nappe d'eau souterraine de 175 stations situées dans la région alluviale de l'Inde septentrionale pendant 1971-1979 pour le terrain irrigué par le canal pour la plupart ainsi que pour le terrain sous l'irrigation de l'eau souterraine seulement.

- PROGRAMMES D'ORDINATEURS POUR L'ELABORATION DE DONNEES PROVENANT D'ESSAIS DU "LABELLED SLUG" ET DU "SPINNER FLOWMETER"

  G. Gosk *(pp. 644-653)*

Pendant l'investigation du dôme de sel de Mors, qui est parmi les localisations possibles pour le dépôt de résidus radioactifs danois, une nouvelle technique pour l'essai de formations peu perméables - le "Labelled Slug Test" - a été développée. La grande quantité de données obtenues pendant ce test rend son traitement à la main difficile et de longue durée. Les principes mis en oeuvre pour le traitement des informations au moyen d'un ordinateur sont indiqués, et les problèmes qui se présentent au cours du traitement sont commentés. Les données obtenues par le "Spinner Flowmeter Test" sont d'habitude utilisées pour une estimation qualitative de la distribution de perméabilité. Des formules et des procédés qui rendent possible une évaluation directe de perméabilité à partir de ce test sont également proposés.

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PROCEEDINGS AND INFORMATION

No. 1 Proceedings of Technical Meetings 1-6 (with summaries in English), 1952.

1. Investigations into the water balance of the Rottegatspolder
2. The water supply for crops I
3. Observations of groundwater levels
4. Investigations by drain gauges in The Netherlands
5. The water supply for crops II
6. The problem of the increasing salinity of ground and surface water in The Netherlands

No. 2 Proceedings of Technical Meetings 7-10, and
Report on the evaporation research in the Rottegatspolder 1947-1952 (with summaries in English), 1955.

7. The study of precipitation data
8. Model research on groundwater flows
9. Measurements and improvement works in basin of brooks
10. Geo-electrical research

No. 3 Proceedings of Technical Meetings 11-12 (with summaries in English), and
Report on the Lysimeters in The Netherlands I (in English), 1958.

11. The water supply of sandy soils
12. Quality requirements for surface waters

No. 4 Evaporation Symposium and
Report on the Lysimeters in The Netherlands II (with summaries in English), 1959.

No. 5 Proceedings of Technical Meetings 13-14 (with summaries in English), 1960.

13. Groundwater levels and groundwater movement in the sandy areas of The Netherlands
14. Water in unsaturated soil

No. 6 Proceedings of Technical Meeting 15 (with summaries in English and French), 1961.
The regime of the Rhine, the Ysselmeer and Zeeland Lake

No. 7 Proceedings of Technical Meeting 16 (with summaries in English), 1962.
The dry year 1959

No. 8 Proceedings of Technical Meeting 17 (with summaries in English), 1963.
The laws of groundwater flow and their application in practice

No. 9 Proceedings of TechnicalMeeting 18 (with summaries in English), 1963.
Water nuisance

No. 10 Steady flow of groundwater towards wells; compiled by the Hydrologisch Colloquium (in English), 1964.

No. 11 Proceedings of Technical Meeting 19 (with summaries in French and German), 1964.
Geohydrological cartography

No. 12 Proceedings of Technical Meeting 20 (in English), 1966.
Water balance studies

No. 13 Proceedings of Technical Meeting 21 (in English), 1966.
Recent trends in hydrograph synthesis

No. 14 Proceedings of Technical Meeting 22, 1968.
Precipitation data (II) and
Report on the Lysimeters in The Netherlands (III) (both with summaries in English)

No. 15 Proceedings and Information no. 15 (in English), 1969.
Soil - water - plant

Proceedings of Technical Meeting 23, 1968.
Seepage
(Will not be published).

Proceedings of Technical Meeting 24, 1970.
Flow in the unsaturated zone
(Will not be published).

No. 16 Proceedings of Technical Meeting 29 (with summaries in English), 1975.
Hydrological investigations for masterplan for the future watersupply in The Netherlands

No. 17 Proceedings of Technical Meeting 25 (in English), 1973.
Automatic processing of hydrological data

No. 18 Proceedings of Technical Meeting 26 (in English), 1974.
Hydraulic research for water management

No. 19 Proceedings of Technical Meeting 27 (in English), 1974.
The hydrological investigation programme in Salland (The Netherlands)

Proceedings of Technical Meeting 28, 1973.
Water quality of Dutch rivers with respect to water management
(Will not be published)

No. 20 Proceedings of Technical Meeting 30 (in English), 1976.
Salt distribution in estuaries

No. 21 Proceedings of Technical Meeting 31 (in English), 1976.
Groundwater pollution

No. 22 Proceedings of Technical Meeting 32 (with summaries in English), 1976.
Systems approach to the management of water resources

No. 23 Proceedings of Technical Meeting 33 (in English), 1977
Precipitation and measurements of precipitation

No. 24 Proceedings of Technical Meeting 34 (in English), 1978.
Urbanization and water management

No. 25 Proceedings of Technical Meeting 35 (in English), 1979.
The relation between water quantity and water quality in studies of surface waters

No. 26 Proceedings of Technical Meeting 36 (in English), 1980.
Research on possible changes in the distribution of saline seepage in The Netherlands

No. 27 Proceedings of technical Meeting 37 (in English), 1981.
Water resources management on a regional scale

No. 28 Proceedings of Technical Meeting 38 (in English), 1981.
Evaporation in relation to hydrology

No. 29a Background papers for Technical Meeting 39 (in English), 1982.
(Netherlands contributions, related to the PAWN-study, for the ECE-seminar-1980).
Policy analysis for the national water management of The Netherlands