THE EFFECTS OF GRAVEL EXTRACTION ON GROUNDWATER HYDROLOGY

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JESUS COLLEGE

A thesis submitted for the degree of Doctor of Philosophy
in the University of Oxford

Trinity Term, 1983

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ABSTRACT

The study which was carried out around gravel pits at Stanton Harcourt (Oxfordshire) and Ringwood (Hampshire), falls into three sections. The first involves an analysis of the groundwater characteristics of the gravels. This includes an estimation of the permeability of the gravels (using single-well dilution methods) and an analysis of the recharge mechanisms operating in them. The latter is based upon observed relationships between groundwater fluctuations and hydrometeorological factors.

Secondly, the hydrological effects of gravel extraction and dewatering are analysed. By monitoring groundwater levels, the nature and extent of the zone of drawdown around the gravel pits is determined. Induced recharge from rivers and recirculation of water from surface ditches into the pits, as a result of dewatering, is identified. Estimates of the proportion of induced recharge in groundwater are then made using chemical evidence. A mathematical model of gravel pit dewatering is developed. The influence of hydraulic conductivity and the initial pumping rate in determining the effects of dewatering is established by a series of sensitivity analyses using this model. As a practical example, the model is used to predict the effects of dewatering a new site near Stanton Harcourt. The extent of the drawdown around each stage in its development is determined, and this is used to evaluate the effects on private sources and agriculture in the surrounding area.

Thirdly, the effects of one type of rehabilitation project (i.e. lake formation) are investigated. A method for predicting the final lake level is explained. Deviation of the observed lake levels from those predicted is explained in terms of the sealing effect of fine sediment plus chemical and biological processes. The effects of lake formation on groundwater levels and flow are described, and the changes in water quality produced by the flow of groundwater through a lake are examined.

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SECTION I. GENERAL INTRODUCTION

CHAPTER 1

INTRODUCTION AND AIMS OF THE STUDY

This is a study aimed at the identification and analysis of the effects of gravel extraction and dewatering on groundwater hydrology. In this thesis, dewatering is used in relation to measures, usually temporary, for the lowering of groundwater levels to permit excavations to be made in the dry below the normal water-table. Dry working is preferred in order to achieve efficiency of working the deposit and to enable systematic bank profiling in line with after-use proposals.

The project was conceived by Dr. B. Finlayson (then of the School of Geography, Oxford University) and the Amey Roadstone Corporation (ARC) - one of the major gravel extractors in the U.K. It is a sign of the growing conflict, in many areas, between gravel operators and water interests, that there is now an accepted need for scientific evidence to be available in order to justify any restriction of planning approval based upon groundwater objections.

The author's own interest in hydrology was formed under the tutelage of Dr. R. Ward, while studying for a BSc. degree in Geography and Geology at the University of Hull. The particular interest in this project arose because it was an opportunity to work on a relatively new research subject. Although the broad structure of the project was already outlined, there was sufficient scope for the author to direct the study into topics of personal interest (eg. computer modelling). At the same time, because of the close links between Oxford University and ARC, this project was an ideal opportunity to study applied hydrology in both an academic and commercial environment. Having a direct link of communication to ARC meant that it was very much easier to interchange ideas with management within the company. Through numerous meetings and discussions, the initial academic approach has been modified to have a commercial basis. It is hoped that the application of scientific principles to problems in the gravel industry, will delineate the procedures necessary for many future dewatering schemes.

Two field areas were suggested by ARC as being ideal sites for this study. These were the company's gravel ~~pits~~ at Stanton Harcourt (Oxfordshire) and Ringwood (Hampshire). Although there were special reasons for chosing these areas, they were also chosen as being representative of the hydrogeological processes which may obtain at gravel pits throughout the U.K.

There was a need to balance against an initial desire to include as much variation in the controlling variables as possible, a limitation in the number of different areas studied because of the cost in time and money visiting each one. Accessibility and logistics favoured the use of a small number of different areas. Ideally, these should be typical of gravel excavations in general, so that the general conclusions of the effects of dewatering can be applied outside the field areas, but contain special problems or features which merit more specific attention. In the Stanton Harcourt and Ringwood areas, all these initial requirements were fulfilled. Additionally, some initial groundwater monitoring had been undertaken in these areas prior to the commencement of this project in 1977, and this data was made available to the author.

The Stanton Harcourt study area is situated within an extensively developed area of gravel excavations along the Upper Thames valley, to the west of Oxford. The Amey Roadstone Corporation extract gravel from a number of pits here, around the villages of Northmoor (the Dix and Brown pits) and Hardwick (the Wadham-Brasenose Pit), which were used as sites for this study. Groundwater levels in the Northmoor area had been monitored prior to this study (i.e. August to October 1977) by Dr. B. Finlayson, as part of a small-scale survey to gather evidence for a public enquiry on proposed gravel excavations at Watkins Farm, near Northmoor.

The Ringwood study area is extensively worked for flint gravel along the R. Avon between Fordingbridge and Ringwood. The dominant feature is a large ARC pit which is currently being developed on the site of an old airfield, south of the village of Ibsley. To the south of the airfield, is a fairly extensive area of lakes and smaller ponds, formed on the site of old gravel workings; while to the north, the floodplain is largely undisturbed. This particular site therefore proved ideal for an investigation into the inter-relationships between gravel lakes and groundwater.

The main reason however for chosing Ringwood as a field site, was the developments on Ibsley Airfield. Permission was sought to dewater the airfield prior to the commencement of extraction on the site. Initially, it was planned to pump the water from the airfield into either a tributary of the R. Avon (Dockens Water), which flows along the southern edge of the airfield, or into an adjacent lake. Objections were raised by the Wessex Water Authority that:

a) Pumping discharge from the excavation could cause pollution of Dockens Water and damage the trout fishing in the R. Avon further downstream, and also possibly produce changes in stream morphology which could increase the risk of flooding.

b) Pumping large quantities of water into a lake already at a dangerously high level could lead to an increased risk of flooding.

c) Dewatering of such a large site (60 to 70 ha) could lead to excessive groundwater drawdown in the area and to changes in the direction of groundwater flow.

An alternative plan was therefore drawn up by ARC, in which the airfield was to be worked 'dry' in a series of small compartments, or cells, of 3 to 4 hectares in size (fig. 1.1). Only one cell was to be worked at any one time, and water was to be pumped into a previously worked area, thus effectively 'recirculating' the pumped water within the total extraction site. The intervening gravel 'bunds' would be extracted 'wet' at a later stage. By this method of working, no water should be lost from the airfield, so any groundwater effects should be confined to a small area around the cell being dewatered. This allowed a comparison to be made between the effects on groundwater from this method of working and the alternative method which was used in the Stanton Harcourt study area. This is the method in which the water is pumped from the working pit into an adjacent drainage ditch, and is therefore removed from the area.

An additional reason for choosing the Ringwood area for this study, was the interest shown by, and the close working relations with, the Wessex Water Authority. They had installed a network of boreholes, stage-boards and autographic lake recorders in the Ringwood area, in order to monitor the long-term effects of gravel extraction, which were surveyed regularly from February 1975. Access to these sites was given to the author, and all data recorded from the network prior to October 1977 was made available. Although the project sites were principally designed to study the effects of ARC pits only, this was later extended in the Ringwood area to include excavations and lakes belonging to different operators, when the author took over responsibility for monitoring the network set up by the Wessex Water Authority.

It should be noted that although the great majority of this work is based on field-work carried out in the two field areas mentioned above, the author has also visited several other ARC gravel pits in different parts of the U.K. This was both to observe and gain experience of other types of dewatering schemes and also to interchange ideas on the groundwater problems associated with dewatering in those areas, in the light of the research in the Stanton Harcourt and Ringwood areas.

There are several different approaches which may be followed when investigating any environmental problem. The method selected here is a direct consequence of the author's own interests in applied hydrology and modelling, and the results of discussions with ARC and the Wessex Water Authority. This is a study which emphasises the use of analytical methods and experiments in the investigation of the real problems faced by the gravel industry and their conflict with water interests. The final aim of this project is to develop a predictive model and/or set of general principles which can be applied in areas where gravel extraction is proposed. Such a model would not only aid in the understanding of the behaviour and response of groundwater to gravel extraction, but would also permit the prediction of future effects.

The problem at hand is therefore one of evaluating the response of gravel aquifers to large-scale withdrawals of water, often at several centres of pumping. The project conveniently breaks down into three main topics. Firstly, it involves an analysis of the groundwater characteristics of the gravel deposits. Secondly, it involves an analysis of the effects of dewatering practices on the groundwater body and the way in which this interacts with other aspects of the hydrological regime of an area. Thirdly, it involves an assessment of the effects of gravel pit resotration. Although each topic is a separate field of study, they are each closely related and together they form a complete picture of the problems of gravel extraction and dewatering.

The effects of gravel extraction on groundwater cannot be studied in isolation. An understanding of the groundwater processes in gravel deposits (i.e. permeability, recharge, etc.) is vital before a model of gravel pit dewatering can be formulated. This in turn requires some appreciation of the geological inputs. The importance of, for example, sedimentary structures in the gravels and the way in which these affect permeability and groundwater flow, may be as significant to the study as the rate of dewatering or size of excavation. Although the scales of study of the geological factors (this includes particle size analysis) and of the groundwater processes are not of the same order as those of the analysis of the effects of dewatering and pit restoration, they do form an important and necessary part of this thesis.

The properties of gravel aquifers, particularly permeability, are known to vary continuously because of their heterogeneous nature. The detailed linkages between the aquifer properties, acting principally through permeability, and the response of the aquifer to dewatering are an important part of this project. It is important therefore that the major controlling factor, i.e. permeability, is determined accurately. For this reason, one section of this thesis is devoted to the analysis of aquifer permeability and its relationship with the grain-size of the gravel deposits.

Identification of the effects of dewatering is a major aim in this thesis as, once achieved, it is then possible to compare these features objectively between sites. In this context, the problems are concerned first, with collecting the data and secondly, relating this to dewatering activities.

A study of groundwater requires a full survey of the water-table to be undertaken. A large amount of time, particularly in the early stages, was therefore spent in the 'field', monitoring not only groundwater levels, but lake levels and river levels. The nature of the study into groundwater levels, requires a substantial number of monitoring sites. Boreholes were initially installed in both field sites on the basis of providing a total coverage of the areas. Approximately one year was then spent in familiarisation of the areas and learning new techniques (such as dye dilution for measuring permeability in boreholes). After this initial data was analysed, it was possible to identify specific problems which required further study and to augment the borehole networks where necessary. Two main topics which were identified during the initial stages of monitoring and which are given extended coverage in this thesis, are seepage from surface water bodies into dewatered excavations (including induced recharge from the River Windrush), and lake-groundwater interactions. One particularly important aspect of gravel excavation which will not be covered in this thesis is the effects on groundwater of using exhausted gravel workings as land fill sites. This is a major topic ~~in~~ its own right, and its effect on groundwater quality is already adequately covered in the literature. None of the excavations studied during this project were used as landfill sites.

By obtaining a better understanding of the hydrological effects of gravel extraction, it is hoped that:

a) future planning will be considered on a more scientific basis and that safeguards and constraints can be established by the operators to protect existing water interests,

b) the evidence from this project will assist in the design, operation and after-use of gravel pits, and

c) any long-term hydrological problems due to gravel excavation will be evident, and measures to rectify the situation can be developed.

An important part of any project of this type, is to review the current literature on the subject. As far as the author is aware, this is the first extensive scientific study of the effects of gravel extraction and dewatering, at least in the U.K. There is a general awareness amongst hydrologists and gravel operators of the potential problems, such as: dewatering, increased evapotranspiration and/or evaporation, changed groundwater temperatures, modification of groundwater quality; however, the author could find no reports of quantitative studies of the effects of gravel extraction. Many of these problems were discussed during a joint meeting of the Hydrogeological Group of the Geological Society and the Institute of Quarrying in 1977, when a series of papers were presented outlining the influence of gravel extraction on groundwater resources in the U.K. (Anon. 1979). Two case studies, one of which concerned the Ringwood site, were also discussed, to illustrate the different approaches to dewatering required in different areas.

The literature which has been found is mostly relevant to only one part of this work. Some isolated studies of gravel pits have been undertaken in other countries, notably by Hamm (1975), Wrobel (1980) and Stundl (1981) in Germany, Peaudecerf (1975) and Prudhomme (1975) in France, and Kelly (1977) in the U.S.A. These studies however have tended to concentrate on one particular aspect of the problem, i.e. the effect of lake formation on water quality or landfill. Hammand Stundl assessed the effects of flooded gravel pits on water quality, while Wrobel and Peaudecerf extended the study of flooded workings to take into account their effects on groundwater flow and lake/groundwater interactions. Kelly modelled the flow of groundwater around gravel pits in order to assess the extent of pollution from waste disposal. In none of these studies was the effects of dewatering of the gravel excavations considered.

The effects of dewatering are not confined to gravel deposits or to the extractive industry. Peek (1969) describes the effect of dewatering phosphate mines in North Carolina on groundwater. Pumping at an average rate of 220,000 m^3/d was required to permit dry-working. The effect of this was to cause an extensive lowering of the groundwater surface and major changes in groundwater flow patterns up to 40 miles from the pumping site. Larsson et al (1977) in Sweden, describe the effects on groundwater caused by the excavation of unlined rock caverns in Pre-Cambrian metamorphics for oil storage. During the period of excavation, a zone of depression developed in the water-table due to dewatering of the water seeping through the rock fractures into the caverns. The maximum drawdown was 25 metres, at a pumping rate of 570 to 1000 m^3/d. In the U.K., Stanton (1978) describes the effects of quarry development, in the Carboniferous Limestone of the Mendips, on water resources. It is thought that sub-water table workings of Whatley Quarry (which is also owned by ARC), will intercept and remove groundwater that would naturally flow to public supply boreholes down-gradient of the quarry.

Groundwater dewatering in the construction industry probably forms the bulk of the literature on the subject of dewatering. The first recorded lowering of groundwater to facilitate construction was for the Kilsby tunnel on the London to Birmingham railway in 1838 (Neboline, 1944). Since then, the same technique has been used throughout the construction industry, i.e. on construction sites, canals, dry-docks, etc. The various methods used in dewatering are summarised by Mansur and Kaufman (1962). These range from simple intercept ditches (similar in design to those used in the dewatering of gravel pits) to sophisticated well-point systems. Mansur and Kaufman also give numerous references of examples of early dewatering schemes in Germany, the U.S.A., Belgium and England. More recently, Farvolden and Nunan (1970) and Frind (1970) have investigated the effects on groundwater of dewatering the Welland Canal construction site in Ontario, Canada. Farvolden and Nunan give a detailed account of the dewatering operations and observations, including drawdown-time curves for observations wells around the centres of pumping. Drawdowns of up to 0.3 metre were recorded at distances of 13 km from the centres of dewatering, at combined pumping rates of 12,000 m^3/d. Frind develops a two-dimensional computer model to study the response of the aquifer to dewatering and to permit the prediction of future effects. Similarly, Aquado et al (1974) have developed a computer model for determining the optimal plan for dewatering a construction site. It predicts the optimum number of wells, their locations, and the rates of pumping needed to maintain groundwater levels below specified elevations at steady state.

This thesis is divided into five unequal sections. The smaller sections (i.e. sections I and II) relate to the background study, whilst the majority of the text (i.e. sections III, IV and V) is concerned with original work and observations. In section I, the background of the two field sites is discussed and the methods used in setting up the monitoring networks are outlined. Much of the physical description of these areas is reviewed from the literature, expanded where possible by personal observations. Section II is concerned with the determination of the permeability of the gravels and its relation to the grain-size of the gravels. Much of this section is based upon field and laboratory experiments. In section III, the groundwater processes in gravel deposits are examined. In chapter 8, the hydrometeorological processes which influence groundwater recharge are discussed, and in chapter 9, these processes are used to describe water-table fluctuations observed in boreholes from both areas. Recharge mechanisms which are thought to operate in gravel deposits are also proposed. A detailed examination of the effects of gravel extraction and dewatering is given in section IV. In chapter 10, the effects of dewatering on groundwater are examined using groundwater contour maps and a numerical model of gravel pit dewatering is described. In chapters 11 and 12, specific problems of gravel extraction in the two study areas are examined. These are the effects of recirculation of pit discharge and induced recharge of surface water, and the interactions between gravel lakes and groundwater. The main conclusions of the work are included in section V, chapter 13.

This thesis is presented in two volumes. Volume One contains the text (Chapters 1 to 13) and Volume Two contains the Figures (relevant to Chapters 1 to 13), the Appendices and Bibliography for convenience of binding.

CHAPTER 2

DELIMITATION AND DESCRIPTION OF THE STUDY AREAS

Introduction

The purpose of this chapter is to briefly outline the physical background of each study area. This will include a description of each to place it in its regional context, followed by an outline of the main features of its topography, drainage, soils and land-use. The main study areas are shown on the following 1:25000 O.S. maps:-

Stanton Harcourt Study Area:
Sheet SP40 - Oxford (West)
Sheet SP30 - Witney (South)

Ringwood Study Area:
Sheet SU 01/11 - Fordingbridge
Sheet SU 00/10 - Ringwood

The boundaries of each area for the purpose of this study were chosen arbitrarily using natural features (i.e. rivers, terrace edges, geological boundaries, etc.) as far as possible. No specific hydrological importance can be applied to these boundaries, since they do not form complete aquifer or drainage basin boundaries.

2.1 Description of the Stanton Harcourt Study Area

The area under consideration is the triangular tract of low-lying land between the R. Windrush on the west, the R. Thames on the south and east, and the village of Stanton Harcourt to the north (fig. 2.1). It is situated 13 kms to the west of Oxford within the valley of the R. Thames, north-east of its confluence with the R. Windrush at Newbridge. To the north-west and west rises the dip-slope of the Cotswold Hills. To the east of the study area, south of the river, rises the prominent features of Wytham Hill (459 083) Hurst Hill (477 041), and Pickett's Heath (484 030).

2.1.1. Topography and drainage

The study area is dominated by the R. Thames which flows in a broad S-shaped valley eastwards from Northmoor to Oxford. At Newbridge (404 014) the R. Thames is joined by a left bank tributary - the R. Windrush. Near the town of Witney, the R. Windrush becomes braided and forms two distinct channels. These reunite once again at Rack End (405 029), slightly north of the confluence with the R. Thames. Between the two rivers, a wide expanse of gravel rises above the alluvium of the present floodplain forming a series of well-marked terraces. The terraces are found mainly on the northern side of the valley of the R. Thames, since the course of the river seems to have progressively worked its way southwards, removing much of the terrace deposits on the southern side of the valley. The south-flowing tributaries of the R. Thames, namely the R. Windrush, R. Evenlode and the R. Cherwell, dissect the terraces and accentuate the width of the main valley at each confluence, so providing the most favourable sites for the preservation of sand and gravel deposits.

The study area as a whole is quite flat, particularly around the village of Northmoor which is situated within the floodplain of the R. Thames at a height of between 61 to 64 metres O.D. To the north-west, there is a slight increase in the gradient of the land caused by a 'step' between the river floodplain and the second terrace. The surface of the higher terrace is again very flat, rising imperceptibly to the north-west.

Because of the flat, low-lying nature of the land, drainage is augmented by a system of ditches and drains. These mark many of the field boundaries in the area. The three principal drains are Northmoor Brook, Linch Hill Brook and Northmoor New Cut (fig. 2.1). The Northmoor and Linch Hill Brooks drain the area west and north of Northmoor village. These join to a single ditch before entering the New Cut, which subsequently discharges into the R. Thames north of Pinkhill Lock (440 071). Northmoor New Cut was excavated to collect drainage from the low-lying land around Northmoor. The R. Thames, being canalised by locks and weirs, is held at artificially high levels, hence ditches cannot drain directly into the river. The right bank of the New Cut is raised into a low embankment which forms a flood bank between the drain and the R. Thames. Its object is to confine waters from minor floods to the narrow strip of land between the New Cut and the R. Thames.

Linch Hill Brook is of particular importance to this study, since it carries the water which is pumped from the gravel pits at Linch Hill.

This, and Northmoor Brook, therefore, carry water throughout the year, whereas many of the smaller ditches only carry flow during the winter months or following periods of heavy rain. Observations suggest that the major drains are in hydraulic contact with groundwater within the gravels, supplying water to them during the summer, and being supplied from the gravels in winter.

Normally the fall in level of the ditches towards Pinkhill Lock is sufficient to provide the necessary gradient for the clearance of storm waters from the Northmoor area without causing flooding. However, when the R. Thames rises, even in relatively minor flood, tail water levels at Pinkhill rise very considerably and the available discharge gradient of the New Cut is greatly reduced. There comes a stage, therefore, when storm waters from the Northmoor area cannot be kept within the banks of the local watercourses, so that they overtop and flood the surrounding land. Flooding can also occur directly from the R. Thames (or R. Windrush) by overtopping of the banks and spilling over or through the flood embankment.

Some 81 floods have been recorded in this area since 1894. The last major flood occurred in March 1947, although less devastating floods are expected every year. The 1947 flood covered the whole valley floor (a width of about 700 metres at that time), except around Pinkhill Farm (437 072) which is situated on a slightly higher knoll. Smaller floods are usually confined within the areas enclosed by the flood embankments. A stage-discharge curve prepared for Pinkhill Lock by the Thames Conservancy estimates the peak discharge in 1947 as 3260 m^3/sec, three times the normal bankfull flow. The river was above its bankfull capacity for 35 days.

2.1.2. Soils and land use

The Stanton Harcourt study area lies immediately to the north and south of areas mapped and described by the Soil Survey of England and Wales (Jarvis, 1973, Hazeldon, in prep.). Land and Water Management Ltd. (1978) have carried out a detailed soil survey in the Northmoor area. Three main soil groups were recognised in this area, separated on the basis of their drainage into:-

a) brown calcareous soils falling within the Badsey Series (Jarvis 1973) - well drained loamy and clayey soils of variable depth over gravel.

Table 2.1 Land-use capability classification for the area around Northmoor, near Stanton Harcourt

(source: Land & Water Management Ltd. (1978))

CLASS	SUB-CLASS	LIMITATIONS	SOILS
2 minor limitations	2w	Moderate A.W.C.[1] due to moderate soil depth over gravels plus Clayey textures, affecting timing and ease of some cultivations	Bc-2 Bc-4
	2s(w)	as 2s above plus slight subsoil wetness	gBc-1 gBc-2
3 moderate limitations	3s	Either Low A.W.C. due to shallow soil depth over gravels, plus clayey textures Or Moderate A.W.C. plus clay textures, making some cultivations difficult and inhibiting root penetration	gBc-3 gBc-5 Bc-2 Bc-4
	3s(w)	As 3s above, but plus slight subsoil wetness	gBc-1 gBc-2 gBc-3
	3ws	Subsoil wetness plus one or both of Low A.W.C. due to shallow depth over gravels Clay textures inhibiting root penetration and making cultivations difficult	cG-2
4 severe limitations	4ws	Subsoil wetness (poorly drained soils) and clayey soil textures	cG-4

1. **Available Water Capacity**

b) gleyed brown calcareous soil - moderately drained variants of the above.

c) calcareous gley soil, variants of the Kelmscott Series (Jarvis, 1973) - imperfectly or poorly drained variants of a) above.

Fig. 2.2 shows the distribution of these soil types in the study area. The imperfectly drained soils are caused by a high water table and a clayey sub-soil which restricts the movement of surface water.

Land and Water Management Ltd. have also produced a land capability map based on the Soil Survey Classification system (Bibby and Mackney, 1972). The factors that principally determine thiscapability classification in the Stanton Harcourt area are:-

1) Soil Limitations (s) - namely the available water capacity (AWC) of the soils; soil texture and structure; and soil depth.

2) Wetness Limitations (w) - namely soil drainage, and flooding.

The resulting land-use capability classification for the areas around Stanton Harcourt covered in their survey is shown in fig. 2.3, and follows the criteria set out in Table 2.1. The land falls approximately equally within Class 2 and 3, although there are some small amounts of Class 4 land associated with poorly drained areas on the floodplain. An important feature of this map is the lack of Class 1 land.

The limitations of the land, which are caused mainly by imperfect drainage, restrictions in rooting depth, and infavourable soil structure and texture, restrict the uses to which it can be put. Pasture, and spring and winter cereals occur in about equal proportions. The latter occurs more frequently on the higher, better drained terraces, whereas the lower floodplain areas are generally given over to permanent grassland.

Stanton Harcourt lies within the main sand and gravel producing area of Oxfordshire. Therefore, a large proportion of the study area is non-agricultural, consisting of existing or worked-out gravel pits. Fig. 2.1 shows the extent of gravel workings within the study area. At the present time, gravel workings are confined to two areas within the study area; at the Dix Pits, south-west of Stanton Harcourt, and at the Brown Pits, south of Stanton Harcourt. Existing workings also occur in the Windrush valley, to the west of Stanton Harcourt. The Hardwick study area lies adjacent to the Wadham-Brasenose Pit.

The Stanton Harcourt area has been designated as a 'Go' area for sand and gravel working by the Oxfordshire County Council (1977). It is to be expected, therefore, that the amount of land given over to sand and gravel working will increase in the future. Because of the high water-table within the gravels of the study area, dewatering takes place at each of the pits mentioned above. Water from the present Dix Pit is pumped into an adjacent worked out pit, now called Dix Lake. Water from the Brown Pits and the Wadham-Brasenose Pit is pumped into nearby drainage ditches, Linch Hill Brook and Standlake Brook respectively.

Worked-out gravel pits in this area are landscaped, and then allowed to flood. Dix Lake is used as a discharge point for water pumped from the adjacent pits. The series of lakes south of Linch Hill; Stoneacres Lake, Willow Pool and Teal Reach, form a commercial trout-rearing and fishery complex. All of these lakes provide ideal habitats for many species of flora and fauna (particularly wildfowl). Vicarage Lake (402 057), for example, was designated a Local Nature Reserve in 1976. Some of the smaller lakes are used to provide water for the washing plant, or are used as settling ponds for the silt and clay.

An alternative form of restoration is to use worked-out gravel pits as landfill sites. However, because of a lack of suitable fill-material, infilling of gravel pits is rare in the Stanton Harcourt area, although several small pits, west of Stanton Harcourt, have been infilled, largely with 'inert waste' (i.e. builders rubble, etc.).

2.2 Description of the Ringwood study area

The study area lies between Fordingbridge and Ringwood in south-west Hampshire (fig. 2.4). Physically this area can be divided into two contrasting units. The fertile, low lying gravel terraces of the R. Avon, and the sparsely populated heath on the higher land to the east and west of the river valley.

The gravel deposits of the R. Avon valley are the major source of aggregate in the New Forest area. The deposits are currently being worked in a northerly direction between Ringwood and Fordingbridge.

2.2.1. Topography and drainage

The area consists of a wide, flat valley flanked on the east and west by steep escarpments which rise up to a series of quite extensive plateaux and interfluves.

The R. Avon, which flows through the valley from north to south, rises as a complex of small streams which have their sources at the junction of the base of the Chalk with the Upper Greensand in the Vale of Pewsey. Upstream of Downton, the R. Avon is a small stream deeply incised into the Chalk, whereas downstream the valley opens out over the Tertiary strata and the river flows in a broad floodplain. Within the study area the river falls from a height of 30 metres O.D. at Bickton (148 127) to about 15 metres O.D. at Ringwood.

To the east of the R. Avon valley lies the New Forest. Erosion by small streams draining from the high ridges (e.g. Ditchend Brook, Huckles Brook, Dockens Water, and Linford Brook), has dissected the valley side into a number of well-marked plateaux, e.g. Gorley and Ibsley Commons. The position is similar, but not as well developed, on the western flank of the valley. There, the plateaux are generally lower and less dissected, the highest, Alderholt Common, being only just over 61 metres O.D.

Between Fordingbridge and Ellingham, the R. Avon flows within a slight 'trench'. In the field this is marked by a distinct 'step', roughly corresponding in position to the 23 metre (75 ft) contour shown in fig. 2.4. As a result, the greater part of the floodplain, being slightly higher than the level of the river, is well drained. The system of artificial drains and ditches, which are so common in the Stanton Harcourt study area, is largely absent here. However, in the lower lying land adjacent to the river, there is a complex system of drains and carriers. These are remnants of the water-meadows which were common in Hampshire between the seventeenth and nineteenth centuries. The water-meadows have in most places fallen into disuse and only the main carriers still contain water, the lesser ditches and channels being dry and partially infilled.

The R. Avon has an even regime. This regularity derives from the spring-fed chalk tributaries which join at Salisbury to form the main stream. The natural river banks are low, so during particularly wet periods the river does overtop its banks. On the other hand, the small tributary streams which drain the relatively impermeable Tertiary deposits are subject to wide fluctuations in discharge. Heavy rainfall causes a rapid increase in discharge with frequent flooding, whereas even after only short periods of dry weather the flow is reduced to low volumes. In the summer and autumn of 1979, for example, the Linford Brook at Blashford was completely dry.

2.2.2. Soils and land-use

It is only possible to indicate broadly some of the soils to be found in this area, since no specific study has been undertaken.

Most of the soils developed in the Hampshire Basin can be assigned to three major groups (Birch, 1964); the podzolic soils, the brown earths, and the gleys. These soils are strongly influenced by the character and distribution of the superficial deposits. Fisher (1971, 1973) found that on all the terraces below 80 metres in the Avon valley and the New Forest, a moderately to poorly sorted, fine sandy loam deposit was common. Its particle-size distribution and poor sorting suggest it is a flood-loam, and the equivalent of the brickearths described in other parts of southern England (Catt and Weir, 1972).

Podzolic soils most often form on the acid parent materials and are closely associated with the plateaux gravels of the New Forest. The less acid and less podzolised soils form the brown earths. These are generally associated with finer textured parent materials, such as the Bagshot and Bracklesham Beds. They also occur on the better drained terrace gravels bordering the River Avon, especially where brickearth is present. In the poorly drained areas of the floodplain, where the water-table is near the surface, or on the higher terraces where the subsoil is clay-rich, gley soils related to both podzols and brown earths are developed.

Two distinct zones with differing vegetation and land-use exist in this study area. These are related to differences in geology, soils and topography. The steep escarpments and plateaux, which flank the Avon valley, consist predominantly of open heath suited to the poor quality, acidic soils. The heaths are generally well-drained and bear a typical dry heath vegetation of predominantly heather and gorse. In the bottom of the valleys which dissect the plateaux, the heathland vegetation is replaced by bog. A good example occurs in the Linwood Bog (177 095), in the valley of Dockens Water. Drainage is impeded in this case by an accumulation of peaty alluvium. By contrast much of floodplain is well drained and characterised by acid grassland. This is mainly used as pasture, although some cereal and fodder crops are grown on the higher terrace.

Much of the floodplain, particularly between Ibsley and Ringwood, is given over to the working of sand and gravel (fig. 2.5). The main pits currently in operation are at Ibsley Airfield and Blashford.

Smaller pits are being worked at North Gorley and also in the higher terraces and plateau gravels at Rockford Common, Ringwood Forest, and Hammer Warren. It is envisaged that developments will in due course extend as far north as Fordingbridge.

Within the floodplain, much of the gravel lies below the natural water table. Under such circumstances, dewatering of the pits is necessary. A result of the high water table is that worked-out pits have left extensive flooded areas. Already some 82 ha of water, mainly between Blashford and Ringwood, exists. With the development of Ibsley Airfield and other sites this figure will increase to approximately 185 ha. The potential of this area for recreational purposes was recognised by the New Forest District Council and the Hampshire County Council, and forms the basis for the Blashford-Ibsley (Draft) Local Plan (1975). The plan is concerned with the restoration and after-use of worked areas within the Avon valley, and specifies a range of uses and schemes of restoration for individual sites which are considered appropriate. A number of recreational activities are already established in the area. Spinnaker Lake is extensively used for sailing; Snails Lake is used for wind-surfing; and a number of lakes are used for fishing. Fig. 2.6 shows the proposed distribution of lakes within the planning area and their proposed after-use. Guidance is given within the Planning Brief for landscaping the worked-out pits. This includes proposals covering bank gradients and shape, the planting of trees and shrubs to provide screening and cover for wildlife, and shoreline materials.

CHAPTER 3

THE GEOLOGY OF THE STUDY AREAS

The following accounts are based upon published literature, supplemented by field observations.[1] Each section describes the nature of the deposits found in each particular study area and discusses some of the theories of their origin. For completeness, brief mention is also made of related deposits which lie in surrounding areas.

3.1 The Stanton Harcourt Study Area

This area is situated in the centre of the Oxford Clay Vale. Over the majority of the area, the Oxford Clay is concealed by superficial deposits of sand and gravel, but it is exposed in the bottom of the deepest gravel excavations.

Figure 3.1 shows the distribution of drift deposits around Stanton Harcourt, which are all generally regarded as being Quaternary in age.

The oldest deposit, the Plateau Drift, occurs as highly eroded patches at various heights on the fringes of the area (not shown in fig. 3.1). First explained as a flood deposit (Buckland, 1821), it has since been described as a glacial deposit (Geikie, 1877), a marine deposit (Lucy, 1878), a fluvial deposit (White, 1897), a fluvio-glacial deposit (Sandford, 1926), and a Tertiary weathering product (Hart, 1976).

Below the Plateau Drift are a number of gravel terraces. Much of the present knowledge of these deposits is due to a series of papers by Sandford (1924, 1925, 1932, 1965). Recent research, however, particularly by Briggs and Gilbertson (1973, 1980) and Gilbertson (1976), based on sedimentary structures, sedimentological evidence, and non-marine mollusca, has led to a revision of many of the details of the chronological sequence and a reconsideration of the environmental relationships and processes responsible for terrace deposition in this area.

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1 Photographs of some of the features described in this chapter are included in Appendix A1.

The new evidence suggests that the majority of the terrace deposits are of Pleistocene age and fluvio-periglacial origin. Formation was by a combination of solifluction and fluvial resorting in a cold, braided-stream environment. It is also contended that inter-glacial periods were typified by meandering streams, which gave rise to more localised deposits. These processes and environments are illustrated by this author's observations of the Summertown-Radley and Floodplain Terraces.

Of the four main terraces, the Summertown-Radley and Floodplain (or Northmoor) Terraces are the most widespread and have been the subject of much of the most recent studies. The two highest and oldest terraces, the Hanborough and Wolvercote Terraces, are fragmented and far less widespread, as a result of erosion. The Summertown-Radley and Floodplain Terraces are of more importance because they provide the greatest proportion of the available gravel resources in the R. Thames valley. Indeed, much of the present knowledge has come about through an examination of sections exposed by gravel workings.

From faunal and sedimentological evidence, Briggs and Gilbertson consider the Hanborough and Wolvercote Terraces to be the result of aggradation under cold (or even glacial) conditions. It was previously thought that the Hanborough Terrace was an interglacial deposit, probably of Hoxnian age, following the discovery of a warm climate vertebrate fauna at the base of the deposit (Sandford, 1924, Pringle, 1926). It is now thought that these are reworked remains from older, interglacial deposits.

The Summertown-Radley Terrace is extremely widespread and underlies the area south-west of Stanton Harcourt. The terrace surface stands approximately 10 metres above the present R. Thames and slopes gently towards the river. The associated gravels range in thickness from 0.6 metres to 5.1 metres, with a mean of 2.6 metres (Harries, 1977). The Dix Pits are situated within the Summertown-Radley Terrace.

Below the Summertown-Radley Terrace, and separated from it by a bluff in which the underlying Oxford Clay is exposed, is the Floodplain Terrace. This is the youngest terrace in the area. Its surface lies close to the modern floodplain, rising to a height of 2-3 metres as flat benches and 'islands' above the river. The gravels of the Floodplain Terrace range in thickness from 1.1 metres to 6.6 metres, with a mean of 3.3 metres (Harries, 1977). Figure 3.2 shows gravel thickness contours for this terrace in the areas around Hardwick and Northmoor. The great variations in thickness reflect the eroded surface of the Oxford Clay below. The linear orientation of the isopachs, particularly around Hardwick, suggests that the deepest gravels are channel infills. Their orientation is roughly parallel to the present R. Windrush. This suggests that the infilled channels represent former courses of the river. The Brown's Pits are situated within the Floodplain Terrace.

The gravels of the Summertown-Radley and Floodplain Terraces are sedimentologically very similar. They consist principally of medium-to-coarse grained, buff-coloured oolitic limestone gravel, with an interstitial fraction of medium grained sand. Staining and cementation is typical of the gravels, particularly in the Summertown-Radley Terrace. The cement is formed from secondary deposits of calcium carbonate with iron or manganese oxides. It occurs principally as coatings on the grains and in some cases cements the grains together. In extreme cases, secondary cementation has led to the formation of thin 'hard-pans' within the gravels.

The origin of the staining is conjectural. It can be argued that it is of periglacial origin. The staining tends to be associated with periglacial structures such as ice-wedge casts. However, in the modern British climate, manganese and iron accretions can develop under conditions of poor drainage. They are particularly prevalent in the zone above the permanent water-table, where reduction and oxidation can combine to concentrate iron or manganese in solution. They would then be precipitated as relatively stable oxides. It is possible, therefore, that the staining may relate to palaeohydrological conditions. These two hypotheses may be related. Periods of impeded drainage due to permafrost may encourage concentration of iron and manganese. If this was true, staining would occur predominantly in cold climate terraces. The lack of true interglacial terraces in the area prevents a direct comparison with the periglacial deposits. Clarke (1979) believes that the staining may be related to past (or present) periods of stable water-levels. The fact that the staining occurs at various levels within the gravels could support this view.

Sand lenses (some up to 4 metres in width) and channel infills are commonly found, particularly in the Summertown-Radley Terrace, and planar-bedding dominates the gravels. The available evidence, from the sedimentary structures and the volume and calibre of the gravels, suggests that the terraces were deposited as channel fills during a period of great sediment production. The gravels are generally sub-rounded to sub-angular in shape. The soft, easily abraded nature of the host limestone suggests that the gravels have been transported relatively short distances. It is possible that the sediment could have been supplied by solifluction from the valley sides, which was subsequently reworked and resorted by streams to produce the bedded terrace deposits. The sand fraction forms an incomplete matrix to the gravels and can be interpreted as an infiltration deposit, let down into the gravels during quiet periods of stream flow. In parts, the sand element may be almost completely absent, producing 'openwork' gravels. This lack of finer material suggests that either stream discharge was at times very high or that little sand was supplied.

The character of the gravels, and the nature of their bedding, imply deposition under a braided stream regime. In particular, the numerous small channel infills and poor or horizontal bedding are characteristic of deposition in braided conditions. In Dix Pit, well defined channel cross-sections occur, particularly in the north-east faces of the pit, while horizontal bedding predominates along the south-east faces. This indicates a prevailing river flow in a south-west to north-east direction.

An indication of climatic conditions at the time of aggradation is given by the occurrence of intra-formational ice-wedge casts, opening out both within the gravels and at their surface, and cryoturbation layers, which are often well picked out by the differential deposition of iron and manganese oxides. Péwé (1966) demonstrated that ice-wedge formation in coarse materials takes place at, or below, a mean annual temperature of $-6^{o}C$ to $-8^{o}C$. In one area of Dix Pit, a highly disturbed and convoluted layer of sand forms a boundary between two distinct gravel deposits. The lower gravels are characterised by a lack of bedding and are generally highly disturbed, whereas the gravels above are less disturbed and show quite distinct planar bedding. It is possible that the sand deposit may mark the boundary between a lower cold gravel deposit and an upper warmer deposit.

In the Floodplain Terrace, numerous blocks of silt and peat have been found. These have yielded abundant non-marine molluscan and coleopteran faunas. At some sites, more extensive lenses of organic material have been found. These were possibly deposited in ponds or in quiet channels on the ancient floodplain. The smaller blocks are probably fragments of more continuous deposits which subsequently became eroded and redeposited. The non-marine mollusca and their palaeoecological implications have been discussed by Gilbertson (1976). The majority of the molluscs are a mixture of aquatic, hygrophilous and xerophilous species, all of which are tolerant to exposure. Many of the species found are now extinct in Britain, being found only in Scandinavia or Eastern Europe where they have definite cold affinities. The evidence therefore suggests an open, treeless floodplain composed of dry gravel or sand ridges and bars, edged with marsh, and dissected by streams, rivers and ponds.

In contrast to the great majority of terrace deposits in the Upper Thames Valley, a number of mainly biogenic deposits having very different sedimentary structures and a different, more diverse molluscan fauna have been discussed by Briggs et al. (1975), Gibbard and Pettitt (1978), and Briggs and Gilbertson (1980). These are interpreted as being interglacial, warm climate deposits. Four main deposits have been identified. Three of these are localised channel-fills and were found at Sugworth, Wolvercote, and Linch Hill. The fourth, no longer visible, was recorded by Sandford (1924) and Dines (1946) in the upper layers of the Summertown-Radley Terrace and has been identified at six or more localities around Oxford. The deposit at Linch Hill was found in 1978 at the base of the Summertown-Radley Terrace in the excavations of Dix Pit. Although poorly preserved, it consists of a series of gravels, sands, silts and peats containing varied vertebrate and invertebrate faunas which infill a shallow, broad channel cut into the Oxford Clay. Like the Sugworth Channel, the Linch Hill deposits show evidence of lateral shifting of a major stream and the progressive accumulation of point-bar deposits. An abundant non-marine molluscan fauna has been identified, which includes thermophilous species. The overall impression of conditions is a range of marsh and aquatic habitats within, and adjacent to, a large river meandering through a wooded temperate landscape.

The consistent picture which emerges from an examination of these Quaternary deposits in the Stanton Harcourt study area is the distinct differences in environmental and sedimentary processes between the cold stages, which were characterised by the aggradation of the terrace gravels, and the interglacials, in which the deposits appear to have accumulated in major meandering stream channels. The deposits reflect the changes in hydrological conditions between the cold and warm periods, i.e. from essentially braided rivers characterised by intense seasonal variations in discharge and high rates of course sediment input in the cold periods, to meandering rivers during the warmer periods.

The re-interpretation of terrace deposition in the Upper Thames Valley has cast doubts on the traditional chronological sequence in the area. The Floodplain Terrace is the only one to have been dated radiometrically. Several radiocarbon dates have been obtained from peaty material within the terrace (Briggs, 1976), including three from the Northmoor area (11,250± 100 yrs. b.p.; 11,062± 70 yrs. b.p.;

Table 3.1 Tentative sequence of Quaternary deposits in the Upper Thames valley

Deposit	Age of deposition	Climatic conditions	Channel pattern
Alluvium	Flandrian	Interglacial	Permanent
Upper Floodplain Terrace	Late Devensian	Sub-arctic	Braided
Lower Floodplain Terrace	Middle Devensian	Sub-arctic	Braided
Upper Summertown-Radley Terrace	Late Ipswichian	Interglacial	Meandering
Lower Summertown-Radley Terrace	(uncertain, possibly Middle Ipswiohian)	Sub-arctic	Braided
Linch Hill Channel	Ipswichian	Interglacial	Meandering
Wolvercote —— ? ——→			
Wolvercote Terrace	Late Wolstonian	Sub-arctic	Braided
Channel —— ? ——→			
Hanborough Terrace	Early Wolstonian or Late Anglian	Sub-arctic	Braided
Sugworth Channel	Cromerian	Interglacial	Meandering

10,931± 70 yrs. b.p.). It should be noted however that these dates refer to the times of formation of the organic layers, thus recording the mildest intervals. They must therefore predate the gravels to some extent. Overall, the dates fall into two groups, averaging 10,960 (range 10,600 to 11,062) and 34,120 (range 29,500 to 39,300) yrs. b.p. respectively. This suggests that gravel aggradation occurred during the cold stages of the Middle and Late Devensian, separated by peat accumulation during the Upton Warren (48,000 - 27,000 yrs. b.p.) and Windermere (12,000 - 10,800 yrs. b.p.) interstadials. Whether gravel deposition was continuous, or whether a phase of inactivity or erosion intervened, is not clear. It is interesting to note, however, that Corser (1978) and Robson (1977) identified two levels in the Floodplain Terrace around Abingdon and Standlake, separated by a surface height difference of 2 to 3 metres and 0.3 metres, respectively. It is not certain, though, whether these represent two distinct phases of deposition, or whether the surface relief of the terrace is merely an erosional feature. The tentative sequence of Quaternary deposits in the Upper Thames Valley is shown in Table 3.1. The Summertown-Radley Terrace is shown as a composite feature spanning an earlier, cold and a later warm phase, as suggested by Sandford (1924, 1925, 1965).

The Floodplain Terrace is commonly overlain by 0.5 to 1 metre of dark silt or silty clay alluvium deposited by the Thames and the Windrush. A sharp boundary divides the two, although in some places layers of peat (up to 0.3 metres thick)may occur at the junction between the gravels and the alluvium. Samples of wood taken from this layer in a temporary exposure south of Witney (360 084) gave a radiocarbon date of 2660± 85 yrs. b.p. (Hazeldon and Jarvis, 1979). This date is thought to be important since it appears to mark a change to environmental conditions characterised by silt and clay deposition. Hazelden and Jarvis postulate that this may have been brought about as a result of human activity, particularly land clearance and cultivation, which caused soil erosion, more rapid surface runoff, and increased the likelihood of short-term flooding.

3.2 The Ringwood Study Area

The study area lies within the north-western corner of the Hampshire Basin, an asymmetric syncline defined by the Tertiary outcrop to the north and the Isle of Wight monocline to the south. Eocene deposits form the heathlands on both sides of the R. Avon and underlie the sand and gravel deposits within the valley. In the study area itself, the Bagshot Beds, which extend north-east to south-west across the valley, underlie the lower river terrace deposits. Higher level gravels are underlain by both the Bagshot and Bracklesham Beds. The geology of the study area is shown in fig. 3.3. A detailed account of the solid geology of the area is given by Reid (1902), so only brief descriptions will be given here.

The Bagshot Beds comprise a series of fine to medium quartz sands with interbedded 'pipeclay' lenses, which become clayey in the lower part of the succession around Fordingbridge, and between Ibsley and Ellingham. The Bagshot beds are exposed at the bottom of gravel excavations in the area.

The Bracklesham Beds form the main part of the Tertiary beds which give rise to the escarpment, east of the R. Avon valley. Two exposures of Bracklesham Beds were examined within the study area. The first exposure is in an old sand pit at the foot of the valley bluff to the east of Moyles Court (165 084), where there is a large outcrop of buff-coloured ferruginous sands. The deposit is for the most part unconsolidated, but in some parts it is indurated with iron oxides and there are large amounts of carbonaceous material present (including fossil wood). Current-bedding, picked out by impersistent layers of carbonaceous material, is common and in the upper part of the section, desilicified flint fragments are widespread. A similar sequence of sands was recorded in borehole R/15 (see fig. 4.2) in a similar topographical situation. The Bracklesham Beds become finer grained to the east and in the second exposure, situated in an undercut bank of Dockens Water (185 102), they consist of a sequence of thin, highly coloured (purple, green and orange) silty and sandy clays, carbonaceous and gravelly in places.

Over the majority of the area and particularly within the R. Avon valley, the Tertiary Beds are mantled by varying thicknesses of Quaternary deposits, predominantly sands and gravels. Alluvium, consisting of silty clays and sands, covers the floodplain adjacent to the R. Avon, whereas adjacent to many of the tributary streams (eg. Linwood Bog), peat typically overlies waterlogged alluvial and bedrock sands.

For the purpose of this study the sand and gravel deposits are divided into a three-fold classification. The first group, the Upper (or Older) River Gravels, occur as dissected plateau deposits between 85 metres O.D. and 127 metres O.D. They are found almost exclusively on the eastern side of the R. Avon valley. The principal spreads occur on Hampton Ridge, Broomy Plain and Bratley Plain.

The second group of gravels, the Middle River Terrace Gravels, occur as discontinuous spreads close to the margins of the R. Avon valley, between 30 metres O.D. and 85 metres O.D. The major occurrences are found on Godshill, Gorley Common, Ibsley Common and Rockford Common, on the eastern side of the valley, and on Alderholt Common and Ringwood Forest, to the west.

The third group of gravels, the Lower (or Younger) River Terrace gravels, occur as continuous spreads within the valley and floodplain of the R. Avon below 35 metres O.D.

The foundation of our knowledge of the terrace gravels in the R. Avon valley was laid by Reid (1902) and later modified by White (1917), Bury (1920, 1923, 1933) and Green (1936, 1946). Green pioneered the use of intensive geomorphological methods to study the problems of river erosion in the Bournemouth area, which Sealy (1955) extended to include the middle section of the R. Avon valley between Ringwood and Salisbury.

The Upper River Gravels, or Plateau Gravels as they are often called, are preserved on the crests and flanks of the highest ridges in the New Forest. The deposits predominantly consist of coarse, sub-angular flints within a variable, although usually sandy or clayey matrix. They have a proven range in thickness of 1.0 to 9.0 metres, with a mean of 3 metres (Kubala, 1980). The precise age and origin of the Upper River Gravels is still disputed. They have variously been described as being deposited under marine, fluviatile, estuarine, or periglacial conditions, or some combination of these. It is generally accepted, however, that those at lower levels are of late Quaternary age, and are in part reworked from older gravels at higher levels (Small, 1964). It is not clear how and when the New Forest was linked to the source areas of these gravels, which were the chalklands to the north and west. Reid (1902) suggested that a Pliocene river originally flowed across the northern New Forest from Salisbury to Southampton. It has been postulated that the lower level Older Gravels are closely linked to the history of the 'Solent River' (Everard, 1954, Fisher, 1975), a postulated major east-west drainage system first thought to have been established in early Tertiary times (Brown, 1960). Kubala (1980) suggests that the poor sorting and high 'fines' content of the Upper River Gravels indicates a polygenetic origin. Originally fluvial (or fluvioglacial) in origin they are thought to have been subjected to solifluction processes and possibly illuviation of 'fines' from a pre-existing cover.

The Middle River Terrace Gravels form discontinuous spreads of clayey, flint gravel on the lower slopes of the New Forest on the eastern side of the R. Avon valley, and on the western margin of the valley. They correspond to Sealy's (1955) Terraces III to VIII, which he demonstrated to represent terraces of an early R. Avon. The Middle River Terrace Gravels range in thickness from 2 to 8 metres (mean 4 metres) (Kubala, 1980). Various sedimentary structures are preserved in a section at Plumley Farm (125 092), including planar cross-bedding, open-work gravels, and impersistent sand lenses. In Ringwood Forest, similar features have been found which are disrupted into pingo-like structures. The evidence suggests that the gravels were subjected to permafrost conditions following deposition in a braided river environment.

The floodplain of the R. Avon is classified for simplicity as a single deposit following Sealy (1955), here called the Lower River Terrace Gravels. This includes three terraces which Green (1946) identified in the lower reaches of the R. Avon and in the Bournemouth area. Although it is more than possible that these terraces also exist within the more upstream sections of the valley, including the study area, morphological evidence alone is lacking. Much of the original surface, particularly within the study area, is dissected by gravel workings, lakes and minor channels, destroying any apparent evidence of a sequence of terraces within the floodplain. It is noted, however, that the IGS identify 4 terraces within the floodplain (Kubala, 1980).

It was noted on page 15 that the R. Avon flows through an alluvium-filled trench which is separated from the slightly higher gravel-bearing land by a marked 'step'. It is not certain whether this marks the junction between two separate terraces, an older river terrace and a present floodplain terrace, or whether it is an erosional feature. M.A.U. borehole evidence (Kubala, 1980) shows that the gravels underlying the alluvium are the same as the terrace gravels exposed on either side. This would seem to suggest that the feature is erosional.

Since the Lower Gravels form the largest single unit within the study area and these are commercially the most important deposit for gravel extraction, they are discussed in greater detail. Sedimentologically, they are similar to the deposits at higher levels, consisting predominantly of coarse, subangular flints in a largely sandy and clayey matrix. The gravels vary widely in thickness, up to a maximum of 10 metres. In fig. 3.4 gravel thickness contours are shown for part of Ibsley Airfield and the area immediately to the north. The data is taken from ARC prospecting boreholes. The isopachs show the great variations in thickness of the gravels over short distances. Since the surface of the gravels is relatively flat, this indicates the irregularity of the eroded upper surface of the Tertiary strata underlying the gravels. There seems to be a general increase in the thickness of the gravels towards the R. Avon.

The Lower Gravels were probably deposited in a high energy braided stream environment, so accounting for their general homogeneity of size. However, thin longitudinal sand bars (up to 30 cms thick) occur locally at all levels within the gravels. Clarke (1979) reports that these sand bars may extend up to 10 metres in length and, in similar deposits near Verwood he identified a sand-filled channel approximately 1 metre deep and 4 metres wide.

Another common feature, indicating a braided river origin, is the 'openwork' texture, i.e. bands of gravel having no matrix support. These openwork bands may occur at any depth in the gravel, and could make ideal routes for hydraulic transfer within the terraces. This is an important point which will be discussed further in later chapters.

The Lower Terrace Gravels contain peat bands, although they are not common. A layer of organic material, which was found in the bank of Dockens Water (152 083), contained a large quantity of well preserved plan macrofossils, including acorns. The layer is 15 cms to 20 cms thick, and is overlain by very coarse, cemented gravel which is up to 1 metre in thickness. Upstream and downstream of this exposure the organic layer thins out and passes into a layer of orange/brown silt, which contains lesser amounts of organic material. The organic lens may represent a buried palaeosol of unknown, although definitely interglacial age, which was preserved by the waterlogged conditions near the river.

The Lower Gravels are notable in that they show little internal structure or bedding. In some gravel pits,.a poor horizontal stratification can be seen, and in some lenses of gravel a poorly preserved imbrication may be discerned. These features are clearer where the gravel has been coated with manganese. The black coatings of manganese contrasts with that of Stanton Harcourt where iron staining is predominant. Whether this is due to the difference in lithology of the gravels or to some other factors is unclear.

Very little is known about the age of the Lower River Gravels. No evidence was found of periglacial features, such as frost-wedges or cryoturbation. However, the nature and structure of the gravels is usually interpreted as being indicative of deposition by fast-flowing periglacial streams, in which conditions of deposition fluctuated rapidly, as indicated by the occurrence of sandy lenses. It is difficult to reconcile such an hydrological regime with an interglacial period. This is more generally associated with low magnitude, high sinuosity streams. No faunal evidence has been recorded but the presence of the floral remains already described would seem to suggest that gravel deposition was interrupted by warmer periods. It is obvious that much closer examination of these deposits than was possible in this study is needed to resolve these questions.

CHAPTER 4

HYDROLOGICAL DESIGN FACTORS IN THE STUDY AREA

Introduction

One of the aims of this study was to investigate the behaviour of the water-table and to identify the effects of dewatering in the immediate vicinity of gravel pits. Groundwater data was gathered from a network of wells which covered both areas adjacent to the main pits and also areas of unworked gravel. Evidence obtained during the course of the study suggested the occurrence of induced recharge from the R. Windrush, due to the dewatering of Wadham-Brasenose pit near Hardwick. A small number of boreholes were added in this area to provide further evidence.

The study areas, therefore, consisted of:-

1) a network of observation wells in the vicinity of Dix Pit and Brown Pits, Linch Hill, and around the village of Northmoor within the Stanton Harcourt study area (fig. 4.1 a and b).

2) a network of observation wells within the Avon Valley of Hampshire between South Gorley and Ringwood, with a special emphasis on Ibsley Airfield (fig. 4.2).

3) a small borehole network at the Wadham-Brasenose Pit site near Hardwick (fig. 4.3).

Each experimental site and observation well was given a reference number for identification purposes. This number is made up of a prefix representing the location of the study area (i.e. SH or R), and a number to denote the site within the particular area.

The distribution of boreholes within each study area was a compromise between the effort to sample all the varied hydrological conditions within each area and the restrictions placed by site accessibility. The hydrological factors which influenced their location were threefold. Firstly, the need to examine well-level fluctuations in areas unaffected by gravel workings and also in areas adjacent to dewatered gravel pits. Secondly, the need to study the hydrological effects of lake formation. This particularly applied to the Ringwood area where an extensive water-based recreation scheme had been planned. Thirdly, the need to examine induced recharge from the R. Windrush.

Site accessibility was an important factor in the location of the boreholes. Firstly, the sites had to be accessible for the drilling rig to reach them and secondly they had to be easily accessible for monitoring.

Three other factors limited the siting of the boreholes. Firstly, in agricultural areas they were not located in the middle of fields because of the interference to crops and possible damage by farm machinery. Secondly, it was not possible to site boreholes in those areas where gravel extraction was scheduled. Thirdly. an important limitation on the number of boreholes installed was the cost of materials and the number which could be installed during the time a drilling rig was available.

A number of observation wells were already in place in each study area before this project started. As far as possible, these were incorporated into the rest of the network which was also supplemented by large-diameter bricked wells, where they were found to be in a suitable condition. All boreholes inserted after October 1977 were installed by the author.

The total number of wells in each study area, when they were fully operational, was:-

a) Stanton Harcourt study area - 41, including 27 polythene tube wells of 3.8 cm diameter and 13 large diameter bricked wells. Sixteen of the tube wells had been installed before October 1977 by the Amey Roadstone Corporation.

b) Ringwood study area - 51, which included 41 wells of 3.8 cms diameter, 5 wells of 5.8 cms diameter, 1 well of 13 cms diameter, 3 wells of 20.3 cms diameter and 1 large diameter bricked well. Twenty-six of these wells had been installed before October 1977 by the Wessex Water Authority.

c) Hardwick area - nine 3.8 cms diameter wells, which were all installed during the course of this project.

Although great care was taken in siting the observation wells where they would be least likely to suffer damage or interference, a total of 13 were subsequently damaged beyond repair. Two further wells, R/1 and SH/48, became unuseable because they did not remain in contact with the water-table.

In the Stanton Harcourt study area, the 13 brick wells and 16 tube wells installed prior to this study were monitored from 2nd August 1977, measurements generally taken at three week intervals. In January 1979, additional boreholes were added in the Stanton Harcourt and Hardwick areas, and monthly measurements were taken covering the 12 month period 8th February 1979 to 19th February 1980. The observations usually took one day to complete by manual dipping, although when the nearby Hardwick study area was also being monitored ~~this was~~ extended over two consecutive days.

In the Ringwood study area, 26 observation wells were installed by the Wessex Water Authority as part of the Blashford Groundwater Study and commencing on 19th February 1975 these were monitored at monthly intervals. In February 1978, the author took over responsibility for monitoring those wells within Ibsley Airfield and these were then monitored at fortnightly intervals. In June 1978, the tube well network was enlarged to include the unworked areas north of Ibsley Airfield and the whole network of 48 observation points was then measured on a monthly basis for a 12 month period between 11th July 1978 and 13th July 1979. Each set of observations took two days to complete. From July 1979 onwards, the observations were confined to the network of wells within the heavily worked area between Ibsley and Blashford. Particular attention was paid to those wells within Ibsley Airfield where the Amey Roadstone Corporation's main operations were concentrated. Three additional wells were inserted on the airfield during April and August 1979 as the gravel workings were extended into new areas. On the 14th November 1978 an Ott Type R16 water level recorder was installed on site R/8.

The Hardwick study area was sited approximately $\frac{3}{4}$ mile east of the village of Hardwick, in an area adjacent to the R. Windrush. Four preliminary tube wells were inserted in January 1979 and an additional five were added during August and September 1979. Four tube wells, SH/51 to SH/54, were installed perpendicular to the river to study the relationship between the R. Windrush and the gravels. The other five boreholes were installed around Wadham-Brasenose Pit to study the flow of groundwater towards the pit.

Observations on the four preliminary wells in the Hardwick area were taken on a monthly basis commencing in February 1979. After the insertion of the further five boreholes, observations were then taken at weekly intervals until July 1980.

In addition to the monitoring of groundwater levels, the levels of the major lakes in the Stanton Harcourt and Ringwood study areas were regularly monitored. This was to study the general fluctuation of lake levels, relating this to the hydrological environment, with special emphasis given to net exchange of water between the lakes and the adjacent gravels. The aim was to see if there was any way in which the future level of flooded gravel pits could be predetermined. This information would be vital for designing restoration schemes for worked out pits.

The usual method of measuring lake levels was to observe the position of the water surface against a stageboard driven into the base of the lake (see plate 4.1 in Appendix A1). A total of 15 stageboards were used and these were placed in the larger lakes, where access was allowed. Four of these boards were in the Stanton Harcourt study area, and the remainder were in the Ringwood area. This reflects the greater density of flooded pits there. All the stageboards in the Ringwood study area had been inserted prior to October 1977 by the Wessex Water Authority. They had also installed Ott Type R16 continuous recorders on the two major lakes in the area, Ivy Lane Lake and Spinnaker Lake. Continuous recorders were also employed on Ibsley Airfield from May 1979, to monitor the water levels in three of the cells. One recorder was permanently sited on cell [illegible] as a 'control', while two others were moved between cells. One recorder was used to monitor the water level in the bottom of the particular cell which was at that time being dewatered, whilst the second was placed on the adjacent cell into which the water was being discharged. The method used to construct the lake level recorder sites will be considered later in this chapter.

No stageboards had been inserted in the Stanton Harcourt study area prior to October 1977. In February 1978, stageboards were placed in Stoneacres Lake, Willow Pool and in the lake next to Linch Hill Cottages. One other was inserted in Teal Reach in April 1979.

Each stagemarker was inserted in that part of the lake which seemed to have the most sheltered position. The large seasonal range in water level in many of the lakes, meant that two stageboards had to be inserted - one to cover the winter levels and another to cover the summer levels. Great difficulty was had in making these sites safe. Many of the lakes were open to the general public as recreational areas so vandalism was a constant problem.

One further stageboard was used. This was located in the R. Windrush adjacent to the Hardwick site. This was used to monitor river stage in relation to groundwater levels in the adjacent gravels.

The method of calculating groundwater levels and lake levels involves subtracting the recorded depth-to-water level from the height of the borehole or stageboard top, the results being expressed in metres above ordnance datum. All well and stageboard tops were tied in to ordnance datum by careful levelling.

Emphasis has so far been on the major factors which have influenced the location of measurement sites within the three study areas. Further discussion is needed on the mode of installation and operation of not only the boreholes and the continuous water-level recorders, but also the other equipment used.

4.1 The Observation Wells

This description applies to those tube wells inserted in the study areas, between June 1978 and September 1979, by the author.

The tube wells were constructed from perforated polythene pipe of 3.8 cms diameter. The perforation consisted of several lines of 0.5 cm diameter holes located about 2.5 cms apart, staggered so that the well was in contact with every part of the aquifer. The holes were drilled in the pipes using an electric hand drill. The pipe was purchased in 3 metre lengths, but these would be joined together with polythene connector-rings to increase the length to that required. A polythene cap was fastened at the lower end of the tube to prevent sediment being forced inside the well as it was inserted into the ground.

It was found, when drilling within the saturated zone of the gravels, that the sand and silt fraction was in a fluid state. It seemed probable, therefore, that fine material would be washed into the boreholes through the perforations and could cause them to silt-up. Consequently, it was decided to wrap the tubes in a fine muslin cloth which would act as a filter. It was not known how this material resisted deterioration, particularly in the saturated zone where it would be open to chemical attack, but it was thought that it would be effective at least until the material around the tube wells had stabilised.

An important consideration in designing the boreholes was the estimation of the required tube well depth. Some of those inserted prior to October 1977 were found to become dry during the summer months. The depth had to be sufficient to maintain contact with the water-table throughout the year. Fortunately, some estimate could be made using well records previously obtained by Oxford University in the Stanton Harcourt study area and by the Wessex Water Authority in the Ringwood study area. At those sites where no previous records were available, the decision was deferred until drilling had commenced. On reaching the water-table, the point at which the gravels became saturated, drilling was continued for a further 2 metres (sufficient, it was assumed, to cover the seasonal range in water levels), or until the base of the gravel was reached. In all but one case, this turned out to be entirely satisfactory.

Because of their simple construction, the tube wells were made up to the required length on site. Usually, an additional length of unperforated tubing was added to project above the ground surface to prevent inseepage of surface water. Where it had been specified by the farmer or land owner, some tube wells were cut off flush with the ground surface and an access hole constructed around each tube using concrete blocks to prevent the site from becoming covered with soil. A polythene screw-cap was firmly attached to the top of each tube well to prevent precipitation falling directly into the tube (see plate 4.2).

4.2 The B40L Auger drill

The drilling of the boreholes was achieved with the use of a 10-inch diameter hollow-stem auger on a B40L drilling rig. This had the ability to drill to a maximum depth of 7.6 metres, which was sufficient to penetrate fully the thickest gravels in each study area. The gravels proved easy to auger, although progress became more difficult where the gravels became clayey. The main problem was the presence of saturated sand and gravel, which because of their fluid state obliterated any attempts to drill the holes using a continuous flight auger. This method was attempted first, but proved unsuitable. When the augers were withdrawn, the saturated gravel was not able to support itself and consequently slumped into the cavity, preventing insertion of the tubing. The use of hollow-stem augers completely eliminated this problem.

Once the required depth of hole had been drilled, the augers were removed from the hole leaving the outer linings in place to support the cavity walls, and the prepared tube (see section 4.1) was inserted into the hole. The lining was then removed, allowing the walls of the hole to collapse around the tube, thus anchoring it in place. Finally, augered material was packed and compressed into the enlarged hole around the tube to prevent seepage of rainfall down the sides of the tube well.

The B40L rig was originally equipped with continuous flight augers which were used for prospecting purposes by the Amey Roadstone Corporation. The hole is drilled to the required depth and then the flights and bit are withdrawn with the aim that a cavity should be left into which a length of tubing is inserted. Mention has already been made of the reasons why this method was not successful.

The hollow-stem augering method which was finally used is based on similar principles. Lengths of hollow-lining, which surround the auger flights, advance with the drilling bit and are used to temporarily case the cavity when the inner auger flights are withdrawn. Although this is a more time consuming method, it was still possible to insert up to six wells in one day. The B40L was operated by the drilling team from ARC (Southern).

4.3 The OTT type KLT Electric Contact Gauge

This instrument was used for the measurement of groundwater depths. It is also capable of measuring the in situ water temperature at any desired depth within the borehole. The gauge consists of a metal probe of 18 mm diameter attached to a measuring tape which is graduated in centimetres and of sufficient length to reach to a depth of 100 metres (see plate 4.2).

The operation of the instrument is extremely simple. The probe is lowered down the tube well until its tip touches the water surface. Because of the conductivity of the water an electric switch is closed, and a signal lamp on the cable drum lights up. A short upward and downward movement of the probe allows an exact sensing of the water level, and the distance between the water surface and a reference edge (generally the top of the tube well) is read from the tape. Two or three readings were taken to reduce observation error.

For the determination of the water temperature, the probe must be submerged. At the desired depth, the temperature (in degrees centigrade) can be read from a scale disk. The disk is turned until an electrical bridge in the point of the probe is balanced. This is indicated by the alternate flashing of the two signal lamps. The exact value is obtained by turning the scale disk slowly clockwise until the switch-over moment of the lamps. The value is then read from a graduation on the window dial. The instrument is capable of measuring water temperatures in the range 0-20°C, to an accuracy of 0.1°C.

4.4 Autographic Water Level Recorders

This section deals particularly with those recorders located at sites R/8, R/54, R/55, and R/56 (see fig. 4.2), although the two recorders installed at sites R/9 and R/10 by Wessex Water Authority are of similar type and construction.

Two types of recorders were used - three OTT Type R16 recorders and one OTT Type X recorder. These instruments are all float operated. The float is attached to one end of a cable which by suitable grooved pulleys, operates an ink pen, so tracing a line on a special paper chart.

The Type R16's are vertical recorders, which were calibrated to operate through a range in water level of 50 cms (see Plate 4.3). The chart drum is rotated by a series of cog-wheels which are driven by a spring operated clock. The cogs can be interchanged so enabling 8 or 32 day charts to be used. In this study 32 day charts were used with all four recorders.

The Type X recorder employed at site R/56 works on the same principle as the R16, except that the chart drum is positioned horizontally (see Plate 4.4). The advantage of the Type X is that when the pen reaches either end of its traverse, it automatically reverses its direction of movement. This enables extremes beyond the normal 50 cms range of water level fluctuations to be recorded. When using the R16 recorders, if the monthly range in water level is greater than 50 cms, the record of any rise or fall which exceeds this limit will be lost because the pen arm will be at the maximum points of the chart. This happened on several occasions, particularly at site R/8.

The Type R16 recorders were housed in their own vandal-proof metal boxes, so no further protection or shelter was necessary. The Type X instrument could not be securely fastened, so a wooden cabinet was constructed to house the recorder in the field. The design of the cabinet was simple, just large enough to cover the recorder and hinged to lift in the same direction as the instrument cover. The cover was secured by a strong padlock and all hinges and bolts were placed so that they could not be removed whilst the shelter was locked.

Each recorder was secured to a wooden platform placed over the top of the tube well (in the case of the recorder on R/8), or over the top of a stilling well (in the case of the lake level recorders). The platforms were bolted to a metal frame which was firmly anchored into the ground and holes were cut out from the platform to allow sufficient clearance for the float and counter-weight.

When measuring the water level in open water, where the water surface is disturbed by wave action, it is customary to place the recorder over a stilling well, placed directly in the lake or river, or set back into the bank. Stilling wells were constructed for each of the lake level recorders, but it was decided to place them in the banks to facilitate chart changing and maintenance. The first procedure was to dig a trench into the bank, using a mechanical excavator, into which a suitable length of 20 cms diameter polythene pipe was inserted vertically. The base of the trench was excavated into the material underlying the gravels so that the stilling well was of sufficient depth to cover the entire range of possible lake levels. Concrete was poured into the bottom of the well to make sure that it was firmly anchored and also to prevent groundwater seeping into the well. One end of an intake pipe, made from 4 cms diameter polythene tube, was then inserted horizontally into the stilling well, just above the level of the concrete base, while the other end projected out into the lake. The trench was then backfilled around the stilling well with the material previously excavated and at the same time the metal frame for supporting the recorder was installed. The inlet pipe projected into the lake from the bank and was above the base of the lake, but below the lowest probable lake level, to avoid the pipe becoming blocked by silt or weeds. The top of the stilling well was well above ground level to prevent any possible damage from flooding and to make access for working easier.

Staff gauges, made from steel poles, were also inserted into each of the lakes which were being monitored, to provide a periodic check between the outside water level and the head in the stilling well (see Plate 4.4).

4.5 Weir Tank

A weir tank was constructed to measure the quantity of water abstracted during dewatering of the cells on Ibsley Airfield in the Ringwood study area. The tank was based on specifications, provided by Wessex Water Authority, for a weir tank which had been proved capable of measuring up to 13,000 m^3/d (J. Eastwood, personal communication). However, this tank was bulky and not very easy to transport, so a smaller tank of similar design was built. The original design had three baffles to reduce turbulence, but this was reduced to two in the smaller tank, taking the form of one underflow and one overflow gate. The actual discharge measuring structure employed was a 90 degree V-notch sharp-crested weir (see Plate 4.5). The overflow was collected in a smaller stilling tank, to prevent erosion of the gravel banks, and then piped, by gravity, to an adjacent lake. The weir tank was connected directly to the dewatering pump by a 20 cm diameter metal pipe which was bolted to an inlet orifice at the rear of the tank.

The weir tank had an approximate capacity of 10,000 m^3/d which was far in excess of the proposed pumping rate, but it appeared advantageous to allow some degree of flexibility to measure a greater rate of dewatering in the future. The tank was constructed of welded ¼ inch steel plate and proved to be quite heavy. However, since it was proposed only to use the tank during the various phases of development at Ibsley Airfield, mobility was not too much of a problem. The weir tank was hoisted by crane onto railway sleepers which were used to spread the weight of the tank and prevent it from sinking into the gravels beneath, particularly when full.

Provisions had been made for a stilling well and a water level recorder to be fitted to the tank, but since it was realised that the discharge would be relatively constant during any particular day the recorder was assigned to another site where there was thought to be a greater need. As an alternative, a staff-gauge was installed in the tank to measure the upstream head of water above the crest of the weir plate. The gauge was positioned just in front of the second baffle to avoid the area of surface drawdown over the weir plate and bolted to the side of the tank so not to interfere with the flow pattern over the weir.

The actual discharge over the weir was calculated from a series of rating tables given by Bos (1976). These give the discharge (in litres/sec), for three commonly used sizes of V-notch, as a function of the head (in metres) above the weir crest. The head was measured twice daily, once in the morning and again in the afternoon, between 19.9.79 and 29.4.80. The observations were made by an employee of ARC. The observations stopped on the 29th April 1980, when the pump serving the weir tank broke down. A replacement pump was installed, but this did not supply the tank.

4.6 Rain gauges

Three rain gauges were used in this study, at sites SH/32, R/68, and at Appleslade, Linwood (179 095) (see fig. 2.4). Sites SH/32. and R/68 were installed in February 1978. The rainfall was collected and measured on the same day that the boreholes were monitored. The design of these gauges was based on the Meteorological Office daily rain guage Mark II. They consisted of a 5 inch diameter copper funnel which was supported by an outer vessel made from plastic pipe. The lower end of the outer vessel was firmly fixed in the ground, so that the funnel aperture was level and 12 inches above ground level. The rainfall was initially collected in a glass bottle, but in winter the water was found to freeze and crack the bottle so they were replaced by 1 litre capacity plastic bottles. The capacity of these bottles was sufficient for most occasions. Both gauges were placed in grassed areas and this was kept cut short to reduce the effects of external influences on the total rainfall catch (see Plate 4.6).

The rain gauge at Linwood is privately maintained and has been used to send daily rainfall records to the Meteorological Office since 1916. The gauge is a Meteorological Office daily rain gauge Mark II. This rain gauge was included even though it is outside the actual study area, because it represented the nearest station where daily rainfall records were available. It was not physically possible to read the other two gauges on a daily basis, and there were no autographic rainfall recorders available at the School of Geography. The nearest station to the Stanton Harcourt study area where daily rainfall records were available was at Farmoor Reservoir (451 065). (see fig. 2.1), which also provided daily free-water evaporation data.

SECTION II. PARTICLE SIZE DISTRIBUTION AND SATUATED HYDRAULIC CONDUCTIVITY

CHAPTER 5

PARTICLE SIZE DISTRIBUTION OF THE TERRACE GRAVELS

Introduction

Section II is concerned with the relationship between the particle-size distribution and hydraulic conductivity of the gravel deposits in the Stanton Harcourt and Ringwood study areas. The question is examined in some detail because both factors are important in determining the nature of the flow of groundwater through the gravel deposits. It will become apparent in later chapters that hydraulic conductivity is the major controlling factor in almost all groundwater problems.

The aim of this section will be to examine the association between particle-size distribution and hydraulic conductivity using data collected during the study. The background and methods used to collect this data are described in this and the succeeding chapter. The existence and conductance of groundwater is a function of the pore size, which in turn is partly dependent upon the size of the particles. In chapter 7 the relationship between grain size distribution and hydraulic conductivity is examined statistically, using results from particle-size analyses and point dilution experiments based upon selected boreholes in the Stanton Harcourt study area.

5.1 Particle size analysis - field and laboratory procedures

This section describes the field and laboratory methods which were used in the collection and analysis of samples of gravel from the Stanton Harcourt and Ringwood study areas. Representative samples of gravel from a total of 23 boreholes in both areas were collected for analysis. The samples were collected from the material brought up on the auger flights during drilling by the author. In most cases this method proved adequate. Bulk samples, consisting of smaller sub-samples collected at frequent intervals along the flights, were taken as being representative of the whole of the material encountered along the total length of the borehole. In those cases where there was an obvious change in lithology within the gravels, separate samples were taken from each layer. All samples were placed in heavy duty polythene bags for transport back to the laboratory.

The majority of samples were collected from boreholes drilled for the construction of observation wells. Because the drilling programmes were designed to provide a network of wells for groundwater investigations and not specifically for the description of the gravel deposits, the distribution of sample points was not evenly spaced across the two study areas.

The method which was used for the grain size analysis of the samples was Soil Classification Test 7(A), described in British Standard 1377 (British Standards Institution, 1967). This covers the quantitative determination of the particle size distribution down to the fine sand size (0.06 mm), the combined silt and clay fraction being obtained by differences.

Because of the large weight of material required to obtain a representative analysis, it was first necessary to divide the sample into three or four conveniently sized sub-samples, each weighing approximately 1 kg. The total weight of sample required depends upon the maximum size of material which is present in substantial proportions (i.e.$>$10%). It was estimated that in the majority of samples being investigated, this maximum size would be between 10 and 20 mm. The minimum weights of sample corresponding to these limits are 0.5 to 2.0 kg (British Standards Institution, 1967). In all cases the total weight of samples analysed from each borehole was in excess of 0.5 kg, and in the majority of cases was in excess of 2 kg.

The procedure used in the particle size analysis was as follows;

1) dry sample in oven at 40 - 50^{o}C,
2) organic material removed by treatment with hydrogen peroxide using method described by Jackson et al (1949),
3) sample dispersion carried out using sodium hexametaphosphate as recommended by ASTM (1963),
4) silt and clay fraction removed by wet-sieving on 0.06 mm seive,
5) remaining sample dried in oven at 110^{o}C,
6) dry sieved for 30 minutes,
7) amount retained on each sieve weighed. The weight of

Table 5.1 **Particle size data summarised for the Stanton Harcourt area**

Sample	Percentage by weight			Statistics	
	Gravel	Sand	Silt & Clay	Md50[1]	Dispersion[2]
Borehole SH/36	66.77	23.13	10.10	-1.36	2.69
Borehole SH/37	66.23	24.47	9.25	-1.24	2.48
Borehole SH/38 (A)	57.22	25.81	6.97	)	
Borehole SH/38 (B)	71.64	22.45	5.91	) -1.43	2.60
Borehole SH/40	64.11	18.64	17.25	-1.20	3.16
Borehole SH/41 (A)	48.80	22.57	28.63	)	
Borehole SH/41 (B)	30.20	63.64	6.16	) -0.43	2.59
Borehole SH/41 (C)	60.05	33.49	6.46	)	
Borehole SH/43	75.59	15.91	8.50	-1.99	2.66
Borehole SH/44	70.82	23.19	5.99	-1.60	2.37
Borehole SH/46	57.25	25.39	17.36	-0.83	3.02
Borehole SH/47	79.02	15.87	5.11	-2.28	2.34
Borehole SH/48	68.06	26.37	5.57	-1.47	2.34
Borehole SH/49	65.82	25.37	8.81	-1.44	2.70

1 Mean grain size in phi units ($\phi = -\log_2$ (grain size in mm))

2 Standard deviation in phi units

Table 5.2 Particle size data summarised for the Ringwood area

Sample	Percentage by weight			Statistics	
	Gravel	Sand	Silt & Clay	Md50[1]	Dispersion[2]
Borehole R/13 (A)	12.30	77.30	10.40	1.18	2.05
Borehole R/13 (B)	76.40	13.20	10.40	-1.86	2.61
Borehole R/16	61.80	24.80	13.40	-1.24	2.94
Borehole R/19	81.76	14.60	3.64	-2.88	2.51
Borehole R/21	70.37	22.97	6.66	-2.17	2.92
Borehole R/22	75.91	18.61	5.48	-2.17	2.56
Borehole R/23	60.30	29.40	10.30	-1.13	2.92
Borehole R/24	53.05	36.75	10.20	-0.94	3.17
Borehole R/26	65.51	31.01	3.48	-1.82	2.71
Borehole R/27	67.20	26.39	6.41	-1.87	2.94
Borehole R/28	80.75	15.43	3.82	-2.78	2.52
Borehole R/29	38.40	45.30	16.30	0.04	3.04
Borehole R/30	80.30	15.84	3.84	-3.15	2.77
Sample 1	81.50	12.70	5.80	-2.86	2.71
Sample 2	65.50	28.35	6.15	-2.04	3.11
Sample 3	16.22	80.78	3.00	0.90	1.84

1 Mean grain size in phi units

2 Standard deviation in phi units

material passing the 0.06 mm sieve was added to the calculated weight of silt and clay removed by wet-sieving.

5.2 Discussion of the results

The results of the grain size analyses are summarised in tabular form in Tables 5.1 and 5.2, and diagramatically in figs. 5.1 and 5.2. Rather than tabulating the complete grain size results, the various sieve sizes are incorporated to give three basic grain size fractions. The limits of the fractions were chosen to conform with B.S. 1377 (British Standards Institution, 1967) and are, gravel (> 2.00 mm); sand (2.00 - 0.0625 mm); and silt + clay (< 0.0625 mm). The results for the Ringwood area (Table 5.2) include three samples dug from exposures in and around Ibsley Airfield. Where different samples were taken from one borehole, these are designated A, B, etc. For each sample, the mean grain size and standard deviation has been calculated using the method of moments (McBride, 1971).

The terrace deposits of Stanton Harcourt and Ringwood, although different in lithology and origin (see chapter 3), are very similar in terms of grain size. Both consist predominantly of gravel sizes; the mean gravel content of the Stanton Harcourt and Ringwood samples is 62.97% and 61.70% respectively. The sand fraction is subordinate, 26.88% in the Stanton Harcourt samples and 30.84% in the Ringwood samples. Least importantly, in terms of weight, is the mean silt plus clay content, 10.15% and 7.46% respectively. In both areas, the gravel: sand:fines ratio is approximately 6:3:1.

The majority of samples from the Ringwood and Stanton Harcourt areas indicate that the gravel deposits are coarse and very poorly sorted. The mean size (in phi units) ranges between -0.43ϕ (approximately 1.4 mm) to -2.28ϕ (approximately 4 mm) for the Stanton Harcourt samples, and between -0.9ϕ (approximately 2 mm) to -3.15ϕ (approximately 8 mm) for the majority of the Ringwood samples. The lower mean grain size of R/13A, R/29 and sample 3 from Ibsley Airfield reflect the dominant sand content of these samples. The standard deviation is the usual measure of sorting (Folk and Ward, 1957, Folk, 1966). Based upon the most often used classification scale for sorting, ranging from 0 phi for very well sorted deposits to 4 phi for extremely poorly sorted deposits, all the samples taken from the Stanton Harcourt and Ringwood areas fall within

Table 5.3 **Particle size data summarised for samples taken from Brown Pit (Sample 1) and Dix Pit (Sample 2 and 3) in the Stanton Harcourt area**

Sample	Percentage by weight			Statistics	
	Gravel	Sand	Silt & Clay	Md50[1]	Dispersion[2]
Sample 1	68.50	29.36	2.14	-1.74	2.07
Sample 2	68.65	29.44	1.91	-1.65	2.14
Sample 3	68.07	29.85	2.08	-1.79	2.30

1 Mean grain size in phi units

2 Standard deviation in phi units

the very poorly sorted class (2.0 - 4.0 ∅).

Mean grain size and sorting are important grain-size parameters, since they have been shown to directly affect the ability of a deposit to transmit water. Hazen (1892) was the first to recognise the relationship between permeability and grain-size diameter for uniformal graded sand. Bedinger (1961) found that a linear relationship existed between the logarithm of permeability and median grain size. Several other investigators such as Kozeny (1955), Marshall (1958), Fair and Hatch (1933), and Griffiths (1955) have developed equations based on the relationships of permeability with porosity and grain-size distribution. Masch and Denny (1966) studied the relationship of permeability with the statistical parameters of grain-size distribution by experimentally determining permeability under controlled conditions and evolved a predictive technique for determining the permeability of material in the -1 to +4 ∅ range.

The relationship between various grain-size parameters and permeability as measured by field tests in the Stanton Harcourt and Ringwood areas is discussed in more detail in chapter 7.

In terms of grading characteristics, gravels are the dominant material in the terraces of the two areas (figs. 5.1 and 5.2). There is, however, a great lateral and vertical variation in the grading characteristics of the gravel deposits. They range from pebbly sands, through sands and clayey gravels, to the very coarse gravels containing over 80% of material greater than 2 mm in diameter. The three samples taken at different levels from borehole SH/41 show some of the variations which can occur over very short vertical distances. The samples collected range from true gravels (sample C) through sandy gravels (sample B), to very clayey gravels (sample A). These sharp variations appear typical of most sand and gravel deposits, particularly those deposited under fluvio-glacial conditions.

The coarsest material found in the terrace deposits of the two study areas are the 'openwork' gravels, which were briefly described in Chapter 3. Two samples from lenses of openwork gravel examined in Brown and Dix Pits, Stanton Harcourt (Table 5.3 and fig. 5.1, samples 1, 3) were analysed. Sample 2 was collected from a frost-wedge in Dix Pit. The two samples of openwork gravel are remarkably similar in their grain size distribution, despite being of different ages and location. It is interesting to note that the material taken from the frost-wedge is indistinguishable from the openwork gravel in terms of particle size. All three samples are characterised by a very low silt plus clay content, with correspondingly greater proportions of the gravel and sand size fractions.

The often made distinction between gravel- and sand- dominated systems is that the principal lithotype of the former is framework or clast-supported gravel, with no upper size limit (Rust, 1977). Gravel-sized particles may be present in sandy deposits, but then they are generally matrix supported. The majority of samples analysed from the Stanton Harcourt and Ringwood areas, being gravel-dominated, fall within the first category. Only four samples are sand-dominated (figs. 5.1 and 5.2). In effect this criterion distinguishes between rivers which transported gravel as bedload and sand in suspension, from those which moved a mixture of sand and gravel along the bed. In the former case gravel would have been deposited first during the falling-stage of the river, which was then infiltrated later by sand and eventually silt and clay. Under certain circumstances, however, very little fine material was deposited, so that the gravels are preserved in an openwork condition, having little support between the pebbles. Matrix-supported gravels would have resulted from the simultaneous deposition of sand and gravel.

There appears to be a relationship between the fines content of the terrace deposits in the Stanton Harcourt area and their topographical situation. Deposits at higher levels generally have a slightly higher fines content. Those samples from boreholes situated in the Floodplain terrace (SH/36, SH/37, SH/38, SH/40, SH/47, SH/48, and SH/49) have a mean silt plus clay content of 8.62% (sand = 24.01%, gravel = 67.36%), whereas those samples from boreholes in the Summertown - Radley terrace (SH/41, SH/43, SH/44, and SH/46) have a mean silt plus clay content of 12.18% (sand = 30.70%, gravel = 57.12%). Whether this represents a change in the hydrological conditions of the major rivers during the late Pleistocene period in this area, or whether it represents a change in sediment supply is not clear. The fact that the terraces are lithologically very similar may suggest that the variation is due to hydrological changes affecting river regime, and its ability to transport coarse material. The higher fines content of the older gravels could also be due to the infiltration of finer material at a later stage following deposition of the gravels.

CHAPTER 6

SATURATED HYDRAULIC CONDUCTIVITY AND ITS MEASUREMENT

6.1 The concept and importance of saturated hydraulic conductivity

The solution of groundwater problems requires that the basic hydrogeological parameters governing the groundwater flow system be known. Of particular importance in the context of this study is a proper determination of the nature of groundwater flow through gravel deposits. Since reliable flow determinations are dependent upon the parameters of pit geometry, groundwater gradient and gravel permeability, it is important that the hydrological parameters are measured adequately.

There are two important categories of flow of water in porous media - saturated and unsaturated (Burmister, 1955). In the first, the pore spaces are completely filled with water and the pressure of water is equal to or greater than atmospheric pressure. In the second, the pore spaces are filled with both air and water, capillary suction forces predominate, and the pressure of water is less than atmospheric (Ward, 1975).

Since this study is primarily concerned with flow below the permanent water-table, the distinction should be made between saturated and unsaturated hydraulic conductivity. Whereas under saturated conditions hydraulic conductivity may be regarded as more or less constant for a given material, under unsaturated conditions the hydraulic conductivity varies with the soil moisture content. In saturated soil, all the pore spaces are effectively conducting water through the system. In unsaturated soils, some of the pores are filled with air which reduces the effective cross-sectional area available for flow. The greater the soil moisture content is, the greater will be the effectiveness of the conducting system and hence the greater the hydraulic conductivity will be.

Where hydraulic conductivity is specified in its unprefixed form in this and all subsequent chapters, the reference is to saturated hydraulic conductivity.

The saturated hydraulic conductivity, sometimes referred to as the coefficient of permeability, is represented by the proportionality constant K in Darcy's law, which for saturated conditions is expressed as,

$$v = -K\,(\partial h/\partial l) \tag{6.1}$$

where $\partial h/\partial l$ is the hydraulic gradient.

It has been shown (Watson, 1966) that Darcy's law is also applicable for unsaturated conditions. The Darcy equation for unsaturated flow is expressed as,

$$v = -K\,\nabla\Phi \tag{6.2}$$

where $\nabla\Phi$ is the gradient of total potential. As in equation 6.1, the negative sign indicates that flow of water is in the direction of decreasing head or potential.

Hydraulic conductivity is a measure of the rate at which water will flow through a porous material. Its value depends on the properties of both the permeating fluid and the porous material. By conducting experiements with fluids of varying density ρ and viscosity μ on a variety of sands of diameter d, under a constant hydraulic gradient, Hubbert (1956) observed the following proportionality relationships:-

$$v \propto \rho\, g$$
$$v \propto 1/u$$
$$v \propto d^2$$

Together with Darcy's original observation that $v \propto -\partial h/\partial l$, these three relationships lead to a new version of Darcy's law,

$$v = -\frac{Cd^2\rho\, g}{u}\cdot\frac{\partial h}{\partial l} \tag{6.3}$$

where C is a dimensionless constant which depends upon the influence of the various media properties, apart from the mean grain diameter, which affect flow. For example, the distribution of grain sizes, the sphericity and roundness of the individual grains, and the fabric (i.e. the degree of interconnectedness of the pores between the grains of the material).

Comparison of equation 6.3 with the original Darcy equation (equation 6.1), shows that,

$$K = \frac{Cd^2\rho\, g}{\mu} = \frac{k\rho\, g}{\mu} \tag{6.4}$$

where K is termed the specific permeability and depends upon the nature of the porous material (i.e. $K = Cd^2$), and not on the properties of the permeating fluid.

The parameter k, the specific or intrinsic permeability, has the dimensions L^2 (Todd, 1959), whilst the hydraulic conductivity K has the dimensions L/T. K represents the apparent flow velocity at unit hydraulic gradient. This definition and its corresponding unit of measurement, metres per day, will be used throughout this work. In this and later chapters, permeability and hydraulic conductivity are used synonymously, both referring to the Darcy K.

Fig. 6.1 indicates the range of values of hydraulic conductivity and specific permeability for a wide range of geological materials. The permeability of natural earth materials varies over a very wide range. In practical terms, this implies that an order of magnitude knowledge of hydraulic conductivity is essential.

6.2 The measurement of saturated hydraulic conductivity

The various approaches to the estimation of hydraulic conductivity fall into three groups. These are:

1) general permeability formulae.
2) laboratory methods.
3) field or *in-situ* methods.

Permeability formulae are generally based on the recognition that the hydraulic conductivity of porous media is a function of their grain-size distribution (see chapter 5).

Essentially the problem reduces to relating the proportionality constant C in equation 6.3 to the properties of the material. One of the earliest studies by Hazen (1892) showed that the permeability varied with the square of the effective grain size. More powerful formulae have since been developed and take into account measures such as statistical grain-size distribution parameters (Masch and Denny, 1966), porosity (Kozeny, 1955), or factors describing packing and grain shape (Fair and Hatch, 1933). These and other formulae are described in most hydrogeological texts (eg. Todd, 1959, Freeze and Cherry, 1979).

Haxen's formula is quite useful because of its simplicity, but the result may be inaccurate to the extent of $\pm$ 200% (Loudon, 1952). Where an estimate of the angularity factor of a material can be made, permeabilities using the Kozeny formulae have been computed to an accuracy of about $\pm$ 20% (Loudon, 1952). According to Hazen (1892), however, the use of general formulae based upon the effective size of unconsolidated material is very difficult for effective sizes greater than 3 mm. Most computations have therefore been limited to sands.

Laboratory methods, since they are carried out on small samples of material that are collected in the field, only provide point values of hydraulic conductivity. The most common type of laboratory apparatus for the measurement of hydraulic conductivity is the permeameter. Laboratory measurements may bear little relation to field permeabilities when dealing with unconsolidated deposits, although they may be conducted under controlled conditions. Reconstituting field samples may introduce large errors, up to 3000% according to Kirkham (1955), because of the disturbed nature of the sample, and because of air which may become entrapped within the material.

Many methods exist for the determination of in-situ hydraulic conductivity. The most reliable method is by controlled pumping tests, which have now become a standard procedure in all large-scale groundwater investigations. This method is based upon observations of changes in groundwater levels in and around a pumped well. From the data collected, estimations of aquifer transmissivity (T), defined as T = Kd, and hence hydraulic conductivity, can be made.

Laboratory methods provide only point values of hydraulic conductivity, whereas pumping tests provide in-situ measurements that are averaged over a large aquifer volume. There are disadvantages with the pumping test method however. Some degree of expertise is required in using the wide variety of analytical and numerical solutions now available in order to achieve an accurate estimation of the aquifer properties. For the data to be interpreted correctly, a prior knowledge of the aquifer characteristics and its behaviour are a necessary prerequisite. Three useful general references on pumping test analysis are Kruseman and de Ridder (1970), Walton (1970), and Rushton and Redshaw (1979). On a practical level, a major disadvantage is the great expense involved in carrying out a pumping test. In a study where an estimate of hydraulic conductivity is required, but where it is not involved with well-field development, a pumping test is usually inappropriate and may not be justifiable.

Many simple and relatively cheap field tests are available that provide adequate estimations of hydraulic conductivity. Most can be carried out on a single borehole and provide in-situ estimates of permeability, representative of the aquifer immediately surrounding the test-site. One of the more commonly used methods is the point-dilution test.

Section 6.2.1 describes in some detail the point-dilution method used in this study for estimating the hydraulic conductivity of gravel deposits in the Ringwood and Stanton Harcourt areas. This includes a discussion of the theory of single-point dilution and also a consideration of the choice of tracer and field and laboratory procedures. The results of these experiments are discussed in section 6.2.2. To test the reliability of the single-point dilution method, two calibrations were made. Firstly, to test the accuracy of this technique, estimates of the hydraulic conductivity of gravels in the River Stour valley of Hampshire were compared with those estimated from a pumping test undertaken by the Wessex Water Authority on the same boreholes. Secondly, to test the reproducibility of the dilution technique, the experiment was duplicated on two boreholes.

6.2.1 The single-point dilution method of measuring hydraulic conductivity

a) Theory

By introducing a tracer substance into groundwater at an upstream location and observing the time required for it to appear at a downstream point, estimates of groundwater velocity can be obtained. This information, together with the existing hydraulic gradient, provides a measure of the permeability of an aquifer (see equation 6.1).

The inherent problem of tracer dispersion along preferred flow paths limits the accuracy of the two-well tracer technique. Consequently, the single-well, or point, dilution method was developed. The method is described in detail by Halevy et al. (1967) and Drost et al. (1968).

The basis of the dilution technique is that the concentration of tracer injected into a well diminishes with time due to the flow of groundwater through the saturated section of the well. The apparent groundwater velocity can then be related to the measured reduction in tracer concentration of groundwater samples taken at various times after injection of the tracer.

Lloyd et al. (1979) list four assumptions that are made in carrying out the point-dilution experiments. These are that

- steady-state groundwater flow conditions prevail,
- the tracer concentration is uniform throughout the flow section,
- the tracer concentration diminution with time is due only to laminar flow of water through the well volume being sampled

(i.e. there is no vertical flow component).

. the borehole dimensions are known.

The first condition is satisfied if water-table fluctuations caused by pumping or by sudden water-level changes in nearby surface water bodies do not occur. The second condition generally exists when small sections of a well surrounded by nearly uniform porous material are selected. If the well-water is mixed before sampling, the resulting apparent velocity represents an average for the entire well section. The third condition is usually satisfied when the well penetrates only one water-bearing layer.

A distinction should be made between apparent velocity and average linear velocity. The apparent velocity (also known as the Darcy velocity), v in the Darcy equation (equation 6.1), is an artificial velocity since it equals the total quantity flowing divided by the full cross-sectional area (voids and solids alike). For this reason it is also known as the specific discharge. In actual fact, the flow passes through only that portion of the cross-sectional area occupied by voids. For a total cross-section A_r, a velocity that represents the discharge divided by the actual cross-sectional area occupied by voids, A_v, through which flow occurs, can be defined. This velocity is known as the average linear velocity, $\bar{v}$, and can be measured as the Darcy velocity v divided by a porosity function n. It should be emphasized that $\bar{v}$ does not represent the average velocity of the water particles travelling through the pore spaces (Freeze and Cherry, 1979). These true velocities are generally larger than $\bar{v}$, because the water particles must travel along irregular paths that are longer than the linearized path represented by $\bar{v}$.

The true velocities that exist in the pore channels are seldom of interest, and they are probably impossible to measure (Freeze and Cherry, 1979). In the remainder of this chapter, therefore, the Darcy velocity v is used, following the similar work on point dilution by Lewis et al (1966) and Lloyd et al (1979)[1]. It means, in effect, that the actual porous medium of sand and gravel is replaced by a representative continuum

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1. Lloyd, J.W., Ramanathan, C., and Pacey, N., 1979, 'The use of point dilution methods in determining the permeabilities of land-fill materials', Water Services, vol.83, pp. 843-846.

for which macroscopically averaged descriptions of the microscopic behaviour (such as hydraulic conductivity) can be provided (Freeze and Cherry, 1979). It is apparent therefore, that since porosity values are less than 1, estimates of the Darcy velocity will be less than the average linear velocity. Also, the hydraulic conductivity estimated using the Darcy equation will tend to be lower than the actual values. It is not thought that this invalidates the use of the Darcy velocity, given that it is the order of magnitude of hydraulic conductivity which is important.

Experimental flow pattern studies (Halevy et al., 1967) indicate that the dilution velocity inside a borehole differs slightly from the apparent velocity that exists in the absence of a borehole. The effect of a borehole in a lateral flow regime is shown in fig. 6.2. The apparent groundwater velocity beyond the disturbed flow zone immediately surrounding the borehole is v and the average tracer dilution velocity across the centre of the borehole is denoted by v^*. If a tracer is introduced instantaneously at concentration C_o into the borehole, at time $t > 0$ the concentration C in the borehole decreases at the rate

$$\frac{\partial C}{\partial t} = - \frac{A \cdot v^* \cdot C}{W} \tag{6.5}$$

where A is the vertical cross-sectional area through the centre of the saturated depth of the borehole, and W is its volume. Upon rearrangement equation 6.5 yields

$$\frac{\partial C}{C} = - \frac{A \cdot v^* \cdot \partial t}{W} \tag{6.6}$$

Integration and use of the initial condition $C = C_o$ at $t = 0$ leads to

$$v^* = - \frac{W}{A.t} \ln \left(\frac{C}{C_o}\right) \tag{6.7}$$

Thus, values of v^* can be computed from data on concentration versus time obtained during point-dilution tests. Estimates of v can be made using the relation

$$v = \frac{v^*}{\alpha} \tag{6.8}$$

where α is a correction factor for hydraulic distortion.

The effect of any hydraulic distortion by the construction of a borehole is difficult to assess, but will obviously be very variable. Gasper and Oncescu (1972) assume in practice that $\alpha = 1$, which is reasonable for small boreholes of up to 100 mm diameter. Since the majority of boreholes used were of a maximum diameter of 55 mm, it is assumed for practical purposes that $v = v^*$ in all cases.

b) Considerations on the choice of tracer

Numerous types of tracers are available for the study of groundwater movement. Organic dyes and salts have been used for many years (Dole, 1906; Slichter, 1905), while more recently many other types of tracers have been employed, including radioactive isotopes (eg. Fox, 1952; Drost et al., 1968), and a wide variety of natural and introduced soluble chemical tracers (eg. Kaufman and Todd, 1955; Waldmeyer, 1958; Dyck et al., 1976). The type of tracer used depends upon the nature of the problem, the amount of time and finance available and the accuracy of the results which are required.

Because of their ready availability, low cost, and ease of detectability, fluorescent dyes are one of the most widely used tracers in groundwater hydrology (Smart and Laidlaw, 1977). Many different dyes are available. Fluorescein was the first to be widely used, but has since been found to be far from ideal. Fluorescein is decolourized by contact with humus, clay minerals and calcite (Atkinson et al., 1973). The most widely used dyes today belong to the rhodamine group and include rhodamine B, sulpho-rhodamine B, and rhodamine WT (which was developed specifically for tracing work). Other dyes which have been used include pyranine, a group of blue fluorescent dyes known as optical brightners (Glover, 1972) such as Photine Cu, and two dyes which were originally used for aerosol tracing, lissamine FF (Yates and Akesson, 1963) and amino G acid (Dumbauld, 1962).

For hydrogeological purposes, an ideal dye tracer should be water soluble, detectable at very low concentrations, absent from natural waters, have low absorbant properties, be harmless and readily available at low cost. Smart and Laidlaw (1977) compared eight fluorescent dyes in laboratory and field experiments to assess their performance in water-tracing experiments. The properties considered included sensitivity and minimum detectability, the effect of water chemistry on dye fluorescence, photochemical and biological decay rates, adsorption losses on equipment and sediment, toxicity and cost. Three dyes were recommended, one of which - Rhodamine WT - was used in these dilution experiments.

The properties of rhodamine WT and its suitability for hydrological work are described by Smart and Laidlaw (1977). The main properties are summarized briefly below,

- low minimum detectability - 0.013 μg/l
- moderately high temperature sensitivity
- fluorescence stable in the range pH 5 - 10
- suspended sediment concentrations above 1000 mg/l cause maximum reduction in fluorescence of only 40%
- low photochemical decay coefficient
- low background fluorescence in most natural waters
- adsorption onto organic and inorganic sediments moderate compared with most other commonly used dyes
- adsorption onto polythene sample bottles minimal.

c) Field and laboratory procedures

The field procedures for the point dilution technique were principally developed by Lewis et al. (1966), but have been modified slightly in this study.

A sufficient quantity of dye in a solution of distilled water was injected into each borehole to bring the initial concentration in the groundwater in the borehole to approximately 1 ppm. The volume of dye injected in each test was 100 ml. The initial concentration of dye to be injected was calculated using the equation

$$C_i = \frac{C_f . V_f}{V_i} \tag{6.9}$$

where C_i is the concentration of the dye to be injected, C_f is the intended concentration of well water (i.e. 1 ppm), and $V_f = V + V_i$ (where V is the volume of water in the borehole and V_i is the volume of dye to be injected, i.e. 100 ml).

The dye was injected into the boreholes by pouring the solution into a funnel connected to a length of polythene tube, the bottom end of which was submerged below the water-level. A weight attached to the bottom of the tube helped in inserting the tubing down the borehole and by a surging motion helped mix the dye throughout the well-water. This was done for approximately five minutes before sampling commenced to ensure as far as possible that the well-water was of a uniform concentration.

To sample the well-water, a Rock and Taylor Automatic Water Sampling Machine was used (Appendix A2). A total of 24 boreholes were tested;

Table 6.1 Results of the point dilution experiments

Borehole	Velocity[1]	Hydraulic Gradient	Hydraulic Conductivity[1]
SH/12	0.037	7.10×10^{-4}	51.75
SH/13(A)	0.019	9.72×10^{-4}	19.47
SH/13(B)	0.017	9.70×10^{-4}	17.97
SH/14	0.024	6.49×10^{-4}	36.60
SH/21	0.009	2.95×10^{-4}	30.50
SH/36	0.034	8.66×10^{-4}	39.25
SH/38	0.075	1.67×10^{-3}	44.91
SH/39	0.108	1.37×10^{-3}	78.61
SH/40	0.034	5.00×10^{-3}	6.80
SH/41	0.053	7.30×10^{-3}	7.26
SH/43	0.147	1.50×10^{-2}	9.80
SH/44	0.051	3.40×10^{-3}	15.00
SH/46	0.030	3.42×10^{-3}	8.77
SH/48	0.120	3.88×10^{-3}	30.93
SH/50	0.034	7.91×10^{-3}	4.30
SH/51	0.023	3.76×10^{-3}	6.11
SH/52	0.038	4.76×10^{-3}	7.98
SH/53	0.103	5.94×10^{-3}	17.34
SH/54	0.043	6.79×10^{-3}	6.33
R/2	0.053	7.55×10^{-4}	70.21
R/8	0.233	7.53×10^{-4}	309.57
R/22	0.038	5.40×10^{-4}	70.36
R/33 (A)	0.013	7.40×10^{-4}	17.56
R/33 (B)	0.019	6.70×10^{-4}	28.34
R/52	0.041	9.49×10^{-4}	43.22
R/53	0.046	1.99×10^{-3}	23.07

1 Values in m/d

18 from the Stanton Harcourt area and 6 from the Ringwood area. On completion of each sampling cycle the samples were returned to the laboratory and analysed, within 24 hours, on a Turner Designs 10 - 005 Fluorometer (Appendix A2).

On completion of the sample analyses, the corrected values of dye concentration (see Appendix A2) were plotted against time on semi-logarithmic graph paper. By fitting a regression line of the form $y = b_o + b_i x$ to the plot, the tracer dilution velocity v^* was calculated by substituting any two values of the tracer concentration (C_o and C) estimated from the regression equation, and the respective time interval t, into equation 6.7

Fig. 6.3 shows the corrected Rhodamine WT concentrations (Cc) plotted against time (t) for three boreholes. The computed regression - lines are also shown. The strongly linear characteristics of the plot for observation well R/8 is typical of most boreholes tested and shows that the effect of groundwater throughflow produces an exponential relationship between dilution and time. It is apparent that the data for boreholes SH/38 and SH/48 provides little meaningful information after 30 and 40 hours respectively. In cases such as these, the analyses were confined to the data from the initial part of the tests.

The saturated hydraulic conductivity was calculated by inserting the value for the average apparent groundwater velocity v (where $v = v^*$) into the Darcy equation.

$$K = v / i \tag{6.10}$$

where i is the hydraulic gradient for the particular site measured at the time of the dilution experiment (Lloyd et al., 1979).

6.2.2 Discussion of the results from the point dilution tests

The calculated saturated hydraulic conductivity values are shown in Table 6.1. The mean K values for Stanton Harcourt and Ringwood are 23.39 m/d and 77.05 m/d respectively. These figures are well within those normally quoted for sand and gravel mixtures (see fig. 6.1). The wide range of values found in such relatively small areas is not unusual. Even greater ranges have been reported from gravels elsewhere. Ridings et al. (1977) found that the hydraulic conductivity of terrace gravels in the Thames valley near Maidenhead ranged between 300 and 3000 m/d.

Table 6.2 A comparison between hydraulic conductivity and grading characteristics

Gravel[2]			Clayey Gravel[3]		
Borehole	Hydraulic Conductivity[1]	Silt & Clay (%) (0.06 mm)	Borehole	Hydraulic Conductivity[1]	Silt & Clay (%) (0.06 mm)
SH/38	44.91	6.44	SH/36	39.25	10.10
SH/43	9.80	8.50	SH/40	6.80	17.25
SH/44	15.00	5.99	SH/41	7.26	13.75
SH/48	30.93	5.57	SH/46	8.77	17.36
R/22	70.36	5.48			
mean	34.20	6.40	mean	15.52	14.62

1 Values in m/d

2 < 10% fines

3 > 10% fines

The variability in hydraulic conductivity is due to the nature of the materials, particularly the interdigitation of beds ~~of~~ very different grain-size distribution with rapid transitions between them. A comparison of the grading characteristics of samples of gravel from a number of boreholes in the Stanton Harcourt and Ringwood areas with their measured hydraulic conductivity is shown in Table 6.2. There appears to be a trend, although clearly the correlation is not that good, which suggests that the hydraulic conductivity is influenced by the silt plus clay content of the gravels. The hydraulic conductivity of those samples with more than 10% fines content is less than 50% that of the samples with less than 10% fines. The sample taken from borehole SH/36 has a relatively high K value compared with other wells in the same grading category. This sample has a total fines content of about 10%, which places it directly on the dividing line between 'gravel' and 'clayey gravel'. The association between hydraulic conductivity and grain size distribution is examined further in chapter 7.

a) Comparison with estimates from a pumping test

The point dilution method yielded results in good agreement with pumping test data. In November 1978 a constant rate pumping test was carried out by the Wessex Water Authority in medium to coarse gravels on the south side of the R. Stour near Longham, Dorset (map reference SZ 060970) (fig. 6.4). These are predominantly flint gravels, and are of a similar nature to those in the Ringwood area. In February 1980, a point dilution experiment was carried out in the same borehole to assess the reliability of this method for hydraulic conductivity determinations.

A production borehole on the site was pumped at 2270 m^3/d for approximately 48 hours. Measurements of the water-table level during the drawdown and recovery periods were made at two observation boreholes at distances of 10 metres and 25 metres from the pumped well. Using a type-curve analysis of the very early data (i.e. from 1 to 5 minutes after the start of pumping), a transmissivity value of 320 m^2/d was obtained (Heath, 1980). Alternatively, the distance-drawdown data for the latter part of the test produced transmissivity values between two and five times greater. The depth of gravel in this area (d) varied between 1.5 and 7.0 metres.

One of the problems encountered during the data analysis was caused by the fact that the groundwater system progressed from a confined to an unconfined state during the initial stages of pumping. This probably accounts for the discrepancy between the Kd value calculated for the early part of the test and that calculated for the latter part.

To determine the validity of the manual analyses, a series of computer simulations were carried out by the Wessex Water Authority (Heath 1980). The numerical model used is described by Rushton and Redshaw (1979, page 231).

The first simulations were performed using the assumption that the hydraulic conductivity of the aquifer did not vary with distance from the pumped well. The results of a typical analysis are plotted against the field data in fig. 6.5 (run 3). These results showed that by using a K value of 116 m/d, the pumped well went dry within the first few minutes of pumping, while the gradient of the observation borehole time-drawdown curve was less than that produced by the field data.

In an aquifer of unconsolidated sands and gravels, variations in permeability (as noted in the Stanton Harcourt and Ringwood areas) are to be expected. An advantage of the digital model used was that permeability changes could be simulated in various parts of the aquifer. The most satisfactory fit to the observed drawdown curve was achieved using a basic aquifer permeability value of 90 m/d, and by increasing the permeability, between the production borehole and a radius of 17 metres, threefold (run 18 in fig. 6.5).

The comparison, using the point dilution method, was carried out on the observation borehole situated 25 metres from the production borehole. The method used was exactly the same as that described in section 6.2.1. The estimated hydraulic conductivity value is 76.8 m/d, which is within 15% of the basic permeability value of 90 m/d estimated from the computer simulations.

b) <u>Duplicate tests on same borehole</u>

As a test of the reproducibility of the point dilution method, the experiments were repeated on two boreholes (SH/13 and R/33). The hydraulic conductivity results are shown in Table 6.1. If a method is to be considered sufficiently accurate for hydraulic conductivity measurement, the results should be reproducible under similar conditions. The results, particular from borehole SH/13 (19.47 and 17.97 m/d respectively) provide further evidence that point dilution is a reliable method of measuring hydraulic conductivity in the field, despite the many possible variables involved in conducting these experiments.

The results have shown that the single point dilution method, using dye tracers, is a reliable and sufficiently accurate technique for determining values of <u>in situ</u> saturated hydraulic conductivity in coarse-grained aquifers and also that elaborate, painstaking (and often expensive) measurements are not always required for field hydraulic conductivity determinations. The variability of permeability, particularly in gravels, is usually so large that precise values are not necessarily warranted or even useful. It seems far more acceptable to use a quick, relatively inexpensive method, such as single point dilution, which can be attempted at a large number of sites to achieve a more representative sample of permeability values.

CHAPTER 7

THE ASSOCIATION BETWEEN SATURATED HYDRAULIC CONDUCTIVITY AND PARTICLE SIZE DISTRIBUTION

7.1 Introduction

The aim of this chapter is to examine statistically the association between hydraulic conductivity and grain size distribution. The analysis is limited to eight sites in the Stanton Harcourt area for which their particle-size distribution (see chapter 5) and saturated hydraulic conductivity values (see chapter 6) have been determined. A similar analysis was not carried out for the Ringwood area, because there were not a sufficient number of sites for which particle-size and hydraulic conductivity data was available.

The statistical analysis is divided into two stages. The association between hydraulic conductivity and particle size is first tested using Spearman's rank correlation coefficient (r_s). The second stage uses factor analysis to generate a number of independent components, whose factor scores are incorporated in a multiple regression model, with hydraulic conductivity acting as the dependent variable. This step removes the problem of intercorrelations between the grain-size classes and permits the use of regression analysis to predict hydraulic conductivity from the grain-size data. The results indicate those combinations of grain sizes which have most influence upon hydraulic conductivity.

There have been several other investigations of the relationship between hydraulic conductivity and particle-size distribution. Slater and Byers (1931) examined the hydraulic conductivity of six humid-forest soil types in Virginia, U.S.A., ranging from clay to loamy sand. They found a negative relationship between hydraulic conductivity and the percentage silt content, but no statistical testing was carried out.

Aronovici (1946) investigated light sandy sediments in Imperial Valley, California. A statistical analysis of these sediments revealed a highly significant correlation ($r = -0.85$, $n = 83$) between the percentage of fines (silt plus clay) and hydraulic conductivity. This was interpreted as indicating that increasing amounts of silt and clay have a greater effect in blocking the pore channels in fine sands.

More recently Bonell (1971, 1976) investigated the relationship between hydraulic conductivity and the texture of glacial till deposits in Holderness, East Yorkshire. Correlation analysis suggested a positive correlation with medium silt (r_s = 0.69, n = 12), and a negative correlation with fine sand (r_s = -0.68, n = 12) and medium sand (r_s = -0.56, n = 12). These results were emphasized by multiple regression analysis based upon principle component scores. It was suggested that medium silt could have been important in the formation of prismatic structures in the shallow layers of the till, producing conditions which favour higher saturated hydraulic conductivity.

The methods used in the proceeding analysis are based largely on those described by Bonell (1971); however, no other reference to a similar study on coarse-grained gravel deposits has been found. It is assumed, following Aronovici (1946), that there is a strong causal relationship between particle size and pore size, so that texture can be related to hydraulic conductivity without a detailed knowledge of the structure of the deposit. Where fine-textured materials are being investigated, the large silt and clay content produces aggregate structures which may well influence hydraulic conductivity to a greater extent than the individual grain-sizes. However, when coarse grained sediments with discrete individual particles are considered, Aronovici's assumption is thought to prevail. A test of the association between hydraulic conductivity and the grain-size of coarse grained materials, such as sands and gravel, should therefore be more useful in indicating which particle size class most influences hydraulic conductivity. From the previous papers it is possible to set up a research hypothesis that in coarse gravel samples, clogging by silt and clay is likely to be very important (i.e. $K \propto 1/(\text{silt} + \text{clay})$), and therefore to test the null hypothesis that there is no correlation between grain-size and permeability.

7.2 Spearman's Rank Correlation

The simplest and most convenient method of examining the relationship between two or more variables is correlation analysis. Several types of correlation are available, but only Spearman's non-parametric rank correlation coefficient is appropriate for eight samples (Siegel, 1956).

Eight boreholes were initially chosen for this investigation. These were boreholes SH/36, SH/38, SH/40, SH/41, SH/43, SH/44, SH/46, and SH/48. This list of sites represents all those boreholes for which

Table 7.1 The saturated hydraulic conductivity (K) and particle size data

Borehole	K (m/d)	Coarse gravel (> 16 mm)	Fine gravel (16–2mm)	Coarse sand (2–1mm)	Medium sand (1–0.25mm)	Fine sand (0.25–0.06mm)	Silt & clay (< 0.06mm)
SH/36	39.25	11.43	55.34	6.29	12.22	4.61	10.10
SH/38	44.91	13.71	50.72	7.01	14.77	7.35	6.44
SH/40	6.80	17.44	46.67	4.65	9.80	4.19	17.25
SH/41	7.26	4.38	41.97	11.66	21.40	6.84	13.75
SH/43	9.80	17.23	58.36	4.72	7.33	3.87	8.50
SH/44	15.00	8.12	62.71	6.06	13.44	3.70	5.99
SH/46	8.77	8.42	48.82	4.89	17.45	3.07	17.36
SH/48	30.93	8.30	59.76	7.79	14.26	4.33	5.57

Note: Particle size data expressed as weight per cent of the original samples. The totals do not add up exactly to 100% due to rounding errors.

Table 7.2 Spearman's rank correlation between saturated hydraulic conductivity (K) and grain-size groups for samples taken from the Stanton Harcourt area

Grain-size variable correlated with K	Spearman's rank correlation coefficient (r_s)	Level of significance (p)
Coarse Gravel (>16 mm)	-0.024	Not significant
Fine Gravel (16-2 mm)	0.571	Not significant
Coarse Sand (2-1 mm)	0.405	Not significant
Medium Sand (1-0.25 mm)	0.000	Not significant
Fine Sand (0.25-0.06 mm)	0.405	Not significant
Silt & Clay (<0.06 mm)	-0.667	$p = 0.05$

grain-size and hydraulic conductivity data were available. Although the number of suitable sites available was limited, they cover a fairly representative range of the hydraulic conductivity values found in the study areas. Six particle size groups were used in this study, based on those defined by the British Standards Institution (1967) (see Table 7.1). The results of the correlation analysis are shown in Table 7.2. A comparison is also made between hydraulic conductivity and the two statistical grain-size parameters (Md_{50} and sorting) estimated in chapter 5 (see Table 6.1).

The most significant feature is the negative correlation between hydraulic conductivity and the silt + clay content ($r_s = -0.667$, $p < 0.05$). This result, even though the level of significance is lower, agrees with that noted by Aronovici (1946). The conclusion is that increasing amounts of silt and clay decrease the hydraulic conductivity of gravels by blocking or reducing the size of the pore channels. This is more remarkable since the silt + clay content constitutes only a relatively small percentage of the samples analysed, ranging from 5.57 to 17.36%. The null hypothesis set up in section 7.1 can therefore be rejected.

Masch and Denny (1966), using statistical grain size parameters to predict the permeability of unconsolidated sediments, noted that permeability was dependent to some extent on the median grain size (Md_{50}), increasing with an increase in Md_{50}. This relationship would be expected since for larger particle diameters there is an increase in the pore space between grains and consequently a greater flow area.

The median grain size calculated for each of the eight boreholes used in the correlation analysis ranges from -0.43 to -1.99 ϕ (Table 6.1). These values fall mainly within the fine gravel grain size group, which makes up by far the largest fraction in terms of weight (41.97% to 62.71%).

The standard deviation, which is used as a measure of sorting was also calculated for each of the eight boreholes (Table 6.1). Values ranged from 2.34 to 3.16 ϕ, which are indicative of very poor sorting. Masch and Denny (1966) found that the hydraulic conductivity of unconsolidated sediments decreased with increasing standard deviation about the mean, since the greater the range between maximum and minimum grain size, the greater will be the opportunity for interstitial clogging to occur. The results in Table 6.1 tend to support this argument, since the two boreholes with the greatest standard deviation (SH/40 and SH/46) were found to have very low hydraulic conductivity (6.8 and 8.77 m/d, respectively).

7.3 Factor Analysis

Introduction

Having investigated the relationships between particle size groups and hydraulic conductivity, a further consideration was the possibility of using particle size data to predict hydraulic conductivity in gravels. A convenient method for the development of a prediction equation is through the use of multiple regression analysis, with hydraulic conductivity acting as the dependent variable and the six particle size groups as the independent variables. The theory of multiple regression and method of derivation is described by Ezekiel and Fox (1963), Snedecor (1956) and Riggs (1968). The advantage of a multivariate approach is that much more of the information contained in the full set of grain size data can be used, rather than just single grain size classes or simple statistical measures (eg. Md_{50}).

Preliminary trials using the original set of untransformed data showed that a high degree of multicollinearity exists between the grain size variables. Multicollinearity refers to the situation in which some or all of the independent variables are inter-correlated. This phenomenon recurred after various data transformations, to ensure approximate normality, had been applied. When extreme multicollinearity exists there is no acceptable way to perform regression analysis using the original set of independent variables (Bonell, 1971). One solution is to create artificially, a new series of uncorrelated variables, each of which is a composite scale of the set of intercorrelated variables, and to use these new components in the regression equation in place of the initial variables.

As a result a factor-analytical technique was used. The complicated computations were achieved by using a standard statistical computer package described by Nie et al. (1975).

The advantage of factor analysis is that it reduces the dimensionality of a set of variables by taking advantage of their intercorrelations. This is achieved by finding the principal axis (or component) of the hyperellipsoid in the N space along which there is maximum variance (Bonell, 1971). The symbol N depends upon the original number of variables being examined, in this case the six grain-size groups. The analysis then constructs further axes in turn, each orthogonal to the previous ones, until all the variance is accounted for. There will therefore be the same number of components as the original number of variables. These newly developed components are totally uncorrelated and independent, so rendering them ideal for use in multiple regression analysis.

Table 7.3 Grain-size data for six additional samples used from the Stanton Harcourt area in the factor analysis

Borehole/ Sample	Coarse gravel (>16 mm)	Fine gravel (16-2 mm)	Coarse sand (2-1 mm)	Medium sand (1-0.25 mm)	Fine sand (0.25-0.06 mm)	Silt & clay (<0.06 mm)
SH/37	23.42	55.60	5.22	8.37	2.28	5.11
SH/47	6.17	60.11	8.47	12.01	4.00	9.25
SH/49	14.87	50.95	6.53	13.90	4.94	8.80
Sample 1	11.75	56.75	8.85	18.80	1.71	2.13
Sample 2	9.74	58.91	7.91	17.45	4.07	1.91
Sample 3	16.99	51.08	8.76	17.55	3.54	2.07

Note: Particle size data expressed as weight per cent of the original samples. The totals do not add up exactly to 100% due to rounding errors.

Table 7.4 The correlation between untransformed medium sand and various transformations of the remaining grain size groups

Grain-size group	Transformation	Pearson's correlation coefficient (r)
Coarse gravel	logarithmic	-0.58
Coarse gravel	square root	-0.59
Coarse gravel	square	-0.60 *
Fine gravel	logarithmic	-0.38 *
Fine gravel	square root	-0.37
Fine gravel	square	-0.35
Coarse sand	untransformed	0.75 *
Coarse sand	logarithmic	0.74
Fine sand	untransformed	0.20
Fine sand	square	0.27 *
Silt & clay	untransformed	-0.11
Silt & clay	logarithmic	-0.27 *
Silt & clay	square root	-0.19

Note: Selected transformations shown by asterisk (*)

Table 7.5 The principal components

Component number	Eignenvelue or component variance	% of total variance	Cumulative %
1	2.60413	43.4	43.4
2	1.72572	28.8	72.2
3	0.87343	14.6	86.7
4	0.55261	9.2	95.9
5	0.22300	3.7	99.6
6	0.02110	0.4	100.0
total variance	6.00000		

To ensure that the results of the factor analysis were as accurate and as representative of all the gravels as possible, the number of samples on which the analysis was increased to fourteen. Six additional sites from the Stanton Harcourt area for which grain size, but not hydraulic conductivity, data were available (Table 7.3) were included. Three of these sites were boreholes (SH/37, SH/47, SH/49), whereas the remaining three were samples collected in the field (sample 1 from Brown Pit (stage C), and samples 2 and 3 from Dix Pit).

Factor Analysis Calculations

The three main steps in factor analysis are (1) the preparation of a correlation matrix, (2) the extraction of the initial factors and the exploration of possible data reduction and (3) the rotation to a terminal solution and the search for simple factors.

Factor analysis requires parametric correlation coefficients (Cooley and Lohnes, 1962), so the very first step in the analysis was to transform the original data in such a way that it approached a normal distribution. Logarithmic and root transformations are frequently used on positively skewed distributions, whereas antilogarithmic and power transformations are used for normalising negatively skewed distributions.

If a transformation produces an approximately normal distribution, then the transformed values will have a significant correlation with an untransformed, normally distributed variable. Calculations proved that only the medium sand fraction of the samples approached a normal distribution. To ascertain the most suitable transformation for each of the remaining grain size groups, up to three transformations were considered for each variable. Table 7.4 shows the different transformations attempted for each variable and their respective correlation coefficients with untransformed medium sand. The transformations which gave the highest correlation coefficients with medium sand were then adopted for use in the factor analysis.

The second step in factor analysis is to construct a new set of variables, based upon the interrelationships exhibited in the original data. The new variables are defined as exact mathematical transformations of the original data, using a method known as principal-component analysis. This is a method of transforming a given set of variables into a new set of composite variables or principal components that are orthogonal to (uncorrelated with) each other.

The six new components, with their corresponding eigenvalues and accumulated variance are shown in Table 7.5. The eigenvalue is the total amount of variance accounted for by each component and is a measure of the relative importance of the function. Since all the variables are normalised, the variance of each is equal to 1. Thus, the total variance in the data equals the number of variables in the set.

Table 7.5 illustrates how each component accounts for progressively less of the total variance. The eigenvalues can be used as a guide to the number of components to be retained for the next stage of the analysis. Guttman (1954) recommended the retention of only those components with eigenvalues greater than 1.0. Jöreskog et al. (1976) suggest that this is not always a satisfactory procedure. Where the objective is to create new, uncorrelated variables, they recommend that the principal component analysis should be conducted using as many factors as there are variables. If too few factors are chosen at the start, severe distortion of the data may occur, particularly where the factors are to be rotated. On this basis all six components were retained, despite the low eigenvalues of components 4, 5 and 6.

The third step in factor analysis involves the rotation of the components into terminal factors (Nie et al., 1975). The exact configuration of the factor structure is not unique; one factor solution may be transformed into another without violating the basic assumptions of the mathematical properties of a given solution. In other words, there are many statistically equivalent ways to define the underlying dimensions of the same set of data. This, in a way, is unfortunate because there is no unique and generally best solution.

Unrotated factors may or may not give a meaningful pattern of variables. The first factor tends to be a general factor which loads significantly on every variable. However, the remaining factors tend to be bipolar (i.e. approximately half of the variables have positive loadings and the other half negative loadings). These are usually hard to interpret. By rotating the solution, each variable is then accounted for by only one significant factor. This is conceptually simpler and easier to interpret. In this analysis a varimax rotation (Nie et al., 1975) was used, which centres on simplifying the columns of a factor matrix. The reason for this will become apparent in section 7.4.

Table 7.6 The component scores for each sample

Location	Component 1 Score	Component 2 Score	Component 3 Score	Component 4 Score	Component 5 Score	Component 6 Score
Borehole						
SH/36	-0.436251	0.249725	0.608595	-0.322050	0.158066	-0.379499
SH/37	-1.465035	0.415169	1.168197	1.344600	-0.368615	-0.777872
SH/38	0.064623	-0.163913	-0.675595	-0.825525	2.799562	0.507638
SH/40	1.300178	-0.695564	0.930139	-0.984999	-0.331723	-2.089777
SH/41	1.516249	0.535611	1.001063	2.195168	1.052708	0.397330
SH/43	-0.883454	-0.896099	0.453974	09.703183	0.053126	-0.987190
SH/44	-1.308079	0.848239	0.024838	-0.641590	-0.139504	0.761552
SH/46	1.282730	1.321928	0.908536	-1.505100	-1.140902	1.418897
SH/47	-0.330304	-2.641668	0.058726	0.195376	-0.661959	1.631485
SH/48	-1.069950	0.564662	0.072816	0.405035	0.037860	0.140520
SH/49	0.383373	-0.281618	0.246897	-0.271949	0.311284	0.410863
Sample						
1	0.164621	0.476873	-1.303893	0.740886	-1.269787	0.232608
2	-0.233446	0.860106	-1.869920	-0.335813	0.054457	-0.736630
3	1.013681	-0.593650	-1.623987	0.709501	-0.554394	-0.527292

Table 7.7 The results of the multiple regression analysis between hydraulic conductivity and components 3 and 5 of the factor analysis

Dependent Variable: Hydraulic conductivity

Independent Variables at 20% significance level: Component 3, Component 5

Degrees of freedom: 5

Variables in the regression set

Variable	Regression Coefficient	Standard Error	t stat.	Partial Correlation Coefficient
Component 3	-15.43936	10.69403	1.44	-0.57383
Component 5	2.954906	5.28201	0.56	0.22233
(Constant)	25.83635			
Multiple correlation coefficient	0.73813			
r^2	0.54483			

Table 7.8 The combination of the original grain-size groups within components 3 and 5

Original Constituent	Component 3	Component 5
Coarse gravel	1.31% $(-0.11445)^2$	1.61% $(-0.12689)^2$
Fine gravel	6.27% $(-0.25040)^2$	6.74% $(-0.25961)^2$
Coarse sand	5.08% $(-0.22539)^2$	3.95% $(0.19875)^2$
Medium sand	11.08% $(-0.33287)^2$	0.40% $(0.06325)^2$
Fine sand	1.72% $(0.13115)^2$	84.91% $(0.92147)^2$
Silt & Clay	74.53% $(0.86331)^2$	2.39% $(0.15460)^2$

Note: (i) Component loadings shown in brackets
(ii) Sum of squares (component loadings) = 1

7.4 Multiple Regression Analysis

Each component has a different factor score for each of the fourteen samples as shown in Table 7.6. These factor (or component) scores constitute the basic data for the multiple regression equation, which is of the form shown below;

$$K = TC_1 + UC_2 + VC_3 + WC_4 + XC_5 + YC_6 + Z \qquad (7.1)$$

where K = saturated hydraulic conductivity,

C_1 to C_6 = the component scores for components 1 to 6,
T to Y = regression coefficients,
Z = constant

Thus the six newly formed components form the independent variables in place of the six original grain size groups. Hydraulic conductivity data was only available for the eight boreholes used in the Spearman's correlation (section 7.2), thus the observation matrix for the multiple regression analysis consists of eight sets of data made up of the hydraulic conductivity and the six component scores for each of these boreholes. Computation of the multiple regression equation was achieved using a standard statistical computer package (Nie _et al._, 1975).

Details of the multiple regression analysis in Table 7.7 show that only components 3 and 5 were selected as being significant at the 20% significance level, the remainder being discarded. The final form of the regression equation was therefore

$$K = (-15.44C_3) + (2.95C_5) + 25.84 \qquad (7.2)$$

Component 3 is the most significant ($t = 1.44$, $n = 5$) and exhibits negative regression and partial correlation coefficients, and although component 5 is barely significant ($t = 0.56$, $n = 5$) it is important because it exhibits positive coefficients. The multiple correlation coefficient is 0.74, which is equivalent to a coefficient of determination of 0.55. Therefore, this regression equation accounts for 55% of the total variance.

Of greater interest is the combination of the original constituents within components 3 and 5 (Table 7.8). These results were obtained by squaring the component loadings shown in parentheses. Since their sum of squares is equal to 1.0, each individual constituent can be expressed as a percentage. The importance of rotating the components now becomes apparent. Each factor now loads significantly on only one variable.

An examination of the structure of component 3 shows that over 74%

Table 7.9 <u>Comparison between the actual and predicted saturated hydraulic conductivity at selected boreholes in the Stanton Harcourt area</u>

Borehole	Actual hydraulic conductivity, Km	Predicted hydraulic conductivity, Kp	Ratio Kp/Km (%)
SH/36	39.25	16.91	43.08
SH/38	44.91	44.54	99.18
SH/40	6.80	10.49	154.26
SH/41	7.26	13.49	185.81
SH/43	9.80	18.98	193.67
SH/44	15.00	25.04	166.93
SH/46	8.77	8.44	96.24
SH/48	30.93	24.82	80.25

Note: Hydraulic conductivity in m/d

consists of the fine grain sizes (silt plus clay). This probably accounts for the negative regression and partial correlation coefficients. The gravel and coarsest sand groups all have negative loadings, indicating that these groups are working in an opposite direction to the fine textured group. Although the signs are reversed, these results compare favourably with the Spearman's correlation coefficients. It is interesting to note that whereas medium sand had a r_s of zero, it is the second most important constituent within component 3 and the most important of the coarse textured groups. Nevertheless, the general pattern shown by Spearman's r_s is maintained, reflecting the dominance of the silt plus clay class in component 3.

The pattern of component 5 shows that fine sand is its chief constituent (nearly 85%). This is in contrast to the Spearman's correlation which indicated that it was only marginally significant. On the other hand it is noticeable that component 5 has a positive regression and partial correlation coefficient, which is in agreement with the r_s for fine sand. However, component 5 has a very much lower significance level and makes only a small contribution towards the total accounted variance in the multiple regression equation. Therefore less importance should be attached to these results.

7.5 Discussion of the results

An indication of the degree of success of the regression equation in the prediction of hydraulic conductivity can be made from Table 7.9. This shows the comparison of the measured (Km) and predicted(Kp) values. The results are also expressed in the form of a ratio, which standardises the predicted values so that their deviations can be compared. The closer the ratio to 100% the more accurate is the prediction.

The closest predictions are found for boreholes SH/38 and SH/46. These boreholes represent the two extremes of the range of hydraulic conductivity values used, showing that the prediction equation is equally useful at both high and low hydraulic conductivity values. The remaining sites give fairly successful results considering the small number of observations employed in the analysis. They compare particularly well with Bonell's (1971) results for fine-grained sediments. In his study the Kp/Km ratio ranged between 18.49% and 670.93%. Bonell noted however that the magnitude of these deviations were not too serious, considering the low values of hydraulic conductivity associated with glacial till.

Table 7.10 **The predicted saturated hydraulic conductivity at six additional sites in the Stanton Harcourt area**

Borehole	Predicted hydraulic conductivity, kp
SH/37	6.71
SH/47	22.97
SH/49	22.94
Sample	
1	42.21
2	54.87
3	49.27

Note: Hydraulic conductivity in m/d

The last step in this study of the association between grain-size and permeability is to calculate the hydraulic conductivity for the six sites in Table 7.3, using the prediction equation and component scores shown in Table 7.6. The results are shown in Table 7.10. Although all, apart from borehole SH/37, are at the upper limit of the observed hydraulic conductivity range, they are well within the range normally quoted for sand and gravels. The predicted hydraulic conductivity values for samples 1, 2 and 3 are particularly high in comparison with the boreholes. These samples were taken from individual lenses of open-work gravel exposed in the pit face, whereas the borehole samples were bulk samples representing the whole length of the borehole and, quite possibly, a variety of lithologies. It is interesting to note that the sample with the highest predicted value, sample 2, was taken from a frost-wedge in Dix Pit, Stanton Harcourt. Although this only represents one small sample, it does indicate that open-work gravels and frost-wedges are associated with zones of higher permeability. This would be a major factor in influencing the distribution and rate of groundwater sepage into dewatered gravel pits. These points are discussed further in chapter 11.

Based upon this analysis, particle size provides a satisfactory measure of pore size for the prediction of saturated hydraulic conductivity in gravels. In particular, the silt plus clay content is very important in controlling the rate of groundwater movement through unconsolidated gravel deposits. The greater the percentage fines content the lower is the hydraulic conductivity. This is the result of the decrease in pore size. For example, in those boreholes where the silt plus clay content equalled or exceeded 10% by weight, the hydraulic conductivity was less than 15 m/d. Conversely, sample 2, which had the highest predicted hydraulic conductivity, also had the lowest content of fines (1.91%). Before any precise conclusions can be drawn about the relationship between the silt plus clay content, the pore size distribution, and therefore permeability, a note of caution should be expressed. Kinsman (1957) pointed out that there are dangers in relying upon correlation coefficients based upon a small sample of sites and this can equally well be applied to a multiple regression analysis.

SECTION III GROUNDWATER-LEVEL FLUCTUATIONS AND RECHARGE IN THE TERRACE GRAVELS

CHAPTER 8

THE FACTORS AFFECTING THE FLUCTUATIONS OF WELL HYDROGRAPHS IN THE TERRACE GRAVELS

The purpose of section III is to investigate the characteristics of groundwater level fluctuations in the terrace gravels of the Stanton Harcourt and Ringwood areas, and to discuss the possible mechanisms of groundwater recharge which operate.

Before this, however, it is necessary to discuss the chief characteristics of the major hydrological and meteorological factors which influence groundwater fluctuations. This takes the form of a review of the concepts in chapter 8. Special attention, however, is paid to the influence of the soil-moisture balance, and a simple model is described which was used to estimate soil-moisture deficits and groundwater recharge. The final section of chapter 8 discusses the influence of gravel extraction and dewatering upon the water balance, and in particular the effect upon evaporation and transpiration rates. The role of groundwater in influencing evaporation and transpiration through capillarity is also examined.

Groundwater levels fluctuate in response to a variety of hydrological phenomena, which can be both natural or artificially induced. In many cases, there may be more than one mechanism operating simultaneously. In chapter 9, emphasis is placed on the influence of natural hydrological and meteorological factors such as precipitation, evaporation, and soil moisture using field data from the Ringwood and Stanton Harcourt study areas. The special influences imposed by gravel extraction and dewatering are discussed separately in chapter 10, because they form such an important part of this study.

Data on groundwater levels were collected from boreholes located within the two study areas (see chapter 4). There were two objectives in collecting this data. The first was to identify the major characteristics of the observed groundwater fluctuations within the gravels. The second objective was to investigate the effects of gravel extraction and dewatering on both groundwater levels and groundwater movement within the aquifers. These objectives are achieved in three ways. The first, which is linked to the study of groundwater fluctuations, is by the analysis of selected well hydrographs in relation to known hydrological and meteorological conditions. Particular emphasis is placed on an assessment of the recharge processes operating in the gravels. This approach is covered in chapter 9. The second - the use of groundwater contour maps - and third - the use of numerical models - are used to analyse the effects of gravel extraction and dewatering. These are discussed in chapter 10.

8.1 Introduction

The factors affecting the fluctuations of a water table in an unconfined aquifer can be classified into three groups. These are:

1. Fluctuations due to natural changes in groundwater storage.
2. Man - induced mechanisms.
3. The effects of external influences such as the tides and atmospheric pressure.

Natural changes in groundwater storage are the most important influence on both seasonal and short term groundwater-level fluctuations. These changes in storage, generally caused by fluctuations in the rate of groundwater recharge and discharge, give rise to gradual or seasonal changes in water levels. More abrupt fluctuations in storage and water-table level may occur where a stream, or a lake, is in hydraulic continuity with an unconfined aquifer. In the Hardwick study area, groundwater levels adjacent to the R. Windrush were found to be related to the height of the river. The inter-relationships between the groundwater of the terrace gravels and the R. Windrush are discussed in Section IV (see chapter 11).

Man-induced fluctuations of groundwater level can result from a number of activities; for example, groundwater abstraction, artificial recharge, irrigation, and drainage. Abstraction of groundwater by pumping, for whatever reason, produces rapid fluctuations in groundwater levels. Long-term groundwater abstraction can result in the gradual depletion of aquifer storage and downward trending groundwater hydrographs. The fluctuations of groundwater levels produced by gravel pit dewatering fall into this last category. The major discussion of their effects is reserved for chapter 10, although some comment is unavoidable in this and the succeeding chapter.

Fluctuations of groundwater levels arising from external influences are much less important. In coastal aquifers, in contact with the ocean, sinusoidal fluctuations of groundwater levels occur in response to tides (Todd, 1959). It has also been observed (Peck, 1960, Turk, 1975), that changes in atmospheric pressure can cause small fluctuations in the groundwater level in unconfined aquifers.

8.2 The influence of meteorological factors on groundwater fluctuations

The water-table represents the balance between groundwater recharge and groundwater discharge. Virtually all recharge to an unconfined aquifer is derived from precipitation directly over the outcrop. Similarly, two of the major components of groundwater discharge, particularly where the water-table is close to the surface, are evaporation and transpiration. Consequently, fluctuations of the water-table under natural conditions are strongly related to the balance between rainfall and evapotranspiration.

8.2.1 Rainfall, and the role of infiltration and percolation

The amount of rainfall which can be made available for groundwater recharge is controlled principally by the factors of infiltration and percolation. Infiltration is used in the context of the entry of rainfall into the soil, whereas percolation is used to describe the downward flow of water through the unsaturated zone towards the water-table.

The process of infiltration has been widely studied. Horton (1933) showed that rainfall, when it reaches the ground surface, infiltrates the surface layers of the soil at a rate that decreases with time. The decline is caused mainly by the filling of soil pores by water, although infiltration is also limited by soil surface, and surface cover, conditions.

Over the great part of the Stanton Harcourt and Ringwood areas, the capacity for infiltration would be expected to be relatively high. The conditions which favour this are low surface gradients and a permanent vegetation cover. In contrast, it would be expected to be lower in the vicinity of the gravel workings, where the vegetation and topsoil has been removed. In these areas, compaction by raindrops and the washing of fine particles into surface pores would tend to reduce infiltration. Possibly a greater factor, however, would be the compaction of the ground surface in the vicinity of pits by heavy vehicles.

The difference between rainfall and evapotranspiration is theoretically the amount of water available for percolation. However, only a percentage of infiltrated water actually percolates through the unsaturated zone to the water-table. The amount varies due to a number of factors, including initial moisture content, evapotranspiration and soil factors (Downing and Williams, 1969).

When rainwater enters the soil, a portion of it is stored in the soil pores, raising the soil moisture content. As the soil becomes wetter, its ability to transmit water increases until it is able to conduct water as fast as the rain is infiltrating the soil surface. Percolation rates are therefore greater when the initial soil moisture content is higher. In terms of variations in unsaturated hydraulic conductivity, the effects of moisture content must be considered in association with the pore size distribution of the soil. At high soil moisture contents, unsaturated hydraulic conductivity is directly proportional to soil texture. When infiltration into a dry soil takes place, the immediate surface layers will be saturated and there will be a decrease in moisture content with depth into the soil. Under these conditions both the suction gradient and the gravitational gradient, as well as adsorptive forces, encourage the downward percolation of water into the soil profile.

After the cessation of rainfall, gravitational drainage continues while evapotranspiration normally takes place. This leads to a redistribution of moisture within the soil, and a drying out of the surface layers. In this way, a suction gradient is created which encourages the upward movement of moisture and which reduces the downward percolation of water in the soil.

8.2.2 Evapotranspiration, and the factors which affect evapotranspiration

As a major subtraction of water from drainage basins, evapotranspiration dominates the water balance and controls such hydrological phenomena as soil moisture content, groundwater recharge, and stream flow.

Except for essentially physiological reactions, transpiration is controlled by the same factors that influence evaporation. It is at this point that the distinction between potential and actual evapotranspiration becomes important. Potential evapotranspiration is said to occur when a vegetated surface is losing water to the atmosphere at a rate unlimited by deficiences of water supply. When water becomes limited in its supply, actual evapotranspiration becomes important. Penman (1948, 1961, 1963), stressed that potential evapotranspiration is controlled essentially by meteorological factors, whereas actual evapotranspiration is also considerably affected by soil and plant conditions.

Penman's original definition of potential evapotranspiration as the evaporation from a short green crop completely shading the ground and with a nonlimiting supply of water, eliminates the effect of soil and vegetation conditions. Vegetation and soil factors become increasingly important as the water supply becomes limited. It is convenient, therefore, to discuss first the major factors which affect potential evapotranspiration, and then to discuss the additional factors which become important when water is a limiting factor.

All formulae for computing evapotranspiration are dominated by meteorological terms. In general, potential evapotranspiration tends to increase as the temperature, solar radiation, and windspeed increase, and as the humidity decreases (Vries and van Duin, 1953, Wijk and de Vries, 1954, Penman and Long, 1960, Bavel, 1966, Rijtema, 1968).

The influence of vegetation on potential evapotranspiration is more complex. One aspect which inevitably affects the transpiration rate is the albedo of the vegetation surface, since this determines the amount of solar radiation that is absorbed and available for evapotranspiration (Monteith, 1959).

It is generally accepted that evapotranspiration will only continue at the potential rate when the vegetation is not short of water (Penman, 1956). It has proved difficult to determine, however, at what stage of soil moisture depletion between field capacity and wilting point, evapotranspiration begins to fall below the potential rate. Soil texture affects the field capacity and permanent wilting point values of a soil and, with soil depth, these control the available water capacity. In a soil with only limited available water capacity, moisture is quickly depleted and evapotranspiration will soon fall below the potential rate. The variation of evapotranspiration when soil moisture becomes limiting is discussed further in section 8.3.

The relationship between soil moisture content and potential evapotranspiration provides an introduction to the additional factors which affect evapotranspiration below the potential rate.

As soil moisture is depleted, the depth and density of the roots of vegetation becomes of increasing importance (Penman, 1963). When the water content of the topsoil is depleted, shallow rooted plants transpire at rates less than the potential rate, whereas deeper rooted plants may continue to transpire at the potential rate for a longer time. The density of plant roots may become important if the rate of potential evapotranspiration is very high and is limited by the rate at which water can migrate through the soil to the roots. Under such conditions the distance of travel within the soil will be minimised by a dense root system.

Other plant factors may result in a marked departure of the actual evapotranspiration rate from the potential rate. Douglass (1967), relating stand density and evapotranspiration from forested areas, concluded that reducing the stand density results in reduced evapotranspiration, largely as a result of changes in root patterns and interception characteristics. Actual evapotranspiration losses are also reduced at the later stages of plant growth. After the ripening of grain crops, for instance, evapotranspiration falls considerably below the potential rate as the moisture demands of the crop are much reduced. Harvesting and ploughing will further considerably reduce the moisture demand (Rushton and Ward, 1979).

8.3 The soil moisture balance

In the previous sections, the hydrological and meteorological factors which affect groundwater-level fluctuations have been reviewed more or less independently. It is possible to draw these together using the concept of a soil-moisture balance. This term refers to the balance between the inflow of water to the soil from precipitation and the outflow of water from the soil by evapotranspiration and groundwater recharge.

Rainfall is not an accurate indicator of groundwater level changes. Recharge is the governing factor. The soil moisture balance allows a continuous record of soil moisture and groundwater recharge to be computed from meteorological records. Such a technique is very useful for relating individual groundwater recharge events to hydrological, meteorological and soil conditions.

That proportion of precipitation falling on the outcrop of an aquifer which does not evaporate is available for infiltration. Part of this infiltrating water replenishes soil moisture deficits, and is ultimately either transpired by vegetation or evaporated directly from the soil. Water percolating below the soil zone moves through the unsaturated zone to replenish groundwater storage. This water balance can be expressed by the following equation:

$$P = AET + \Delta\, SM + \Delta\, GWS \qquad (8.1)$$

where P = precipitation, AET = actual evapotranspiration, Δ SM = change in soil moisture, and ΔGWS = change in groundwater storage.

8.3.1 A soil moisture balance model for the Ringwood area - concepts

The conventional method of estimating recharge is based on the studies of Penman and Grindley (Penman, 1948, 1949, 1950; Grindley, 1967, 1969). Recharge is viewed as a function of effective rainfall (precipitation minus actual evapotranspiration), which is distributed according to a simple land-use model.

A computer program was written to calculate a simple soil moisture balance for the Ringwood area (the Ringwood Soil Moisture Model). Ringwood was chosen because daily rainfall and evaporation data were available, and because the results were used to relate fluctuations in groundwater level, observed at borehole R/8 using an autographic recorder, to changes in the soil moisture balance (see chapter 9). The model used is a very idealised representation of a complicated system, and relates to the situation in which the plant roots cannot tap groundwater. A flow chart showing the major steps in the calculations is shown in fig. 8.1. The program listing is shown in Appendix A3 (program 8.1).

The calculations were made on a daily basis. If longer time periods are used, short-term variations in soil moisture storage are often masked by the averaging effect of weekly or monthly input data. The basic data consisted of daily precipitation values, provided by Mrs. G. Haines from the Linwood site (see chapter 4), and daily values of grassland potential evaporation, produced by the Meteorological Office Rainfall and Evaporation Calculation System (MORECS) (Wales-Smith and Arnott, 1980).

MORECS was designed to produce objective estimates of evaporation (potential and actual), soil moisture deficiency (potential and actual) and effective rainfall, for areas of grassland and real - land usage. Daily values of potential evapotranspiration are calculated, using a modified Penman equation, for each of the 188 (40 x 40 km) grid squares into which Great Britain has been divided. The average values for each grid square are set against interpolated average values of measured rainfall, and a day-to-day running balance is obtained (Anon, 1978). When potential evapotranspiration exceeds current rainfall, soil moisture reserves are depleted and a soil moisture deficit accumulated. The calculations take account of the different water-holding capacities of various soil types, the different rooting depths of various crops and the differing rates of water extraction by various crops when soil moisture reserves have been partially exhausted.

When evapotranspiration exceeds rainfall, vegetation has to draw on reserves of water in the soil to satisfy the current evaporative demand (Grindley, 1967). A soil moisture deficit is then considered to occur. Provided that a reasonable amount of water exists in the soil, it is assumed that vegetation can transpire at the maximum rate, and actual evapotranspiration equals the potential rate,(Headworth, 1970). However, once the soil moisture deficit reaches a certain point, plants are unable to draw sufficient water from the soil and the transpiration rate falls below the potential rate.

One of the problems in assessing actual evapotranspiration is the extent to which water in the soil is available for transpiration between field capacity and wilting point. Some authors (eg. Kramer, 1952, Lassen et al., 1952, Thornthwaite, 1954) postulate a divergence between actual and potential evapotranspiration immediately a soil moisture deficit develops. At the other extreme, evapotranspiration is believed to continue at the potential rate until soil moisture is depleted to the wilting point. The problem has been discussed by Penman (1963). Rushton and Ward (1979) show four alternative drying curves for a vegetated soil which illustrates the way in which potential and actual evapotranspiration diverge. This diagram is reproduced in fig. 8.2. Curve 2 represents the equal availability concept put forward by Veihmeyer and Hendrickson (1927) and used in the Ringwood Soil Moisture Model described in this section. This concept assumes that the actual change in the soil moisture storage is equal to the potential change until a limiting value of the dry weather depletion (B in fig. 8.2) is reached, when evapotranspiration is assumed to cease. Holmes (1961) pointed out that the accuracy of these approximations depend on the soil type and vegetation characteristics. In a sandy soil,for example, the roots can withdraw water rapidly from the soil pores, and evapotranspiration may proceed at close to the potential rate until the wilting point is almost reached. Alternatively, in a clay-rich soil the water is held more tightly, and its movement to the roots is slow. The Veihmeyer model is therefore thought to be an adequate approximation for sandy soils, such as those found over the terrace deposits in the Ringwood area.

The limit to the dry weather depletion used in this study is 150 mm. This value was determined using principles detailed in the Meteorological Office's Soil Moisture Extraction Model (SMEM) (Wales-Smith and Arnott, 1980). The limiting dry weather depletion is regarded as the total available moisture within the assumed rooting depth of the dominant vegetation (in this case taken to be permanent grass). In the SMEM it is assumed that vegetation is able to extract freely, through its relatively dense system of shallow roots, 40% of the soil moisture (named MAX) held within its total rooting depth, and that up to 2.5 MAX may be extracted by deeper roots with increasing difficulty.

Representative values of MAX are given by Wales-Smith and Arnott (1980) for different vegetation types and soils of differing water availability. These are in good agreement with results obtained by Penman (1948, 1949, 1950) and Smith (1967, 1975). The available water capacity of a soil is defined as the amount of water held in the soil between field capacity and the permanent wilting point, and is the water available to plants. Its value depends upon the texture of the soil and the rooting depth of the vegetation. A representative value for the type of soils to be found in the Ringwood area (see chapter 2) is 100-200 mm per metre (Dunne and Leopold, 1978). From Wales-Smith and Arnott, the value of MAX for such soils under permanent grass is 60 mm, giving a limiting dry-weather depletion (2.5 MAX) of 150 mm.

In terms of its use by the SMEM, the concept of MAX is very similar to the root constant concept introduced by Penman (1949). This is defined as the soil moisture extractable at the potential evapotranspiration rate. In other words, by using 2.5 MAX as the maximum soil moisture deficit that can be accumulated without restricting evapotranspiration, the results will correspond to potential soil moisture deficits and will therefore tend to overestimate the actual soil moisture deficit in the Ringwood area, depending on whether one accepts the Veihmeyer or Penman concept of evapotranspiration. Consequently, the values of moisture surplus, which can be assumed to contribute greatly to groundwater recharge, may be underestimates. An increase in the value assigned to the root constant (or MAX) leads to a decrease in recharge, by delaying the date at which the summer moisture deficit is overcome.

8.3.2 Calculations of the soil moisture balance for the Ringwood area

The methods and principles used in calculating the daily soil moisture balance are best explained by examining the results shown in Appendix A4 (Tables 8.1 and 8.2). The computer program was run with data covering the period 1st January 1979 to 30th June 1980. January 1st was used as the starting date so that an initial soil moisture deficit of zero could safely be used. It was assumed that by this date, autumn and winter rainfall would have overcome the summer moisture depletion. It will be noted that the soil moisture deficit was reset to zero on the 1st January 1980. By assuming that evapotranspiration occurs at the potential rate until the total available water capacity is exhausted, this resulted in a large summer soil moisture deficit in 1979 which had still not been overcome by the end of the year. The soil moisture deficit on the 31st December 1979 was calculated to be 36.6 mm. Resetting the deficit to zero gives a more accurate representation of soil moisture changes during 1980, particularly during winter and spring when evapotranspiration tends to occur naturally at the potential rate.

Daily precipitation values are listed in column 1 of Tables 8.1 and 8.2. Column 2 contains values of potential evapotranspiration which were produced from MORECS. The difference between precipitation and evapotranspiration, termed effective rainfall, is shown in column 3. Negative values indicate an excess of evapotranspiration over rainfall. The difference, in such cases, is made up by withdrawals of soil water, hence the daily soil moisture deficit is equal to this amount. The accumulated SMD is shown in column 6. When the maximum soil moisture loss of 150 mm is reached, actual evapotranspiration continues at the potential rate only so long as precipitation is sufficient. If precipitation is less than the potential rate of evapotranspiration, actual evapotranspiration is equal to the precipitation. If there is no rainfall, actual evapotranspiration is zero. Actual evapotranspiration is shown in column 4. Alternatively, when rainfall exceeds evapotranspiration, moisture deficits are depleted. The change in soil moisture storage is shown in column 5. Thus, the 27.7 mm excess rainfall occuring on the 18th January 1979 will raise the accumulated soil moisture deficit, recorded at the end of 17th January, from - 0.5 mm to field capacity. The change in storage was therefore + 0.5 mm.

Table 8.3 Recharge rates (in mm) estimated for the terrace gravels of the Upper Thames Valley

	1971	1972	1973	1974	1975	1976	1977	mean
Jan.	99	49	9	9	65	2	52	41
Feb.	10	37	8	44	30	2	84	31
Mar.	18	28	0.8	13	50	2	21	19
Apr.	5	4	3	0	2	0	1	2
May	2	4	2	0.8	3	0	1	2
Jun.	12	2	10	5	0	1	3	5
Jul.	0.5	2	6	3	2	1	0	2
Aug.	6	1	2	8	0	4	5	4
Sep.	2	4	3	10	8	9	0	5
Oct.	10	3	3	5	1	11	1	5
Nov.	8	6	4	28	5	24	2	11
Dec.	13	10	4	22	3	87	6	21
Total	185.5	150	54.8	147.8	169	143	176	148

(source: Thames Water Authority)

Once the soil reaches field capacity, no more water can be stored in the soil and further excess-water is assumed to leave the soil by gravitational drainage. The amount of water that cannot be stored is termed the moisture surplus and is listed in column 7. Thus, on the 18th January 1979, 27.2 mm of rainfall cannot be stored in the soil, and it is this amount which is assumed to recharge the groundwater. The surplus is accumulated in column 8.

8.3.3 Conclusions from the model

The accumulated moisture surplus in column 8 of Table 8.1, indicates that the total recharge in the Ringwood area during 1979 is estimated to be 144 mm. In 1980, the total recharge for the six months up to the 30th June is estimated to be 121.7 mm (Table 8.2). No official recharge estimates were available for the Ringwood area to make a comparison, but recharge rates for the Thames valley gravels, upstream of Oxford, have been produced for the years 1971 to 1977 by the Thames Water Authority (Table 8.3). The average annual recharge was calculated to be 148 mm. This compares favourably with that estimated for the Ringwood area during 1979 (i.e. 144 mm). Given that the two areas are very similar hydrologically, and subject to fairly similar meteorological conditions, this suggests that the Ringwood Soil Moisture Model and, therefore, the initial assumptions are a fair representation of soil moisture conditions in the Ringwood area.

The results produced by the Ringwood Soil Moisture Model (and confirmed by the TWA data for Oxfordshire) indicate that by far the greatest proportion of recharge during any one year is concentrated in the winter and spring months. In 1979, all groundwater recharge in the Ringwood area is estimated to take place between January and May. In the Stanton Harcourt area, using the TWA data in Table 8.3, approximately 84% of the total annual recharge occurs between November and March. This is an oversimplification, since the Ringwood Soil Moisture Model implies that when a soil moisture deficit exists, all rainfall in excess of evapotranspiration is used to make good the soil moisture depletion. That means to say that there is no direct percolation to groundwater until the soil moisture deficit is zero. It will become apparent in chapter 9, however, that this assumption is invalid, since groundwater recharge (reflected by increases in groundwater levels) has been observed during the summer months when large soil moisture deficits exist. The main point is that individual storms can reduce the soil moisture deficit to zero, especially in winter, and cause recharge to occur even when there is an overall moisture deficit for that month.

The results of the calculations using the Ringwood Soil Moisture Model indicate the general trends in the soil moisture balance to be expected in southern England over the year. During winter and early spring, rainfall exceeds evapotranspiration and a moisture surplus accumulates. Soil moisture deficits do occur, but these are generally small and short-term, lasting only a few days. Between May and September, the trend is reversed and evapotranspiration exceeds rainfall. As a result, groundwater recharge virtually ceases and a gradual soil moisture depletion takes place. At the end of this period, the limiting dry weather depletion is reached, and transpiration virtually ceases. From October onwards, as rainfall increases, the soil moisture deficit is gradually reduced.

The changes in soil moisture conditions described above will be reflected in the saturated zone by similar changes in groundwater storage. In chapter 9, changes in groundwater storage, as represented by recorded fluctuations in water-table level, will be related to elements of the soil moisture balance as calculated above.

8.4 The effects of gravel extraction and dewatering on the water balance

8.4.1 The effects of flooded pits on evaporation and transpiration rates

It is convenient at this point to introduce the possible effects of gravel extraction and dewatering on the water balance, and particularly on evapotranspiration. The factors which will be of most importance in affecting rates of evaporation and transpiration are:

a) changes in the nature of the evaporating surface, and
b) changes in the height of the water-table.

It is important to stress that the continuing formation of artificial lakes, as a result of gravel workings, constitutes an important and increasing loss from the water budget in areas such as Stanton Harcourt and Ringwood, where gravel extraction is a predominant feature of the environment. In fig. 8.3, monthly actual evapotranspiration and open-water evaporation estimates for the Stanton Harcourt area are plotted over the period 1970 to 1976. Open-water evaporation data were not available for the Ringwood area, so a comparison could not be made. The evapotranspiration data was provided by the Meteorological Office and is based upon data from Oxford, but this is sufficiently close to Stanton Harcourt to enable the comparison to be made. The open-water evaporation data was provided by the Thames Water Authority from daily evaporation-tank measurements recorded at Farmoor Reservoir, near Stanton Harcourt (see fig. 2.1).

The ratio of open-water evaporation to potential evapotranspiration varies between 1 : 0.82 and 1 : 0.46 in the Stanton Harcourt area. Open-water evaporation, therefore, exceeds evapotranspiration by 18% to 54% per year. These values compare favourably with Neumann (1953), who found that the water use of a short crop (including grass) was approximately 75% of the open-water evaporation. Penman and Schofield (1951) listed three reasons why this is generally so:

1) the higher albedo of the vegetation
2) closure of the plant stomata at night
3) diffusion impedance of the stomata

Both curves in fig. 8.3 indicate that the highest rates of water loss are experienced in June and July, with the lowest rates in December. The overall difference between total rates of evaporation from an open-water surface and evaporation from a vegetated surface is due to the influence of the vegetation and the soil.

Whilst showing that, over the year, open-water evaporation exceeds evapotranspiration, it is apparent that this relationship is not always true when examined on a monthly basis. During the spring and early summer months, for instance, evapotranspiration is equal to and, in some cases, greater than open-water evaporation. This corresponds to that period during which rainfall is highest, and the soil is capable of supplying moisture to the vegetation to maintain evapotranspiration at the potential rate. From mid-summer onwards, however, evapotranspiration falls below the open-water evaporation rate. It is at this time that the soil moisture deficit is increasing most rapidly and vegetation finds it increasingly difficult to extract moisture from the soil. In the late autumn and winter months, the difference between evapotranspiration and open-water evaporation reaches a maximum, as the former falls almost to zero. This is due to the fact that in winter the growth, and therefore the water demand, of vegetation is at a minimum.

8.4.2 The effects of water balance changes on groundwater levels

Increased evaporation from flooded gravel pits may give rise to lower than average groundwater levels. Since the majority of flooded excavations in the study areas are in close connection with the surrounding groundwater body, the increased evaporation which results from the formation of open-water bodies represents an increase in groundwater discharge from the terrace gravels. Stoneacres Lake near Stanton Harcourt covers approximately 15 hectares. Using a mean annual evaporation rate of 735.3 mm, this represents a total water loss from the lake of 110,295 m^3 per year. The long-term (1915-1975) annual rainfall measured at Oxford is 650.5 mm and represents a total addition to the lake of 97,575 m^3 per year, excluding any direct runoff which may enter the lake from the surrounding land. The deficit of 12,720 m^3 will be made up by a decrease in groundwater storage. However, given the extent and the hydraulic characteristics of the gravels surrounding the lake, it is unlikely that the increased evaporation from any single lake of this size will have a measurable effect on groundwater levels. On the other hand, the development of a number of large lakes in close proximity would have a cumulative effect, and may eventually lead to lower groundwater levels in these areas.

8.4.3 The effects of the removal of vegetation by gravel extraction on the water balance

A significant change in local rates of evaporation and transpiration may result from stripping the land of vegetation during site preparations for gravel extraction. Over most of the study areas, transpiration is the dominant factor in the total water loss from the land surface. The presence of a vegetation cover effectively reduces evaporation from the soil by shading the surface from the effects of the sun, by reducing wind speed, and by increasing the relative humidity of the lower layers of the air. This was substantiated by Penman (1963) and Ritchie (1972), who estimated for various vegetation types that transpiration is at least twice as great as evaporation from bare soil and that in many cases the ratio is in excess of 3:1. Only during the colder months, when the vegetation cover is reduced, would conditions favour higher evaporation losses from the soil. It is at this time, however, that climatic conditions favour only low rates of evaporation. It is concluded, therefore, that removal of the vegetation cover during the early phases of gravel extraction, followed later by the creation of large lakes, may have a significant effect on local rates of evaporation and transpiration.

8.4.4 Summary

A common objection to gravel workings, particularly those which are flooded, is that they could lead to changes in local climatic conditions, i.e increased rainfall or atmospheric humidity as a result of increased evaporation from open-water areas. However, climatic conditions are largely determined by the nature of air masses, and disturbances within them, which cross the British Isles. It is most unlikely that these will be significantly altered by local disturbances. Local microclimatic conditions, on the other hand, may be affected by gravel workings. However, the effects of this will be virtually unrecognisable. Of much greater significance is the effect on the local soil moisture balance, and the way in which this may affect agriculture.

8.5 The influence of groundwater on the water balance

Evaporation from a soil surface is governed by the same meteorological factors that govern evaporation from open-water, since soil evaporation is merely the evaporation of the films of water surrounding the soil grains and filling the soil interstices (Ward, 1975). The supply of moisture, in the case of free-water evaporation, is always so plentiful that it exerts no limiting influence on the rate of water loss. On the other hand, evaporation from soils is generally less than open-water evaporation, not because the climatic conditions are different, but because there is an insufficient supply of water in the soil to be evaporated (Viehmeyer and Brooks, 1954). Theoretically, if the soil is kept constantly wet, evaporation from a bare surface may even exceed free-water evaporation (Chang, 1965). Thus, the most important factors affecting evaporation from bare soils are those which influence the availability of moisture within the surface layers of the soil.

It is the moisture content of the first few centimetres of surface soil which influences evaporation the most. Once the surface begins to dry, evaporation rates will drop sharply (Fortier, 1907, Harris and Robinson, 1916). Upwards movement of moisture through the soil, therefore,

Table 8.4 Monthly soil moisture deficit (in mm) estimated for the Upper Thames Valley

	1970	1971	1972	1973	1974	1975	1976	1977	mean
Jan.	0	0	0	9	6	0	24	0	5
Feb.	0	0	0	9	1	0	21	0	4
Mar.	12	0	0	13	7	0	28	2	8
Apr.	12	0	0	7	41	6	61	17	18
May	39	33	31	35	76	28	98	38	47
Jun.	90	22	57	63	101	92	130	50	75
Jul.	99	80	90	62	111	115	138	94	99
Aug.	89	64	112	95	107	133	131	84	102
Sep.	90	81	111	86	45	104	69	66	82
Oct.	100	47	113	72	14	75	24	51	62
Nov.	14	21	57	62	3	34	4	13	26
Dec.	6	5	13	29	0	31	0	1	11
Total	551	353	584	542	512	618	728	416	539

(Source: Meteorological Office " Fortnightly Bulletin of Estimated Soil Moisture Deficit ") assumes importance. Numerous experiments (Parshall, 1930, Tanner, 1957, Veihmeyer, 1938, Viehmeyer and Brooks, 1954) have shown that soil evaporation is at a maximum when the water-table is at, or near to, the surface. Evaporation decreases rapidly as the water table falls, until a critical depth is reached when any further fall produces only a slight change in the evaporation rate. This critical depth depends largely on the capillary characteristics of the soil, and varies with soil texture and particle size.

The effect of evaporation is to create a suction gradient within the soil which rapidly becomes greater than the opposing gravitational gradient. This encourages movement of water towards the soil surface. Generally, the height of capillary rise is greatest in fine-textured soils, and very seldom exceeds 1.0 metre in height.

The rise of water from the water table by capillarity can also affect the transpiration rate. The water requirements of plants can rarely be met by the available water capacity of the soil. In the Ringwood area, the results of the Soil Moisture Model have shown that the available soil water is insufficient to compensate the climatic moisture deficit (i.e. the difference between total evapotranspiration and total rainfall). As a result, crops will suffer moisture stress unless water can be supplied to the root zone from another source, such as from below the water-table. Where soils are of fine texture and the water-table is within about 1 metre of the soil surface, capillarity could be expected to supply water to the root zone. Land and Water Management Ltd. of Cambridge conducted some simple experiments to determine the height of capillary rise in gravels of the Stanton Harcourt area. These experiments are described below.

8.5.1 An experiment to determine the capillary rise of groundwater in gravels

Table 8.4 shows the monthly soil moisture deficit for the Upper Thames valley area as compiled by the Meteorological Office for 1970 to 1977. A soil moisture deficit begins to develop around April and increases throughout the spring and summer until a maximum is reached in August. The average soil moisture deficit in August is 102 mm. From September onwards, the soil moisture deficit is less relevant, since rainfall exceeds evapotranspiration, and the demand for water from vegetation is falling. Using 110 mm as the mean available water capacity, and accepting that crops will be under moisture stress when 50% of the available water capacity has been used (i.e. 55 mm), an additional 47 mm of water will be required in an average year to maintain optimum growth. For the three summer months of June, July, and August, when the deficit would be expected to be highest, this would be needed at an average rate of 16 mm per month.

The question is, how much of the water required can be supplied from groundwater? As part of a study carried out in the Stanton Harcourt area by Land and Water Management Ltd. (1978), they conducted experiments to determine:

a) the height to which water rises from the water table by capillarity

b) the rate at which the rise occurs, and the amount of water transmitted.

Samples of gravel from two soil pits at Northmoor were collected and placed in cylindrical columns, in which a 'water-table' was maintained throughout the experiment at a height 2 cm above the base of the gravel. The surface of the columns were kept in a dry condition by means of an air blower, so as to create a 'demand' for water uptake. Sufficient water was added daily to maintain the water-table at a constant height. Figs. 8.4 and 8.5 show the rate of water uptake for two columns (A and B) during the period of the experiment. Fig. 8.4 shows that a relatively large amount of water must be initially added to maintain a constant water level, but after 2 or 3 days the rate of uptake diminishes until after approximately eight days a constant rate of uptake is established. The initial rapid uptake is due to capillary action, whereas the subsequent slower rate of uptake results from variations in soil moisture suction and surface tension which occur once the initial demand is fulfilled.

Examining the period of the experiment when the rate of uptake was comparatively constant, the equivalent monthly rate of moisture uptake was calculated to be 12.3 mm per month. Most of this is not due to capillarity, but to suction and/or tension movements.

The results of experiments by Wind (1961) indicated that the maximum value of capillary flow is dependent upon height above the water-table rather than on suction imposed at the soil surface. Fig. 8.5 indicates that once the height of the gravel column is increased from 30 to 36 cm, the rate of water uptake diminishes rapidly. So long as the column remained at a height of 30 cm, water was transmitted fairly rapidly by capillarity. Approximately 45 mm of water per month could be transmitted upwards under these conditions.

Table 8.5 **Soil moisture content of two gravel samples from the Northmoor area, Stanton Harcourt, following experiments to determine the height of capillary rise**

Depth from surface (cm)	Moisture content (% of dry weight)	
	Column A	Column B
0- 3	0.5	1.4
5- 8	1.2	7.7
10-13	7.4	12.8
15-18	8.6	13.4
21-24	10.2	14.1
27-30	11.6	13.3
33-36	13.5	17.4

(Source: Land and Water Management (1978))

8.5.2 Conclusions from the experiment

Two main conclusions emerge from these results. Firstly, significant amounts of water can be transmitted upwards from a water-table in the Northmoor gravels by capillarity, so long as the area of demand (either the root zone or an evaporating surface) is within 30 to 36 cm of the water-table. Secondly, if the water-table falls to 36 cm or more below the area of demand, considerably less water is available. These conclusions imply that the capillary fringe in the Stanton Harcourt gravels is about 30 to 36 cm. This was shown in another way. On completion of the experiment, the soil moisture content of the samples was estimated at various depths down the columns. The results are shown in Table 8.5. The trend of progressive dryness towards the surface is the same for both columns and for 7 to 10 cm below the surface, the soil is very dry. The capillary fringe is therefore taken to be 25 to 30 cm in height above the water-table. These figures compare favourably with those reported by Keen (1927), who found that capillary lift contributed insignificantly to total evaporation losses after the water table had receded to about 35 cm below the surface in coarse sand.

Up to 45 mm of water per month can be transmitted upwards through gravels from below the water-table if the demand is there. Therefore, so long as the water-table is within approximately 30 cm of the root zone it is likely that plants can obtain the 16 mm of water per month to maintain active growth from groundwater.

During the dewatering of gravel workings, drawdown of the surrounding water-table will occur. The areas that may be adversely affected by a lowered water-table are those which currently benefit from a high groundwater level; but, given the degree of drawdown and the capillarity of the gravel, would cease to do so. Some areas will not suffer, either because at present they draw no benefit from the groundwater, or because the degree of drawdown will not be great enough to make any significant difference.

It follows (from the preceding paragraphs) that in the Stanton Harcourt area at least, those areas able to benefit from groundwater are those where the water-table is either within the soil profile or less than 30 to 40 cm below the soil in the gravels. It is generally evident that where dewatering lowers the water-table to a depth greater than 40 cm below the top of the gravels, capillary rise will not be sufficient to take moisture into the soil layer above, nor will plant roots be able to bridge the dry gravel barrier. It is likely therefore that in areas which previously benefited from a high water-table, the effect of a drawdown in the water-table will be to cause many agricultural crops to come under moisture stress, particularly in summer, and for evaporation and transpiration to be reduced.

In chapter 10 this discussion is expanded in the light of empirical data. The areas around Northmoor where the existing water-table can benefit crop growth are delimited and then, by modelling the drawdown around proposed gravel workings, the extent to which these areas will be affected by the predicted degree of lowering of the water-table is estimated.

CHAPTER 9

THE RESPONSE OF GROUNDWATER LEVELS TO HYDROLOGICAL AND METEOROLOGICAL FACTORS

9.1 Introduction

In chapter 8, the processes involved in recharge and groundwater level fluctuations were considered conceptually. This was done looking from the surface down, through a discussion of infiltration, evapotranspiration and the soil moisture balance. In this chapter, the processes are considered from the groundwater-end using field observations.

Fluctuations in groundwater level are related to hydrological and meteorological conditions within the study areas, using individual hydrographs from selected boreholes. The fluctuations in groundwater level are classed as being either seasonal or instantaneous. Section 9.2 deals with the seasonal fluctuations in groundwater level, using selected hydrographs from boreholes in both the Stanton Harcourt and Ringwood areas. These fluctuations are explained in terms of seasonal variations in rainfall and evaporation. Section 9.3 concentrates on the short-term or instantaneous response of the water-table to individual rainfall events, using continuous groundwater level records from borehole R/8, situated in Ibsley Airfield, Ringwood. By using continuous recordings, individual and generally short-lived fluctuations in groundwater level, which would probably be missed by monthly or even weekly measurements, can be observed. This discussion will pay particular attention to the relationship between the instantaneous fluctuations and the soil water balance which was calculated for the Ringwood area (see chapter 8).

Groundwater recharge replenishes the total volume of water held in an aquifer and it is this quantity of water which in the long term may be available for dewatering from gravel excavations. It is therefore of great importance in any assessment of groundwater problems.

According to the classical theory of Penman and Grindley (Penman, 1948, 1949, 1950; Grindley, 1967, 1969) recharge fails to occur when the presence of a soil moisture deficit exists. Consequently, recharge is generally thought to be restricted to the winter and spring months (October to April). Several aspects of chapter 8 reveal deficiencies in this theory. A reappraisal of the recharge process suggests a more realistic model. Kitching et al. (1977), for example, report that recharge was recorded for almost every month during experiments with two large lysimeters on the Bunter Sandstone of North Nottinghamshire. Observations from the Ringwood area similarly indicate that groundwater recharge may take place even when a soil moisture deficit exists and is not limited to the winter months. Alternative mechanisms of recharge which operate in the gravel aquifers during winter and summer are proposed in section 9.4.

9.2 Seasonal fluctuations in groundwater level

For the purpose of analysing and discussing seasonal fluctuations in groundwater level, hydrographs were constructed for a representative sample of boreholes from each study area (SH/9, SH/18, SH/27, R/2, R/4, R/8, R/32 and R/33). These hydrographs are shown in figs. 9.1 to 9.8.

Precipitation data proves very useful in explaining seasonal groundwater-level fluctuations in the terrace gravels of the Stanton Harcourt and Ringwood areas. The monthly rainfall totals, together with the accumulated departure from the mean, for the periods covering the hydrographs are plotted in figs. 9.9 and 9.10 for the Stanton Harcourt and Ringwood areas respectively.

Below-average rainfall during the drought of 1975 and 1976 is reflected in fig. 9.10 by the increasing cumulative deficit of monthly rainfall below the mean. This can be compared with the very low groundwater levels observed in the Ringwood area over the same period, particularly in the winter of 1976. The 1975 and 1976 winter groundwater levels are between 0.5 metre and 1.0 metre below average winter groundwater levels observed in the proceeding years. No groundwater data was available for this period from the Stanton Harcourt area.

The mean monthly rainfall for the Ringwood area is calculated for the period January 1975 to June 1980, which incorporates the two very dry years of 1975 and 1976. During the period 1977 to 1980, therefore, monthly rainfall values were generally higher than the average. This is reflected by the rapid reduction in the accumulated mean monthly rainfall deficit during the winter of 1977. Similarly there was a rapid increase in groundwater levels over the Ringwood area. In all observation boreholes the water-table recovered to its pre-drought level. The seasonal variations in rainfall (high in winter and low in summer) during 1978 and 1979 are reflected by corresponding increases and decreases in the accumulated departure from the mean, and by corresponding fluctuations in groundwater level.

In general terms, the groundwater hydrographs from both the Stanton Harcourt and Ringwood areas indicate that below average monthly rainfall during the summer correlates with falling groundwater levels, whereas above average monthly rainfall in the winter produces sustained groundwater recharge and increasing groundwater levels.

9.2.1 The components of seasonal groundwater fluctuations

The groundwater hydrographs in figs. 9.1 to 9.8 can be divided into two seasonal components. The first, which generally occurs between December and April, is the recharge or rising-limb. This is characterised by rapid increases in groundwater level. The second, which generally occurs between May and November, is the falling-limb or recession curve. This is characterised by a steady fall in groundwater levels. During this period, individual rainfall events have very little, or only short-lived, effect on the overall recession curve.

The recharge limb of the hydrographs is generally very steep and maximum groundwater levels are reached very soon after the onset of winter recharge. For example, the hydrograph for observation well R/4 (fig. 9.5) shows that between September and November 1976 the groundwater level increased sharply by approximately 1.45 metre, i.e. an increase of 0.725 metre per month.

Maximum groundwater levels are generally of short duration. The hydrograph peak is quickly followed by the onset of groundwater recession, which may continue for anything up to nine months. Compared with the recharge limb, the slope of the recession limb is appreciably less steep. For example, the hydrograph for borehole R/4 (fig. 9.5) shows that in 1977, groundwater recession lasted from February to August, representing a fall in groundwater level of approximately 0.90 metre, i.e. a decrease of 0.13 metre per month.

Theoretically, under natural conditions, groundwater recession is, like river stage, governed by the recession constant (Headworth, 1972). It can recede at a lesser rate, however, if infiltration is superimposed on the natural recession. If the rate of infiltration exceeds the rate of recession then groundwater levels will rise. The usual method of evaluating the recession constant is to plot the rate of recession, either logarithmically or linearly, against time. Tangents to the receding portion of the graphs are constructed and from these the recession constant can be determined.

Using continuous recordings of groundwater level from borehole R/8 (fig. 9.6), the receding portions of the hydrographs were examined in detail. The rate of recession i.e. the fall in groundwater level (measured in metres) per day, was calculated for all ranges of water level and plotted against the water level on linear graph paper (figs. 9.11 and 9.12). These graphs show a wide scatter of points due to most values having a recession rate lessened by varying amounts of infiltration. However, an envelope curve drawn through the highest rates of recession, defines the relationship between the groundwater level and the maximum rate of groundwater recession.

The recession graphs consist of a series of straight lines. According to Headworth (1972), the breaks in slope probably coincide with lithological variations in the material through which the borehole is sunk, i.e. steeper rates of recession indicating less permeable horizons and flatter rates of recession indicating more permeable horizons. A major discontinuity, visible on both graphs, seems to occur at a depth of about 3.0 metres. Since the recession constant appears to increase with depth, this suggests that the permeability of the gravels in the vicinity of borehole R/8 also increases with depth.

9.2.2 Variations in the timing of seasonal groundwater fluctuations and the effects of dewatering

The onset of groundwater recovery following the summer recession generally occurs during the period September to December. Any variations tend to occur between years rather than between boreholes. In any particular year, the recovery tends to occur at the same time throughout each study area. This event generally corresponds to the onset of above average rainfall, reflected by the reduction in the cumulative departure from the mean (see figs. 9.9.b and 9.10.b), and the reduction in soil moisture deficit.

Conversely, the timing both of the peak groundwater level and of the onset of groundwater recession is quite variable between boreholes. In the Stanton Harcourt area, for example, maximum groundwater levels occur over a five month period, which is generally between February and June. In the Ringwood area the timing is slightly less variable, the peaks tend to occur between February and April.

The exact time of the maximum groundwater level can be related in some instances, to two factors. These are:

1) depth to the water-table
2) distance from a dewatered pit

In the groundwater year October 1978 to September 1979, the maximum groundwater level in observation well SH/27 occurred during June 1979. At observation well SH/18, it occurred four months earlier (i.e. in February 1979). At the time of their respective peaks the depth to the water-table was 2.7 metres and 1.15 metres respectively.

In fig. 9.13, the month in which the maximum groundwater level was reached during groundwater year 1978-1979 has been plotted for each borehole in the Northmoor area. There appears to be an inverse relationship between the distance of the boreholes from Brown Pit (stage C) and the timing of the groundwater maximum, i.e. the groundwater peak occurred increasingly later in those boreholes nearest the dewatered pits. The maximum groundwater level at those boreholes closest to the pit, for example SH/1, was delayed four months relative to boreholes only 200 metres further away from the pits. The reason for this is not clear, since recovery began at the same time throughout the area (i.e. in November 1978). One possible reason is that because of the steep hydraulic gradients created by dewatering, a great proportion of the total recharge of the aquifer eventually flows towards the pit. This would have the effect of prolonging the recovery period and increasing the recharge at the boreholes closest to the pit. The average range between the minimum and maximum groundwater levels in 1978-1979 at those boreholes which had a June 1979 maximum was 1.02 metres. For those boreholes where the peak occurred in February or April 1979, the ranges were only 0.78 metres and 0.90 metres respectively.

9.2.3 Variations in the range of seasonal groundwater fluctuations and the effects of dewatering

In both study areas, the total fluctuations in groundwater level are relatively large, considering the total depth of the gravels. In most cases, the total fluctuation is between 0.75 and 1.0 metres per year and during normal years there is only a small annual variation in minimum and maximum water levels. Recovery of the water-table to approximately the same level occurs irrespective of the minimum level of the previous autumn. This indicates that winter and spring recharge is always sufficient to replenish groundwater storage depleted during the summer. The constancy of winter groundwater levels suggests that groundwater discharge points, such as rivers or lakes in hydraulic contact with the gravels, are a controlling factor.

In some boreholes close to dewatered pits, however, groundwater levels vary from year to year depending on their actual proximity to the point of dewatering. The hydrograph for observation well R/4, for example, shows that there was a decrease in overall groundwater levels between 1977 and 1980, i.e. the autumn minimum and winter maximum occur at increasingly lower heights. The years 1975 and 1976 are excluded because of the extraordinary drought conditions experienced during those years. Two possible explanations are put forward for the decrease and evidence exists to support both proposals. Firstly, there was a decrease in annual precipitation between 1977 and 1980. Secondly, there was an increase in groundwater abstractions due to the dewatering of gravel pits on Ibsley Airfield.

There was a significant decrease in annual precipitation from 989.4 mm in 1977 to 716.5 mm in 1979 (fig. 9.10). This could mean less rainfall available for recharge and so lower groundwater levels.

Dewatering commenced on Ibsley Airfield in 1978. Prior to September 1979, the dewatering was spasmodic and ineffective in controlling the water levels in cells 5, 6 and 7. It was not until work commenced on cell 8 in September 1979 that continuous dewatering took place. Fig. 9.14 shows the daily rate of discharge from cell 8 between September 1979 and April 1980. The discharge was measured using a V-notch weir (described in chapter 4). Discharge increased steadily between September and December 1979, until in January 1980 there was a sharp increase from 1000 to 2500 m^3/day. It remained around this higher level until the end of the recording period. The increase in the rate of abstraction corresponds to lower winter maximum groundwater levels at borehole R/4, i.e. the maximum level in 1980 is 0.24 metres lower than in 1979 and 0.40 metres lower than in 1978. Perhaps of more significance is that the point of dewatering has moved progressively closer to borehole R/4 over the same period. This will have the effect of increasing the water-table drawdown at that point (see chapter 10).

The reduction in groundwater level observed at borehole R/4 is not so apparent at any of the other boreholes on Ibsley Airfield. Although there has been a slight decrease in the height of the maximum groundwater levels in some boreholes (R/32 for example), there has been very little change in minimum groundwater levels. This suggests that the reduction in winter groundwater levels is more a result of the decrease in rainfall. For example, the total winter rainfalls at Linwood (measured between October and March) for the period 1977-1980 are,

	winter (mm)	summer (mm)
1977	780.2	381.4
1978	545.0	295.5
1979	448.5	305.2
1980	402.4	n/a

Summer rainfall totals, on the other hand, do not show a similar decrease over the same period. This is reflected in the overall constancy of minimum groundwater levels in most boreholes. In the case of borehole R/4, therefore, the decrease in minimum groundwater levels would seem to be the result of the additional effect of dewatering.

In the reverse situation to that at borehole R/4, movement of the point of dewatering away from borehole SH/1 resulted in an overall increase in groundwater level between 1977 and 1980 of almost 1.4 metres (fig. 9.15). In October 1977, dewatering of Brown Pit (stage A), which is directly adjacent to the borehole (see fig. 4.1), ceased and dewatering of stage B commenced. The overall result was a steep rise in the groundwater level at SH/1 of over 1.5 metres between December 1977 and May 1978. A better idea of the overall effect is shown by comparing the minimum levels of 1977 and 1978. The difference is +1.0 metres. A similar trend was observed in several of the other boreholes which are situated close to Brown Pit (stage A).

In fig. 9.16, the difference between the 1977 and 1978 minimum groundwater levels has been plotted for each borehole in the Northmoor area. There are distinct areas of positive and negative movement which coincide with areas affected and unaffected by dewatering respectively. The greatest recovery (represented by positive values) occurs in those boreholes nearest to Brown Pit (stage A) and decreases with increasing distance from the previously dewatered pit. A line has been drawn to show the limit of the area which experienced an overall recovery of groundwater levels following the completion of the dewatering of stage A. Outside this line of zero change there is an overall decrease in groundwater level (represented by negative values). This is more consistent with the observed decrease in rainfall (see fig. 9.9). The line of zero change corresponds approximately, therefore, to the limit of the zone of depression produced by the dewatering of Brown Pit (stage A).

Dewatering of Brown Pit (stage B) ceased in October 1978, being replaced by stage C. This was followed by a further overall increase in groundwater level at borehole SH/1 of 0.4 metres between November 1978 and September 1979. Similar increases were recorded at other boreholes close to stage B, ranging from 0.38 metres at SH/4 to 0.14 metres at SH/10. The groundwater level at borehole SH/1 continued to rise through the winter of 1980, reaching a recorded maximum level of 63.17 metres O.D. in January 1980. At this point in time, the water-table in some places was very close to the surface. For example, at borehole SH/1 the depth to the water-table was only 0.53 metres. Consequently most of the ditches in this area were at 'bankfull' level. At Greystones Cottage (fig. 4.1), the adjacent ditch was observed to be carrying water for the first time since the start of the study in October 1977.

9.3 Short-term fluctuations in groundwater level

Examination of charts taken from the autographic recorder placed on borehole R/8 indicated that moderate and heavy rainfalls cause relatively large water-level responses. These responses can be divided into two types, having the following general characteristics:

Type 1 - small amplitude, steep rising limb, short peak and exponential recession

Type 2 - large amplitude, less steep rising limb, often multi-peaked and a slow recession.

The type 1 responses are generally short-duration events, the majority lasting no more than 24 hours (the time being measured from the point of initial rise in groundwater level to the point at which the water-table falls to its pre-response level). On the other hand, the type 2 responses are generally longer in duration and are, in some cases, composed of more than one smaller response (i.e. multi-peaked). The most important distinguishing characteristic, however, is that the type 1 responses occur when there is a soil moisture deficit, whereas the type 2 responses occur when there is a moisture surplus. This suggests that different recharge mechanism operate under different conditions. The movement of water to the water-table as a sheet, or by displacement, is suggested in section 9.4.1 to produce the type 2 response. The type 1 response is more difficult to explain. Various mechanisms are discussed in section 9.4.2, including flow through preferred routeways and lateral flow.

Analysis of the hydrographs from borehole R/8 is separated into two sections. Section 9.3.1 deals with the winter and spring period when a soil moisture surplus, or only a small deficit, exists within the root zone and type 2 responses are most common. Section 9.3.2 deals with the summer period when large soil moisture deficits exist and type 1 responses are predominant. Various examples of the two types of responses are examined diagrammatically. The basic format consists of the daily amount of rainfall at the head of each diagram, with the corresponding groundwater hydrograph below. Effective rainfall is used because this will be more representative of field conditions and will remove negligible quantities of rainfall. In addition, the daily soil moisture deficit is shown at the bottom of each diagram. The effective rainfall and soil moisture data are taken from table 8.1 (see Appendix A4).

9.3.1 Winter and spring responses

During the winter and spring period, the majority of groundwater responses occur when there is a moisture surplus and are typically of type 2 character. On some occasions, however, when there is a moisture deficit, heavy winter rainfall may produce a type 1 response.

A typical type 1 response is depicted in fig. 9.17, which illustrates the effect of an isolated rainstorm event. A single storm took place on 5.3.79 when 13.4 mm rainfall fell (an effective rainfall of 11.0 mm). This followed a relatively dry period when only 10.8 mm rainfall was recorded in 20 days. The soil moisture condition at the end of 4.3.79 showed a deficit of 10.5 mm. The heavy rainfall on 5.3.79 overcame this deficit and produced a small moisture surplus of 0.5 mm. The hydrograph of R/8 begins to rise early on 6.3.79, at the end of which a soil moisture deficit of 1.3 mm was calculated. Although relatively slow at first, the hydrograph shows an almost vertical rise during the last 12 hours of 6.3.79, reaching a peak at 2400 hours. It is significant that the peak response took place at least 24 hours after the end of precipitation. This was followed by an exponential decline. The total rise was only 2.2 cms.

An example of a type 2 response is shown in fig. 9.18. Heavy rainfall occurred between 23.3.79 and 26.3.79. On 22.3.79, a small soil moisture deficit of 6.3 mm existed, but the heavy rainstorms in the following four days transformed this to a surplus of 6.8, 1.1, 2.0, and 1.5 mm on each of these days. The significant features of this response are that the gradient of rise is not so steep as that found in fig. 9.17. In addition, the peak is more prolonged, so that the hydrograph is plateau-shaped. Also, the decline in groundwater level is much slower than that shown in fig. 9.17. Ten days after the peak, the water-table had still not returned to its pre-storm level. This contrasts with the very short period it takes for the groundwater level to rise the same distance during storm recharge. As a comparison, it took approximately 36 hours for the water-table to reach its pre-storm level in fig. 9.17.

Further examples of type 2 response are shown in figs. 9.19 to 9.21. These all occured during winter or early spring when there was only a small soil moisture deficit and heavy rainfall produced a water surplus on consecutive days. All are similar in shape, i.e. they have a slow rise, a prolonged peak and a very slow recession.

After a short lapse of several hours following the last rainstorm on 11.4.79 (fig. 9.19), the hydrograph develops into a uniform recession resembling a gently graded straight line. This marks the beginning of the summer groundwater recession and it corresponds to the start of a constant (increasing) soil moisture deficit. Prior to this date the soil moisture conditions are changing rapidly, because of the more frequent rainstorms.

Figs. 9.20 and 9.21 illustrate two large responses, both having a total range of approximately 0.2 metres, which occurred during the winter of 1980. The first response followed twelve consecutive days of heavy rainfall between 28.1.82 and 8.2.82. This eliminated a small soil moisture deficit of 3.7 mm on 28.1.82 and produced a moisture surplus on the following nine days. Although there are relatively minor responses on 31.1.80 and 2.2.80, the main response does not begin until 3.2.80. This is six days after the onset of the moisture surplus. The gradient of rise is relatively gentle and, in addition, the peak is not attained until 9.2.80, at least three days after the termination of effective rainfall. There is a recurrence of the uniform recession gradient during the post-storm period.

The response shown in fig. 9.21 is similar in many ways. A prolonged wet period between 21.3.80 and 31.3.80 produced a major response which commenced on 23.3.80. A feature of this response is that the recharge-limb is very irregular, contrasting with the generally smooth rise shown in earlier examples. The irregularities are a response to the alternating soil conditions, of moisture surplus and moisture deficit, which occur between 18.3.80 and 31.3.80. The hydrograph peak is not reached until 3.4.80, at least three days after the termination of rainfall. This peak, at 21.70 metres O.D., also represents the maximum winter groundwater level observed at borehole R/8. The groundwater level begins to fall about two days later and this marks the onset of the summer recession. As in fig. 9.19, this marks the development of a constant soil moisture deficit.

9.3.2 Summer responses

Groundwater responses observed during the late-spring and summer period exhibit certain contrasting features from those observed during the winter. Two cases are examined which occurred between the end of April and the beginning of June 1979.

The first example followed an intense rainstorm of 16.4 mm which occurred on 30.4.79 (fig. 9.22). On 29.4.79, a fairly large soil moisture deficit of 28 mm existed within the root zone. The response is a typical type 1, having the characteristic steep rise and short peak, followed by the exponential recession.

The significant feature of fig. 9.22 is that the response coincides with a major precipitation event, despite the occurrence of a large soil moisture deficit. This can also be seen in fig. 9.23. The three main responses represent a significant departure from the general summer recession curve. This was due to an unusually wet May, in which a total of 101.6 mm of rainfall was recorded in the Ringwood area (compared with the 1975-1980 mean of 53.5 mm). Much of this (over 80 mm) was recorded during the second half of the month, changing a soil moisture deficit of 41.9 mm on 18.5.79 to a moisture surplus on 27.5.79. This period is most interesting, however, because of the occurrence of both type 1 and type 2 responses.

Two type 1 responses occurred, on 22.5.79 and 26.5.79, which are basically similar to that in fig. 9.22. Both occurred during the period when there was still a significant soil moisture deficit. They differ, however, in the form of their recession curves. Although the initial fall is exponential, this is quickly replaced by a more gradual recession curve which is very similar to the normal summer depletion curve. It appears that these responses occur when intenses rainstorms, which produce the initial steep rise, are followed by a number of days of less intense rainfall which modify the recession.

The type 2 response is shown in fig. 9.23. It occurredonly two days after a typical type 1 response, and corresponds to the change from a soil moisture deficit to a soil moisture surplus on 27.5.79. This point is important since it clearly illustrates the dependence of these two very different types of response on moisture conditions within the soil.

One further interesting point is that the 31.3 mm of rainfall which fell between 19.5.79 and 22.5.79, when there was a soil moisture deficit, produced a total rise in groundwater level of only 2.2 cms on 22.5.79. On the other hand, following only 28.9 mm of rainfall between 27.5.79 and 29.5.79, the rise in groundwater level between 29.5.79 and 1.6.79 was approximately 0.8 metre. This was during a period when a moisture surplus existed.

9.4 The mechanisms of groundwater recharge in terrace gravels

The major characteristics of groundwater fluctuations within terrace gravels, exemplified by borehole R/8, have been outlined in the preceeding sections. It now remains to assess the possible recharge mechanisms operating in the gravels.

The groundwater hydrographs show rapid responses, irrespective of groundwater level, to moderate and heavy.rainfalls throughout the year. The distinction made between type 1 and type 2 responses suggests that different recharge mechanisms operate during the winter and summer periods.

It is generally regarded that shallow groundwater recharge is the result primarily of vertical percolation through the larger air-filled interstices to the water-table, once the material in the unsaturated zone is at field capacity (Miller and Turk, 1949, Baver, 1956).

Smith (1967) concluded that infiltration in sands is a simple hydraulic process subject only to modification by capillarity. The basis of his theory was that when a given infiltration sheet travels through an already moist sand it merely picks up the unsaturated water ahead of it and leaves an equal quantity behind (providing the sand was initially at field capacity). The entire amount of infiltration then reaches the water-table as recharge. If the sand is initially below field capacity, proportionally less of the infiltrating water would reach the water-table.

The Hortonian view proposed by Smith may well apply to the type 2 responses which are typical of the winter months when the soil moisture deficit is low, but an alternative mechanism of recharge must be considered to explain the type 1 responses which occur during the summer. The purpose of this section, then, will be to assess the possible factors contributing to groundwater level responses to rainfall, dealing first with winter conditions (type 2 responses) and then with summer conditions (type 1 responses).

9.4.1 Groundwater recharge mechanisms during the winter months

If the orthodox theory of recharge is considered, it would be anticipated that the response interval (the time lag between the onset of effective rainfall and the initial response of a groundwater hydrograph) would be a function of the depth to the water-table. On this basis it could be envisaged that the time lag would be small in the case of boreholes in gravels of high hydraulic conductivity where the water-table was a relatively short depth below the surface. However, an analysis of the continuously recorded hydrograph from borehole R/8 does not support this theory.

During the winter and early spring period there is a relatively long time lag between the commencement of rainfall and the initial response, despite the soil moisture deficit being very low or non-existent. In figs. 9.17 to 9.21, all the responses occur at least 24 hours after the commencement of rainfall. The maximum response interval measured was approximately 70 hours in January 1980.

Using 24 individual type 2 responses from borehole R/8, a correlation analysis shows that there is only a poor correlation ($r = 0.442$, $n = 24$) between the depth to the water-table and the response interval. It should be noted, however, that only daily rainfall totals were available. No information was available on the rainfall intensity or, more importantly, on the time at which the rainfall commenced. It was assumed for the statistical analysis that the rainfall on any one day commenced at 00.00 hours. In extreme cases, therefore, the calculated response interval could be in error by 24 hours. The accuracy could have been improved significantly by installing a recording rain gauge. Unfortunately, one was not available in the School of Geography at the time this study was undertaken. There appears, therefore, to be very little evidence to support the hypothesis that recharge is primarily due to direct percolation of rainwater to the water-table. Some other recharge mechanism must be involved which is not a function of the response interval. The view of recharge as an inevitable and complete consequence of surface infiltration has over the years been questioned and tested, and new theories proposed.

One long standing hypothesis is that water-table fluctuations are the result of the displacement of existing soil water by newly infiltrating rainfall. Horton and Hawkins (1965) were the first to describe this process, which was later termed translatory flow by Hewlett and Hibbert (1967). Their concept is based upon the fact that small interstices or capillary pores will contain water to a much greater height above the water-table than will large interstices after drainage has occurred. The process of displacement is said to be the result of a strong horizontal potential matrix suction gradient which causes percolating water to be deflected away from the larger pores in which vertical gravitational percolation initially takes place. The horizontal flow is eventually transmitted to the smallest interstices where the tension on the water held within them is suddenly released. This results in a downward displacement of the water previously held in the soil.

The speed at which water is transmitted through the unsaturated zone depends to a great extent upon the hydraulic conductivity of the soil. When the soil is saturated, all the pores are filled with water and conducting, hence the hydraulic conductivity is at a maximum. As the soil begins to dry out, it becomes increasingly unsaturated, so that some of the pores become partially, or completely, air-filled. The first pores to dry out are the largest ones, so greatly reducing the overall conductivity of the soil. Hillel (1971) stated that the transition from the saturated to unsaturated state can entail a decrease in hydraulic conductivity of up to 1/100,000th of its value at saturation.

Under field conditions, the soil is generally unsaturated to varying degrees. Since water tends to be drawn into pores which are already water-filled, percolation will become concentrated in the smallest pores while the largest, and most conductive, pores remain empty. In this case the numerous lenses of coarse gravel which are generally found in the terrace gravels may act as a barrier to unsaturated flow and increase the tortuosity of the flow paths.

It is proposed that the displacement theory of recharge can be adopted to explain the type 2 responses which occur exclusively during the winter period. The implication that each input of rainfall does not necessarily have to traverse the entire unsaturated zone before a response is registered in a well may account for the poor correlation between the response interval and the depth to the water-table. Similarly, except following the most intense rainfall, percolation will generally be slow, since it is restricted to the smallest pores. The displacement hypothesis may therefore explain why the type 2 responses are characterised by a prolonged peak and slow recession.

9.4.2 Groundwater recharge mechanisms during the summer months

According to the classical theory, recharge only occurs when a soil moisture surplus exists. From experimental work on both unconsolidated and consolidated porous media, Brooks and Corey (1966) concluded that a threshold level of saturation must be reached before any water transmission takes place. This would involve making up any soil moisture deficit. Consequently, recharge is usually throught to be restricted to the winter months of September to March.

The hydrographs in figs. 9.22 and 9.23 show that recharge does occur during the summer months, despite the presence of a soil moisture deficit. Kitching et al. (1977) report that recharge to the Bunter Sandstone of North Nottinghamshire was recorded for almost every month using lysimeters. An alternative recharge mechanism must therefore be postulated to that which occurs in the winter.

Any method suggested must be capable of producing a rapid response, even when there is an overall soil moisture deficit, with a response interval that is not linked to the depth of the water-table. Two possible mechanisms are suggested; rapid percolation through preferred routeways, and lateral flow to boreholes.

Surface water reaches the water-table via a number of different routes. Intergranular seepage augments the moisture content of the soil and satisfies any moisture deficit before recharge may occur. Water may also enter and flow through crack systems in the unsaturated zone, thus reaching the water-table with little or no effect on general soil moisture conditions.

Rapid percolation is well documented for highly fissured rocks such as chalk and limestone. The 'direct input' recharge mechanism would be expected to occur when rainfall is particularly intense and the surface layers of the soil become saturated. Drainage would then become concentrated in those channels having the greatest hydraulic conductivity. Smith et al. (1970) estimated from tritium profiles taken from a chalk aquifer that approximately 15% of the effective rainfall is transported through fissures to the water-table, whereas the remaining 85% reaches the water-table by intergranular flow. Foster (1975) concluded that fissure flow in the chalk may be even more dominant than Smith et al. suggested. Downing and Williams (1969) also demonstrated that direct recharge may occur in the highly fissured Lincolnshire Limestone.

It is easy to comprehend that direct recharge occurs in fissured rocks, but more difficult to apply to unconsolidated gravels, particularly when there is a well developed soil profile. However, some preferred routeways may exist, particularly in summer when the soil is dry and the moisture content low. These may be desiccation cracks, old root systems, animal burrows, or even internal features of the gravels themselves, such as frost-wedges. It is interesting to note that in chapter 7, the highest hydraulic conductivity value was predicted for a sample of gravel taken from a frost wedge in Dix Pit, Stanton Harcourt. Subsurface 'pipes' and springs, in which subsurface flow is concentrated, have also been observed within the gravels (chapter 11). Although it is not known how widespread these are, it is clear that on a local basis at least, they serve to concentrate subsurface flow which would produce conditions favouring rapid recharge.

The mechanism of 'direct input' could produce rapid recharge, but the effect would be limited to the duration of the rain-storm. It could, therefore, explain the short peak and rapid recession of the type 1 responses. However, the relatively long response intervals of some type 1 responses (e.g. fig. 9.23) is not consistent with this theory. Also, this hypothesis would not account for the type 1 responses which occur in the winter when the soil is at field capacity (e.g. fig. 9.17).

The second possible mechanism is lateral seepage from a perched water-table, which could occur in a variety of ways. The first, which was proposed by Bonell (1972) in boulder clay, involves lateral seepage from a shallow perched water-table. Its position coinciding with a reduction in the structural development of the soil, and therefore in the rate of vertical percolation, as a result of the termination of roots and ploughed conditions. Whilst being only a transitory phase during the process of percolation, this part of the process could account for the rapid water-table response soon after the commencement of rainfall, particularly since the boreholes are not sealed above the water-table. This mechanism could operate both in summer and winter.

More general seepage could occur from water temporarily 'perched' above naturally occurring discontinuities in the unsaturated zone (i.e. interbedded lenses of fine sand and silt), during the later stages and the immediate period after a rainstorm. Alternatively, if during vertical percolation the water traverses bands of high permeability gravel (i.e. lenses of 'openwork' gravel), there may be a strong horizontal component to the flow which could result in lateral seepage. The length of time which a perched water-table remained would depend upon the duration of effective rainfall.

The question is whether the perched water body remains static until it is dissipated by vertical seepage, or whether there is any significant lateral movement which is intercepted by the boreholes. The overall topographical gradient in the area of borehole R/3 appears far too gentle to induce rapid lateral movement of subsurface water. If there were any substantial lateral movement it is probably confined to the steeper ground at the edges of the terraces. The more widespread situation is the probable occurrence of a 'static' perched saturated zone being intercepted by the wells. The exponential decline of the type 1 responses could be a function of a static perched saturated zone, which is being slowly dissipated by vertical percolation towards the saturated zone.

Under certain conditions the infiltration rate may be greater than the vertical hydraulic conductivity, so that a temporary perched water-table is formed.

Type 1 responses which occur in winter generally follow short, intense rainstorms. The one on 6.3.79 (fig. 9.17), for example, followed 13.4 mm of rainfall which fell on the previous day. Hansen (1955) and Whipkey (1965) noted that the wetting front may act as an impediment to vertical water movement due to short-term rainfall intensities exceeding the general subsurface percolation rate. Thus a saturated zone may temporarily develop above the wetting front creating lateral seepage. Whipkey cited entrapped air as the major cause of this factor.

The above discussion suggests that a complex process is involved in summer recharge, with the possibility that some groundwater movement takes place within a perched water-table zone. This is a temporary saturated zone associated with short, heavy rainstorms. It would seem likely that during the winter period the permanent zone of saturation would extend into the upper layers of the strata and coalesce with any perched saturated zone, so that type 1 responses are rare during this time of the year.

Field evidence exists for the occurrence of perched saturated zones. In November 1979, seepage was observed approximately 2 metres above the base of the gravels in Brown Pit (stage C). The seepage originated from a zone lying above a layer of grey silty material only 10-20 cm thick. Seepage was also occurring at the base of the gravels, presumably from the permanently saturated zone.

9.5 Summary

Fluctuations in the water-table occur in response to groundwater recharge. Two types of groundwater fluctuation have been recognised, which are termed seasonal and instantaneous. The seasonal fluctuations can be explained in terms of seasonal climatic variations, particularly rainfall and evapotranspiration. Such fluctuations generally occur at the same time, and the range between minimum and maximum groundwater levels is the same, each year. This has been shown however to be modified by dewatering. The peak groundwater levels may be delayed several months by dewatering and, in the long term, groundwater levels decline as a result of dewatering, although they recover rapidly when dewatering ceases or the position of the centre of abstraction changes.

Instantaneous fluctuations of the water-table occur in response to individual periods of rainfall. Two types have been identified, based on shape and size, which are also dependent on soil moisture conditions. Type 1 responses generally occur in summer and autumn, following an intense rainstorm, and when there is an overall soil moisture deficit. This field evidence confirms the conceptual evidence which was outlined in chapter 8, namely that recharge can occur despite a soil moisture deficit.

Type 2 responses, which tend to produce a greater increase in water level, commonly occur in winter and spring, when there is a soil moisture surplus.

Alternative recharge mechanisms have been put forward to explain the two types of instantaneous response. Type 1 responses, which are characterised by a steep rising-limb, are said to result from the rapid percolation of infiltration through preferred routeways and/or the interception of a perched static water-table by the boreholes. Conversely, type 2 responses, which are characterised by a prolonged peak and slow recession, are said to result from the downward displacement of water previously held in the soil.

CHAPTER 10

THE EFFECTS OF GRAVEL PIT DEWATERING ON THE SPATIAL DISTRIBUTION AND FLUCTUATIONS OF GROUNDWATER

Introduction

The chief characteristics of fluctuations in groundwater level have been assessed in the preceeding chapter by an examination of the hydrographs of selected wells. In this chapter, the spatial pattern of groundwater is investigated, with particular reference to the effects of gravel pit dewatering. The aims of this chapter are twofold. In the first section, the spatial variations in groundwater movement around dewatered pits in the two study areas are examined by the use of groundwater contour maps. In the second section, a numerical model of groundwater flow around a dewatered pit is developed.

10.1 The spatial distribution of groundwater around dewatered pits in the two study areas

Todd (1959) and Davis and De Wiest (1966) discussed the basic principles involved in the construction and interpretation of groundwater contour maps and these were applied in this work. A number of contour maps for each study area have been chosen to illustrate selected hydrological conditions. Particular attention has been paid to the periods of maximum and minimum antecedent groundwater conditions (i.e. late winter and late autumn), but the contour maps have also been chosen to reflect the variations brought about by developments in the dewatering of the two areas.

It should be noted that in the construction of these maps, use has not been made of the whole borehole network in each study area. The emphasis of this chapter is on the distribution of groundwater around gravel pits, so that some boreholes which are now known to lie outside the radius of influence of the main areas of dewatering have been omitted. With hindsight, it might have been better to have limited the area of the borehole networks. More boreholes should have been sited closer to the pits, where the influence of dewatering is greatest.

The existing borehole networks provide a broad view of the distribution of groundwater over the study areas (i.e. the macro-study areas). There is a very important area, a few metres wide, directly adjacent to the dewatered pits (the micro-study area) which has not been monitored at all adequately. For this reason, the groundwater contours immediately adjacent to the pits had to be largely interpolated. Because of the scale of the maps and the steep hydraulic gradients it has not been possible to draw all the contours adjacent to the pit, but they do give a reasonable indication of the extent to which the distribution of groundwater has been disturbed by gravel excavation. Also, for clarity, the groundwater levels at some boreholes, particularly in the Stanton Harcourt area, have been omitted from the maps.

10.1.1 Stanton Harcourt

Five selected groundwater contour maps are shown in figs. 10.1 to 10.5 for the area around Linch Hill and Northmoor. These cover the period November 1977 to February 1980. This particular part of the study area was chosen because of the greater concentration of boreholes, and the opportunity that it gives to study the effects of the dewatering of the Brown Pit complex. This consists of a series of four excavations (or stages) to the south of Stoneacres Lake. Each stage was dewatered and, after the exhaustion of the gravel, was allowed to flood naturally with water. As each of the four excavations was dewatered in turn, there were consequent changes in the position of the drawdown zone. These are illustrated in the five maps. The extent of the drawdown zone is shown on the maps by the position of the groundwater divide, which represents the line of zero drawdown.

The minimum water table level for 1977 occurred between late October and early December. The groundwater contours for 12 November 1977 are shown in fig. 10.1. Over the area as a whole, the groundwater contours reflect the surface topography. The regional flow of groundwater is in an easterly direction, towards the R. Thames. In general, the rate of flow appears to be appreciable. The apparent lack of any discontinuity in the water-table around the gravel-alluvium boundary indicates that the gravels form a continuous spread beneath the alluvium. Assuming that the gravels are in hydraulic continuity with the river, the direction of groundwater flow suggests that, in this stretch at least, the R. Thames is being recharged. Further downstream, in the Wytham-Binsey area, the river has been shown to be losing water to the gravels (Anon, 1977).

The general easterly flow of groundwater in the Northmoor area is maintained throughout the year with very little appreciable change (figs. 10.2 to 10.5).

On a small scale the water-table is influenced by the subsurface topography of the Oxford Clay-gravel boundary. For example, there is an increase in hydraulic gradient in the vicinity of borehole SH/8 (see fig. 4.1 for borehole locations). An examination of the gravel isopachs in fig. 3.2 indicates that there is a channel-shaped depression in the surface of the Oxford Clay in the vicinity of borehole SH/8.

To the north and east of Linch Hill the water-table exhibited a fairly uniform slope, but around the points of dewatering, a groundwater depression formed. This created a groundwater divide down-gradient of the pits and caused a diversion of groundwater flow (as shown by the arrows in figs. 10.1 to 10.5).

The most important feature of figs. 10.1 to 10.5 is the zone of depression centred on the gravel pits at Linch Hill. This zone of depression was produced by the dewatering of:

a) Brown Pit (stage A), which ceased in October 1977
b) Brown Pit (stage B), between October 1977 and September 1978, and
c) Brown Pit (stage C), which started in September 1978 and continued until the end of the monitoring period in July 1980.

The gravel in Brown Pit (stage A) was exhausted in September 1977 and, after site restoration, the dewatering pump was switched off on 14 October 1977. For a short time during September and October both stages A and B of Brown Pit were being dewatered. The drawdown zone around stage A was still very pronounced on 13 November 1977 (because of the short time since dewatering ceased), but it had merged with the zone of depression produced by the dewatering of stage B. By 2 February 1978, stage A (now called Willow Pool) had not yet completely refilled with water. Although the zone of depression on that date was almost entirely due to the dewatering of stage B (fig. 10.2), there was still a considerable flow of groundwater towards Willow Pool which produced a distinct zone of drawdown around the lake. Most of the flow into the lake appeared to be taking place along the northern and western banks, whereas along the eastern and southern edges of the lake, there was a steep hydraulic gradient towards the new pit.

In figs. 10.1 and 10.2, the groundwater contours east of the Linch Hill gravel pits are a fairly accurate picture of the natural conditions. The lack of borehole information to the west of these pits, means that the contours in this part of the maps have been drawn with substantially less confidence. There is no direct evidence to suggest that the zone of depression extended so far west as Manor Farm in November 1977 (fig. 10.1), although more accurate maps drawn after the additional boreholes were added in January 1979 (figs. 10.4 and 10.5) do show that this area was affected by later stages of dewatering.

In mid-winter, that is when the highest well levels occur, the general direction of groundwater movements remained unchanged (fig. 10.2). However, the zone of depression was not a static feature. The contour maps show that the total drawdown and the area of influence varied during the year. Between November 1977 and February 1978, for example, there is evidence to suggest that there was a reduction in the extent of the zone of depression, particularly in the western area of the maps. This was due to the effects of winter recharge, which also led to an average increase in groundwater level of 0.25 metres in the gravels to the east of the groundwater divide. There was, however, no increase in hydraulic gradient. Within the drawdown zone itself there was also a substantial increase in groundwater levels (over 0.5 metres in some places). Some of this must have been due to the effects of increased groundwater recharge in winter, but around stage A the recovery was also related to the gradual filling of Willow Pool.

By April 1978, Willow Pool had completely flooded. This event was accompanied by a significant change in the pattern of groundwater flow around Linch Hill. The dewatering of Brown Pit (stage B) ceased at the end of September 1978, whereupon the excavation and dewatering of Brown Pit (stage C) commenced. Fig. 10.3 shows the groundwater contour map for 10 October 1978, at which time the drawdown produced by stage B should have been near the maximum. With the final flooding of Willow Pool, the extent of the zone of drawdown was greatly reduced, as shown by comparing the position of the groundwater divide in figs. 10.1, 10.2 and 10.3. The most significant changes in the groundwater pattern occurred immediately to the east and north of Willow Pool, between Greystones and West End House. Prior to April 1978, there was a pronounced radial flow of groundwater towards Willow Pool, particularly from the area east and north-east of the present lake. After April 1978, there was a permanent reversal in the groundwater flow pattern over many of these areas. To the east of Willow Pool there was a change from a south-east to north-west to a north-west to south-east direction of flow. This was most prominent between boreholes SH/19, and SH/20; SH/5, SH/6 and SH/7; and SH/11 and SH/14 (see fig. 4.1). The change to the north-east of the lake was even more marked. The steep hydraulic gradient between boreholes SH/5, SH/6, and SH/7, and boreholes SH/9 and SH/10 (figs 10.1 and 10.2) had completely vanished after April 1978 (fig. 10.3).

The final flooding of Willow Pool in the spring of 1978 led to the recovery of groundwater levels in almost all the private wells in the Northmoor area. Prior to the flooding of Willow Pool, the extent of the zone of drawdown was such that many private wells in the Northmoor area were substantially dewatered. The most affected were wells SH/19 and SH/20, and to a lesser extent SH/22, SH/23, SH/24 and SH/25 (see fig. 4.1). After the change in the position of the dewatering and the flooding of Willow Pool, the position of the groundwater divide moved eastwards, so that by October 1978 only SH/19 was within the drawdown zone. Although the recovery at the other wells was slow (the groundwater level at SH/20 on 10 October 1978 was only 0.07 metre above that on 12 November 1977), by February 1979 the groundwater level at SH/20 was 0.46 metre higher than in February 1978 (this compares with a maximum difference of around 0.2 metre in areas unaffected by the dewatering).

By the 8 February 1979 the groundwater pattern had changed further (fig. 10.4). Dewatering was restricted to Brown Pit (stage C) and stage B (now called Teal Reach) was in the process of flooding. The addition of further boreholes to the network in January 1979 enables a more accurate picture of the groundwater pattern to the south and west of the Brown Pit complex to be drawn. The contour map shows that the drawdown zone was concentrated in the area around stages B and C and that the hydraulic gradient was steepest to the west of stage C. There is also now direct evidence to suggest that the area around Manor Farm had been dewatered by all stages of Brown Pit. To the east of the pits, the drawdown zone was greatly reduced, and the groundwater divide had receded further. As a result, well SH/19 was then on the limit of the zone of depression (i.e. zero drawdown). There was still, however, a significant hydraulic gradient around Teal Reach, because the pit was not yet completely flooded.

Between October 1978 and February 1979, the recovery of groundwater levels to the east of the Brown Pit complex, previously affected by the dewatering of stages A and B, produced a ridge of groundwater extending eastwards between Greystones and Watkins Farm (fig. 10.4). This is in marked constrast to figs. 10.1 and 10.3, in particular, which show that

Table 10.1 Fluorimeter readings of water samples taken from Stonesearch Lake, Northmoor, following input of rhodamine dye in borehole SH/45

Time from dye input (hours)	Fluorimeter meter readings (in meter units)	
0	0.049	background reading prior to dye input
0.5	0.049	
1.0	0.049	
1.5	0.049	
2.0	1.200	first indication of dye entering lake
4.0	0.510	
6.0	0.450	
8.0	0.450	
10.0	0.160	
12.0	0.148	
14.0	0.124	
16.0	0.100	
18.0	0.081	
20.0	0.074	
22.0	0.067	
24.0	0.059	
26.0	0.050	
28.0	0.048	
30.0	0.048	
32.0	0.045	
34.0	0.055	
36.0	0.060	
28.0	0.046	concentration returns to normal levels

there was previously a groundwater trough in this area. The trough was caused by the retreat of the groundwater contours towards the centres of pumping during the dewatering of stages A and B.

The ridge of groundwater became a persistent feature of the area and, as shown in the contour map for 12 February 1980 (fig. 10.5), increased in extent through the development of the ridge west of Greystones. By February 1980, Teal Reach had completely flooded and an extension to Brown Pit (stage C) was being excavated, so that both parts were being dewatered. The recovery of the groundwater levels around Teal Reach was marked by a further shift westwards of the groundwater divide and a reversal of the hydraulic gradient south of the lake. All private wells, including R/19 (see fig. 4.1), were well outside the drawdown zone. The groundwater level at R/19 was the highest recorded at this well throughout the study.

The shape of the groundwater contours is complicated by the shape of the pits and by the presence of the lakes. However, the exact relationship between the lakes and the groundwater is not certain. The hydraulic connection between the lakes and the aquifer must be limited to some extent by a lining of earth, or in some cases Oxford Clay, which is used to contour the banks (plate 10.1). It is not known exactly to what extent this limits the movement of water between the aquifer and the lake, but there is evidence to shown that it does not form a complete seal.

The various excavations at Browns Pit took approximately 6 months to become completely flooded (this is taken as the point at which lake levels became stabilised and at which the surrounding drawdown zone had disappeared). This cannot have been entirely the result of net precipitation into the excavation. Some inflow of groundwater must occur through the low permeability barrier and from dye tracing experiments it appears to be quite rapid. Rhodamine dye was injected into borehole SH/45 (see fig. 4.1) and subsequently identified by fluorometer analysis in samples of water taken from Stoneacres Lake, approximately 20 metres away, only two hours later (table 10.1). Borehole SH/45 is itself situated within a layer of overburden, up to 30 metres in width in some places, which was used to contour the sides of the original pit. The steep hydraulic gradient observed to the west of Stoneacres Lake (figs. 10.4 and 10.5) marks the position of this layer of low permeability material. A dye dilution experiment of the kind described in chapter 6 was carried out in borehole SH/45 and provided a permeability value of 1.3 m/d for this material.

Table 10.2 Groundwater temperatures along transect adjacent to Willow Pool, Northmoor (locations shown on Fig. 4.1)

Date of Monitoring	Willow Pool Temperature (oC)	Groundwater temperature (°C) (along east - west transect)				Average groundwater temperature (oC) in Stanton Harcourt area
		SH/1	SH/2	SH/3	SH/4	
12. 9.78	17.5	14.7	12.8	12.4	12.3	13.3
8.11.78	11.5	14.0	11.7	11.5	11.4	11.7
21.11.78	9.9	13.4	11.4	11.1	10.9	11.0
14.12.78	6.3	12.1	10.1	10.0	9.6	9.5
11. 7.79	20.5	12.3	13.2	12.7	12.6	12.6
9. 1.80	3.4	4.1	-	-	6.2	6.2
12. 2.80	6.3	5.8	-	-	6.3	6.8

Natural temperature variations in groundwater adjacent to Willow Pool provide evidence that water also flows from the lakes into the aquifer. Table 10.2 shows water temperatures measured at various times between September 1978 and February 1980 at Willow Pool, and corresponding groundwater temperatures measured at boreholes SH/1, SH/2, SH/3 and SH/4 which form a transect perpendicular to the lake shore (see fig. 4.1). The average groundwater temperature measured over the Northmoor area on the same date is also given. There is a temperature gradient between the boreholes and, on average, there is a difference in temperature of approximately 2°C between SH/1, which is nearest to Willow Pool, and the other boreholes. This indicates that seepage into the aquifer is taking place, at least through the eastern bank of the lake. Seepage probably occurs throughout the year. In summer and autumn, groundwater in borehole SH/1 is warmed, relative to groundwater in other parts of the aquifer, by mixing with warm lake water. Cold water from the lake in winter, produces a relative cooling of the groundwater at borehole SH/1.

10.1.2 Ringwood

The groundwater contour maps of the Ringwood area enable a comparison to be made between the effects on the water-table of dewatering large pits and the effects produced when a large site is divided into small units or cells. Six selected groundwater contour maps have been drawn for the Ringwood area, covering the period July 1975 to February 1980. The maps cover the area east of the River Avon, between Ibsley village in the north and Ringwood in the south. This excludes some boreholes which were situated to the north of Ibsley, but these lie outside the main area of gravel workings. To date, gravel excavation is confined to the area south of Ibsley. The main excavations during the study period were the series of pits on Ibsley Airfield, cells 3 through 8 being excavated between 1977 and 1980 (see fig. 1.1). The other gravel workings in the area during this time were at the eastern extension of Linbrook Lake, the pit to the west of Spinnaker Lake, and the series of excavations north of Blashford Farm (west of the A338). The only pits to be dewatered were those on Ibsley Airfield, the others being wet pits.

In the case of the pits worked by the Amey Roadstone Corporation, dewatering first took place at Ellingham (now Ellingham Lake) during 1975.

Regular groundwater and lake measurements were taken by the Wessex Water Authority, commencing in March 1975, on an extensive network set up in the area south of Ibsley. A groundwater contour map has been constructed from data supplied by the water authority, based on levels taken on the 9 July 1975, and this is shown in Fig. 10.6. From this map several important features emerge:

(a) There was a general fall in groundwater levels from north-east to south-west (i.e. towards the R. Avon). The hydraulic gradient appears more gradual in the vicinity of Ibsley Airfield (which, apart from early excavations in cell 1, was then undeveloped) and north of Ibsley, where levels were in the order of [illegible]2.5 metres O.D., but a much steeper gradient was found south of the airfield where extensive areas of gravel had been worked. This is probably due to the decreased hydraulic conductivity of the worked areas, produced by the removal of large volumes of highly permeable aquifer material and the creation of large bodies of open water.

(b) It is apparent that, compared with the Stanton Harcourt area, there is a clearer relationship between the groundwater and the lakes. The water level in a number of lakes appears to be related to the groundwater level in their north-eastern (i.e. upslope) margins. As a result, the water level at the south-western margin of the lakes is artifically higher than the natural water-table. This relationship would suggest that the lakes are in continuity with the water-table in the gravels (unlike the pits in Stanton Harcourt, those in the Ringwood area were not lined with overburden or clay prior to flooding). The interactions between lakes and groundwater are studied in greater detail in Chapter 12.

(c) A well developed asymmetrical cone of depression formed around Ellingham Pit, which is elongated in the direction of groundwater flow. The hydraulic gradient was very steep to the north and east of the pit, whereas to the south the gradient was much less (although the zone of influence extended over a greater distance). This effect can also be noticed around the Brown Pits in

Stanton Harcourt (particularyin figs. 10.3, 10.4 and 10.5), where the zone of depression is elongated to the east.

The extent of the effects on the water-table of dewatering Ellingham Pit, led to a change in the local planning policy concerning future developments. Permission for further dewatering was only given if stringent controls were adopted by the developers. The general procedure was that only small areas (or cells) should be dewatered at any one time, and the discharge from the pit should be pumped into an adjacent cell, on the premise that any effect on the water-table would then be localised. One of the aims of this study, therefore, was to monitor the water-table over Ibsley Airfield to assess the efficiency of this system.

By March 1978, development of Ellingham Pit had ceased (although not all the aggregate had been extracted) and the excavation had flooded. Groundwater levels had recovered and all evidence of the cone of depression had vanished (fig. 10.7). The major centre of extraction was now Ibsley Airfield where cells 3, 4, 5 and 6 were being excavated. Cells 1 and 2 had already been worked out and were flooded. Permission to dewater the airfield sites had not yet been given by the water authority, hence wet-digging of the gravel was taking place and each of the working cells was also flooded. The difficulties of extracting gravel from beneath the water-table in an area which is not being dewatered has an adverse effect on productivity. To counteract this problem the excavations were limited to a shallow depth (principally to the area above the water-table). Under such working conditions, the available gravel in each cell was exhausted at a much faster rate, hence four cells were being developed at once.

The obvious advantage, in terms of the effect on groundwater, of wet-digging is that no cone of depression is produced. The hydraulic gradient across Ibsley Airfield in fig. 10.7 is virtually identical to that in fig. 10.6. The only visible change was a slight increase in the hydraulic gradient on the eastern side of the airfield, i.e. between the eastern margin of the lakes which were at a level of about 21.92 metres O.D. and borehole R/6 (22.86 metres O.D.).

Under autumn conditions, that is when the lowest groundwater levels occurred, the general pattern of groundwater movements remained unchanged, as shown in fig. 10.8 for 11 October 1978. The major differences between the groundwater contour patterns in figs. 10.7 and 10.8, is that in the latter a slight decrease in the hydraulic gradient is evident in certain areas. This is shown by the lower numbers of contours, principally in the area around Ibsley Airfield. In fig. 10.7, the fall in the height of the water-table between boreholes R/6 and R/2 (see fig. 4.2) is 2.03 metres, whereas in fig. 10.8 the fall is only 1.65 metres, constituting a 19% reduction in the hydraulic gradient. Similarly, to the east of Ivy Lane Lake there was a 26% decrease in the hydraulic gradient between boreholes R/23 and R/34 (see fig. 4.2).

The situation on Ibsley Airfield in October 1978 was the same as that in the previous March, in that permission for the dewatering of the site had not yet been formally granted. Wet digging was confined to the north-eastern corner of the airfield, in cells which were permanently flooded, although the lower lake and groundwater levels in the autumn did allow for slightly easier working.

Permission to dewater the airfield was finally granted in the autumn of 1978, and pumping commenced on cells 4 and 5 (which had been combined) on 13 November 1978. In compliance with the water authority regulations, the pumped water was discharged into an adjacent cell, which in this case was cells 6 and 3 (which had also been combined). Fig. 10.9 shows the groundwater contours for the 1 February 1979. The most noticeable feature of this contour map is that there is no evidence of the development of a zone of drawdown around the dewatered area. Apart from an increase in groundwater levels, which was due to normal winter recharge, this map is virtually identical to that drawn prior to dewatering (fig. 10.8). Although pumping of water from the cells was continuous, this had very little effect on the overall level of water within the pit. The excavation remained flooded, hence no appreciable drawdown of the water-table occurred.

Continuous recorders were installed by the author in cells 4/5 and 3/6 in May 1979, at the request of the water authority, to monitor the water level within the dewatered pit and within the lake into which the water was being pumped. Fig. 10.10 shows the lake level hydrographs between May and October 1979 (when the dewatering of cell 4/5 was stopped). This was the period of natural recession and the water level in the dewatered pit fell from 22.10 metres O.D. to 21.80 metres O.D. The hydrograph for cell 1, which was completely flooded, has been added to show that despite continuous abstraction, the water level in cell 4/5 remained high overall.

Between May and June 1979, the water level in cell 3/6 was higher than in the dewatered cell (i.e. 4/5), which is as one would expect when pumped water is being discharged from one cell to the other.

Table 10.3 Groundwater temperatures on Ibsley Airfield, Ringwood showing the influence of recirculation from cell 3 + 6 (locations shown on fig. 4.2)

Date of Monitoring	Cell 3+6 (°C)	Groundwater temperature (°C)			
		P/52	P/51	P/53	P/22
22.3.80	4.85	4.05	[illegible]	7.2	8.1
22.4.80	12.0	12.3	9.6	9.5	8.9
21.5.80	15.9	16.[illegible]	10.9	11.5	10.0

However, because the excavation remained flooded at all times, the outflow must have been balanced, in part, by inflow from the gravels to the north and east, and/or by recirculation of water through the bund between the two lakes. From July 1979 onwards, however, the relationship was reversed, and the water level was highest in the dewatered cell. This re-established the regional north-east to south-west hydraulic gradient, and implies that there was a transfer of water from cell 4/5 to cell 3/6 through the intervening bund as well as via the pumping. One possibility is that the flow of water out of cell 3/6 to the south and west was greater than the amount pumped in from cell 4/5 and that the latter was recharged by the gravels to the north and east. One thing which has not been taken into account is the size of the excavation being dewatered. Given the large size (equivalent to two cells) and the great depth of water in the excavation when dewatering started, it is very likely that the capacity of the pump was insufficient to have any significant effect on the overall water level.

In September 1979, pumping ceased from cell 4/5 and new excavations started on cells 7 and 8. The former was not dewatered and later in that month the bund seperating it from cell 4/5 was removed. On the 20 September 1979, dewatering commenced on cell 8, the water being discharged into cell 3/6. Since the excavation was dewatered from the start, it was much more successful and, despite the fact that the excavation was never completely dewatered, it was possible to excavate to the base of the gravels. The level of water in the excavation was greater in winter (20.80 metres O.D. in February 1980) than in late autumn (20.52 metres O.D.). This is attributed to increased recharge from the gravels. Most of the visible inflow appeared to be entering along the eastern face of the pit, where the latter forms the bund between the excavation and cell 3/6. This is confirmed by water temperature data. Table 10.3 shows water temperatures taken at borehole R/52 (see fig. 4.2), which lies almost directly between the two cells, and cell 3/6, compared with groundwater temperatures at various other boreholes within the airfield. Recirculation from cell 3/6 in winter had a considerable cooling effect on the groundwater at borehole R/52, relative to other boreholes, whilst in spring it had the opposite warming effect. The difference in head between the pit and cell 3/6 (a distance of 25 to 30 metres) was, on average, approximately 2 metres, indicating that there was a steep hydraulic gradient across the bund.

Figs. 10.11 and 10.12 show the groundwater contour maps on 29 November 1979 and 15 February 1980 respectively. The important feature to note in both maps was the development of a cone of depression around cell 8. The drawdown zone was very localised, however, and this was in marked contrast to the extensive drawdown zone developed around Ellingham Pit (fig. 10.6) or those found around the Brown Pits in Stanton Harcourt. Beyond a radius of about 25 metres from the pit, there appears to have been no effect on the water-table. Even at boreholes close to the pit it was very difficult to distinguish any appreciable drawdown. Fig. 10.13 shows the hydrographs for boreholes R/52 and R/53 (see fig. 4.2) which are directly adjacent to the excavation (i.e. at a radius of 20 metres to 30 metres). During September, there was no evidence of any sharp reduction in groundwater level at either of these boreholes because of the dewatering. On the contrary, it appears that during September and October, the summer groundwater recession came to an end, and levels began to rise quite normally. The only possible consequence of the dewatering was that at boreholes R/52 and R/53, the maximum groundwater level was delayed when compared with other boreholes on Ibsley Airfield. At boreholes R/52 and R/53, the maximum did not occur until April and March respectively, whereas over the rest of the airfield the timing varied between January and February. All the evidence suggests, therefore, that the system of dividing a large site into small working units for dewatering can be very beneficial in terms of reducing the effects on surrounding groundwater levels.

10.1.3 Summary of field data

In shallow terrace gravel areas, large-scale excavations and dewatering schemes have been shown to alter the shape of the groundwater surface in a limited area surrounding the excavation. The following points are a succinct summary of the main conclusions to be drawn from the field data shown in figs. 10.1 to 10.13:

1) In the Stanton Harcourt area, the regional flow of groundwater is in an easterly direction towards the R. Thames.
2) Dewatering of the Brown Pits at Linch Hill between 1977 and 1980 produced a series of cones of depression which resulted in the temporary dewatering of some private sources in the village of Noorthmoor.
3) The position and extent of the cone of depression varied in response to the changes in position of the particular stage of Brown Pit being dewatered.

4) A groundwater divide was formed down-gradient of the dewatered pits, which altered the overall pattern of groundwater flow, i.e. within the zone delimited by the divide, groundwater flow was radial and directed towards the dewatered pit.
5) The zone of depression was still evident around each stage of Brown Pit up to 6 months after dewatering of that stage had ceased.
6) The eventual flooding of each stage brought about a return to the pre-extraction groundwater-flow pattern in those adjacent areas which were not affected by the dewatering of the subsequent excavation.
7) In the Ringwood area, the regional flow of groundwater is in a south-westerly direction towards the R. Avon.
8) The hydraulic gradient across the floodplain terraces was substantially increased south of Ibsley Airfield in an area where a large number of gravel lakes have been formed.
9) No significant cone of depression was observed around any of the cells on Ibsley Airfield which were wet-dug.
10) In contrast with the relatively large excavations at Ellingham and Linch Hill, those cells on Ibsley Airfield which were dewatered produced very localised zones of drawdown (i.e. no more than 25 metres radius from the pit).

10.2 The Expanding-pit model - a numerical model of gravel pit dewatering

The drawdown effect on local groundwater systems of gravel pit dewatering is well known and accepted as fact amongst both hydrogeologists and gravel-pit operators. The majority of the opposition to gravel-pit development, on hydrological grounds, centres on the possible adverse consequences of localised and, in some cases, widespread reductions in mean groundwater levels by dewatering. Rarely has there been any attempt to justify these claims with true scientific evidence. A major drawback has been the lack of scientific study into the effects of gravel pit dewatering and the inherent difficulties involved in trying to predict groundwater behaviour around dewatered gravel pits.

The purpose of this section is to describe the use of a digital model (named the Expanding-pit model) which can be used to simulate the behaviour of an unconfined aquifer around a dewatered pit. The advantages of a model of this sort are twofold. Firstly, it can be used as a predictive tool and, as such, would be useful to gravel pit operators in the development of future sites. Secondly, ~~it~~ is a significant aid in understanding the behaviour of aquifers. Physically realistic values of parameters are chosen for each simulation and the effect of changing the magnitude of these parameters in turn is investigated. It then becomes apparent which factors dominate the aquifer response and which have only minor significance.

In section 10.2.1, the basis of the digital model will be outlined, and in section 10.2.2, a brief description of how the computations were made is given. A sensitivity approach is used in section 10.2.3 to investigate the significance of hydraulic conductivity, and the quantity of water abstracted, on the response of a hypothetical aquifer to dewatering. In section 10.2.4, the investigation is extended to examine the use of the model as a predictive tool. This is done by using the model to predict the drawdown of the water-table around a projected gravel pit development in the Stanton Harcourt study area.

10.2.1 Description of the Expanding-pit model

Groundwater problems can be divided into two groups depending on whether the aquifer is confined or unconfined. Many methods are available for the analysis of confined aquifers, but the unconfined aquifer is more difficult to analyse because of the free surface boundary. No general mathematical method of solution is available, although there are a few solutions for the case of single wells in unconfined aquifers. There are also a number of approximate solutions based on simplifying assumptions, which are useful in a limited number of practical situations. As an alternative, numerical solutions to particular problems can be obtained. The most successful approach is the finite difference method, in which the governing equations are written in finite difference form, resulting in a set of simultaneous equations.

The digital model to be described is based upon a numerical pumping test method developed by Rushton and colleagues at Birmingham University, although it has been substantially modified and re-written by the author (in consultation with Dr. Rushton). A concise description of the original model is given in Rushton and Redshaw (1979), and the numerous papers by Rushton which are listed in the bibliography. For this reason only brief technical details will be given in this section, which will concentrate on the modifications made to the original model in order to simulate the environment of a gravel pit.

The model developed by Rushton is based upon a discrete space - discrete time approximation to the differential equation for two-dimensional, radial, time-variant flow of groundwater to an abstraction well. It is a more versatile alternative to the more usual analytical solutions and curve-fitting techniques for solving these differential equations. It also has the advantage that as many conditions as necessary can be included in a single solution.

Both in the original model and the one to be presented below, the initial assumption is made that vertical components of flow are sufficiently small to be neglected. Therefore, the appropriate form of the differential equation describing horizontal radial flow in a section of aquifer such as that shown in fig. 10.14 is, according to Rushton and Redshaw (1979),

$$\frac{\partial}{\partial r}\left(mk_r \frac{\partial s}{\partial r}\right) + \frac{m}{r} k_r \frac{s}{r} = S \frac{\partial s}{\partial t} + q \qquad (10.1)$$

where s = drawdown below an arbitrary datum

r = radial coordinate

m = saturated thickness of aquifer

k = radial permeability

t = time

S = storage coefficient

q = recharge per unit area

The aquifer is divided into a mesh, with the spacing of the mesh or nodal points increasing logarithmically in the radial direction. This is more convenient than using constant increments of radius, since it results in a closer spacing of nodes adjacent to the abstraction point, where the most substantial changes in drawdown will be expected to occur, and a wider spacing at greater distances from the abstraction point. The abstraction point forms the inner boundary of the model, while the outer limit of the aquifer is normally represented by specifying a no flow condition at the outermost node.

Simultaneous equations are set up at each node balancing the flows into and out of it, in accordance with the principle of continuity. These equations are solved directly by Gaussian elimination at the end of each time step to produce values of the free surface drawdown at each node (defined here as the value of the groundwater potential below an arbitrary datum). The drawdown at each node is initially set to zero. Flow towards the abstraction point, and hence the development of the cone of depression, is initiated and maintained by the discharge during each time interval.

Rushton's numerical model has proved to be a very powerful and versatile tool in evaluating complex groundwater problems. When this approach was applied, by the author, to the study of gravel pit dewatering, the major problem was how best to represent a complex feature such as a gravel pit. The majority of previous examples of the use of the numerical model has involved the evaluation of pumping test data from abstraction wells of small diameter. One of the questions first raised was whether the same model could be used for the analysis of large scale abstractions from large excavations. Similarly, because gravel pits are not static features, a major problem was how to model the growth in size of a pit. The alternative was to represent a pit as being of a constant large radius. This was not considered suitable to provide accurate predictions of groundwater behaviour, particularly during the very early stages when a pit is still of a relatively small radius.

The simplest representation of a gravel pit is that of a circular well of very large diameter. Many previously existing solutions (i.e. Theis, 1935, Hantush, 1964) are unsuitable for the analysis of large-diameter wells because they do not take into account the large storage capacity of the well itself and therefore over-estimate the drawdown. An analytical solution proposed by Papadopulos and Cooper (1967) is more useful, but this only calculates the drawdown at the well face. The numerical model developed by Rushton was designed to include the effect of the water contained within the abstraction well. Rushton and Redshaw (1979) used their numerical technique to investigate certain standard problems, one of which was the effect of water contained in large diameter wells. Comparisons between the numerical results and analytical values, using the expression of Papadopulos and Cooper (1967), for the drawdown in a large diameter well showed excellent agreement, although no comparisons were possible at points within the aquifer. The numerical model seems perfectly suitable, therefore, for the analysis of drawdown around a gravel pit which is represented as a very large diameter well.

The problem of incorporating the increase in pit diameter through time can be overcome by representing it as a series of discrete, incremental steps. Starting with the first step, in which the pit radius is set at a relatively low value, each subsequent step represents a proportional expansion in the pit radius until, in the final step, the pit radius reaches the desired maximum value. This concept is better understood by describing in more detail the system of nodes and mesh spacing.

A node represents a theoretical point within the aquifer at which the drawdown at the end of each time step is calculated. Of particular importance is that node R(1) represents the region within, in this case, the gravel pit; that the pit radius is R(2); and that the boundary of the aquifer is R(NMAX). The spacing of the nodes can be adjusted by varying the value of the mesh increment, Δa, where $\Delta a = \ln(10^{1}/m)$. For example, to produce ten mesh intervals for every ten-fold increase in radius, Δa is set to 0.23025851 (i.e. $\ln(10^{1}/10)$), so that each node is positioned at a radius of $R(N) = Rwell.10^{AN}$ where $AN = 1.0 / 10.0 \times (N-2)$. Different mesh spacings may be used, and this requires only the redefinition of Δa and AN.

Certain considerations should be taken into account when choosing the mesh spacing. By decreasing the value of Δa (i.e. by increasing the mesh increment), the nodes become closer together, especially in the important area immediately adjacent to the abstraction point, and the drawdown values approximate more closely to the theoretical values. According to Rushton and Redshaw (1979), any finite difference errors are proportional to the square of the mesh interval. The amount of data required from a solution is also an important consideration. In areas where the water-table is expected to show steep gradients or rapid changes, then fine meshes are recommended. However, as the time interval since abstraction began increases, a coarse mesh can give adequate results for most problems.

The number of steps in the expanding pit numerical model depends upon the mesh spacing chosen. The solution for the first step is obtained at the initial pit radius, which is input at execution time. For the second step, the pit radius is increased from its original value (Rwell) to the radius at node Rwell + 1 and so on at the end of each subsequent step until the maximum pit radius (RMAXwell) is reached. Therefore, with a large number of nodes, the greater will be the number of steps between Rwell and RMAXwell. This will give more accurate results, but should be balanced against the economy of the additional calculations required. The mesh can be designed so that the final increment coincides with the desired position of the maximum pit radius by adjusting the mesh spacing as described above.

The times at which the calculations are performed are important, as is the time interval between each incremental step in pit radius. Clearly the choice of the time step (DELT) is of great significance.

Too large a time step leads to inaccurate solutions, whereas too small a time step is wasteful in effort. If ~~the~~ drawdown immediately after the start of pumping, when water-levels are fluctuating most rapidly, is of the greatest significance, then initial time steps need to be very small. However, with increasing time the fluctuations in groundwater level decrease, so that very small time increments become less significant. At some point, therefore, it is desirable to increase the time increment. One convenient way of doing this is to introduce a logarithmic time increment.

The initial time step is set to a very small value (DELT = 1.0 x 10^{-14} days), so that an accurate representation of the time-drawdown curve can be achieved at the smallest radius. Thereafter, the time interval between calculations increases by a factor of $10^{0.1}$, so providing ten time steps for a ten-fold increase in time. Two undesirable features of using a logarithmic time increment are (a) that solutions are not computed at intervals of convenient units (i.e. days), and (b) that at times greater than 10 days or so, the time increments become so large that a great deal of relevant data may be lost. To overcome these problems, it has been decided to use a system incorporating logarithmic time increments over the early stages of the solution and uniform increments over the later stages. From a time interval of 5 days after the start of pumping a uniform increment of 1 day is used. Experience of using the logarithmic scheme has shown that the value which would be the closest to a whole unit of days is 5.009 days. At 60 days after the start of pumping, the increment is increased to 7 days. It is generally found that at this stage any changes in groundwater levels are very small. Other values can be substituted in the program to suit individual requirements.

The time interval between each increment in pit radius is calculated on the basis of a theoretical rate of gravel extraction. The projected rate of extraction used in the examples to be discussed later in this chapter, based on figures supplied by the Amey Roadstone Corporation, is $500 m^3/d$. The time interval (T days) between increments in pit radius is therefore calculated as

$$T = PITVOL/500 \qquad (10.2)$$

where PITVOL (the volume of aquifer between successive pit radii) is calculated as

$$PITVOL = \pi \cdot (R_{n+1}^2 - R_n^2) \cdot D \qquad (10.3)$$

where R_n is the present pit radius, R_{n+1} is the next pit radius, and D is the depth of the aquifer.

Direct precipitation into a normal open well is generally so small that it is disregarded in most pumping test analyses. When dealing with large gravel pits, however, the effects of precipitation falling directly into the pit may be quite important. The total recharge to the pit from precipitation can be calculated by the equation

$$\pi . Rwell^2 . PREC \qquad (10.4)$$

where PREC is daily precipitation (L/T). This positive recharge, which will lead to a reduction of the drawdown in the pit, is balanced against the quantity of water abstracted by the pump, QABST, at the pit. QABST represents the total volume of water abstracted from the pit in unit time. In the early stages of the test, the majority of this total is contributed by the free water contained initially within the pit; the remainder being drawn from the aquifer. QABST is represented as an equivalent negative recharge and is calculated from the pumping rate QPUMP(L^3/T) (which is an input parameter) by the equation

$$QABST = -QPUMP/(2.\pi . DELA) \qquad (10.5)$$

where DELA is equivalent to Δ a. Thus the effect of precipitation is included by incorporating equations 10.4 and 10.5 to produce the final equation for the equivalent recharge at node 1, which is

$$QABST = \frac{-QPUMP/(PREC. \pi .Rwell^2)}{2.\pi .DELA} \qquad (10.6)$$

When the pit radius reaches RMAXwell the solution can be terminated by setting a maximum time increment for the final step (TMAX). When this time is reached, the calculations are stopped, and the final drawdowns at the remaining node positions are printed. A recovery solution can then be initiated by the input of a new abstraction rate of zero.

Under steady-state conditions, recharge to the aquifer is balanced by the quantity of water discharged at the abstraction point. To allow for recharge to the aquifer, a recharge parameter (RECH) is included in the equations at each node. This implies that recharge occurs over the whole of the aquifer, and that all this recharge is intercepted by the abstraction point. It is therefore only an approximation to the true state of affairs.

When recharge over the aquifer is equal to the discharge from the pit, an equilibrium state will be reached, with the drawdown at the nodes remaining constant with time. In the Expanding-pit model an additional routine has been incorporated which checks whether equilibrium conditions have been reached within the aquifer. If the pit has reached the maximum desired radius (RMAXwell), the program checks for steady-state conditions at the end of each time step by comparing the current drawdown at every node with the drawdown at the end of the previous time step. If the drawdown is equal at each node, the calculations are stopped, otherwise the equations are solved for the next time step. For equilibrium conditions to be reached generally requires a long period of pumping. To prevent the simulation ending prematurely requires that TMAX be set to a very high value or that the test for TMAX being reached be removed, so that the simulation only stops when equilibrium conditions are reached. The latter method can generate large amounts of data (depending on the time increment used) and should be used with caution.

The Expanding-pit model calculates, for each increment in pit radius, the length of time required to dewater the pit to a given water-level at a predefined initial rate of abstraction. When the required drawdown is reached, the model then calculates the discharge necessary to maintain that water-level constant at the pit-face.

The purpose of dewatering a gravel pit is to reduce and maintain, as far as possible, the depth of water within the excavation. Where dewatering is successful, the bottom of the pit may actually be 'dry'. This is represented in the numerical model by setting a maximum drawdown (DAT) at nodes R(1) and R(2). The actual value of DAT will depend upon the field conditions being modelled, but to represent the desired conditions in the pit as far as possible, it should be set to fractionally less than the depth of the excavation (BASE). Under ideal conditions the pit would be dewatered until the water-table at the pit face (Rwell) was equal to, or lower than, the height of the base of the pit, i.e. for a fully penetrating pit, DAT would be equal to the depth of the aquifer. Under normal circumstances, however, this situation is only rarely achieved, and normally a seepage face develops near the base of the pit. In the numerical model, therefore, DAT is set to a height above the base of the excavation, (a) to represent a seepage face, and (b) to prevent the model from calculating excessive drawdowns at the pit nodes (i.e. below the base of the pit) and thus producing zero discharge.

In the Expanding-pit model, the drawdown D at the end of each time step is printed for

(a) the pit itself (R1)
(b) the outer boundary of the aquifer (RMAX)
(c) five 'observation wells'

The positions of the 'observation wells' are supplied at the execution time of the program. The observation wells are positioned at nodal points, hence the user must specify the numbers of the nodal points required. At the beginning of each new increment in pit radius, because the nodes are re-numbered, the position of the 'observation wells' will change. If more exact positions are needed, for example when comparing simulation data with drawdowns measured in a borehole at a known distance from the abstraction point, the mesh could be re-designed so that nodes coincide as near as possible with the required positions. The discharge from the pit, QPUMP1 (L^3/T), is also printed at the end of each time step. At the end of each increment in pit radius the drawdown at all nodes is printed.

The extensive pumping of an aquifer can dewater a significant proportion of the aquifer. The volume of water removed is proportional to the volume of aquifer above the cone of depression surrounding the abstraction point. This value serves as a rough guide for assessing the magnitude of the effects of dewatering. It is calculated at two points within the model; at the point when the water level in the pit reaches the maximum desired drawdown and at the end of each increment in pit radius, by the equation

$$Ve = \pi \cdot (r_n^2 - r_{n-1}^2) \cdot \frac{(s_n + s_{n-1})}{2} \qquad (10.7)$$

where Ve = volume of an element

r_n and r_{n-1} = radius from centre of pit to nodes n and n-1

s_n and s_{n-1} = drawdown at nodes n and n-1

The total volume $V(L^3)$ is calculated by summing Ve for each element in the modelled aquifer. This gives the total volume of aquifer that is dewatered; the actual volume of water removed from the aquifer is obtained by the expression V.S, where S is the storage coefficient.

10.2.2 Executing the computer program

Fig. 10.15 shows a cross-section through a hypothetical gravel aquifer of radius Rmax, which is assumed to be radially symmetrical. In the centre of the aquifer a circular pit of initial radius Rwell is dewatered at an initial rate QPUMP. The diagram illustrates most of the input terms used in the model, and the values are typical of those used in the sensitivity analyses in section 10.2.3.

A minimum of ten input records are required to execute the Expanding-pit model. Records 1 to 8 contain the basic aquifer parameters and record nine contains the data on pumping rates and maximum drawdown required. The solution can be continued at different pumping rates after a given time interval, i.e. to simulate a stepped pumping test or the addition/removal of a pump, by adding additional type 9 records (these have an extra field (TSTOP) specifying the length of time for each of the additional steps). A recovery phase can be simulated by adding an additional type 9 record at the end of the input data with QPUMP equal to zero. An end-of-job record (type 10) should be included at the end of the input data to signify termination of the solution. This is the same as a type 9 record except that QPUMP is given a value less than zero. A more detailed description of the input records needed to execute the computer program is given in Appendix A3. A listing of the program, which was written in FORTRAN for an ICL 2980 computer, is also given in Appendix A3.

10.2.3 A sensitivity analysis of the Expanding-pit model

The reliability of a mathematical model of groundwater flow is a major factor in its use as a predictive tool. Cedergren (1977) suggests that the most important source of inaccuracy in the modelling of groundwater flow problems is the lack of precise information about the main input parameters, particularly permeability, and that this leads to uncertainty over the results. For example, in an unconfined aquifer, small changes in permeability can cause significant variations in groundwater flow. The only means of determining the dependence of the solution on the permeability, or any other parameter, is to carry out a sensitivity analysis. This requires that separate solutions be obtained using a range of possible values for selected parameters.

A sensitivity analysis was carried out to assess the effects of varying (a) aquifer permeability and (b) abstraction rate, on gravel pit dewatering and particularly upon groundwater drawdown. Consider an unconfined gravel aquifer, having an initial saturated depth of 4.5 metres and a storage coefficient of 0.08 (fig. 10.15). A fully penetrating gravel pit will be considered, of initial radius 10 metres, which expands in eleven incremental steps to a maximum radius of 100 metres. This represents a final excavated area of 3.1 ha. (about the size of one cell on Ibsley Airfield). An outer impermeable boundary is assumed to exist at a radius of 1 km from the pit. Recharge and precipitation are fixed at 0.0004m/d and 0.0018m/d respectively.

The initial conditions are that the water-table is horizontal and that water is pumped from the well at a constant rate. For the first series of simulations, a range of permeabilities varying from 10m/d to 150m/d was considered, the initial pumping rate being fixed at $5500m^3/d$. For the second series of simulations, the permeability was fixed at 35m/d and a range of pit discharges varying from $1400m^3/d$ to $7000m^3/d$ was used. The maximum drawdown in all simulations was set to 4.4 metres.

The values chosen for the input parameters are typical of those to be found in the Ringwood and Stanton Harcourt study areas. The range of permeability values used reflects those determined for the gravels in the study areas by dilution gauging (see table 6.1) and the discharge values cover the range quoted by the Amey Roadstone Corporation for the types of pumping equipment used in dewatering their excavations.

The copious amount of data produced by the computer simulations has been summarised using the following techniques:-

(a) time - drawdown curves
(b) distance - drawdown curves
(c) time - discharge curves
(d) volume of aquifer dewatered - time curves

To simplify presentation of the results, data from three selected tests in each group are discussed. These are the ones using the minimum and maximum values, plus one intermediate value. The effects of varying the permeability are discussed first, followed by the effects of varying the pumping rate.

(i) <u>The effects of varying permeability</u>

The influence of permeability upon each of the following factors is considered in turn in this section:-

1) water-table drawdown around a dewatered pit
2) the shape of the cone of depression
3) the volume of aquifer dewatered
4) the recovery of the water-table after the cessation of dewatering
5) the discharge from the pit.

The results are discussed for three simulations in which the permeability was fixed at 10m/d, 75m/d, and 150m/d.

In fig. 10.16 to 10.18, the drawdown of the water-table at various observation points is plotted against the time since pumping started. The overall impression is that water-table levels are lowered faster by dewatering when the permeability is increased. This is particularly significant at greater distances from the pit. For example, when the permeability was set at 10m/d, the drawdown between 500 metres and 1000 metres from the pit remained at zero throughout the test. When the permeability was increased to 150m/d the water-table, at a radius of 1000 metres, fell 1.92 metres in 470 days. Closer to the pit there is very little variation in drawdown at different permeabilities, proximity to the point of abstraction appears to be of greater importance.

When pumping first starts, groundwater is removed from the aquifer immediately adjacent to the point of abstraction. Flow then becomes established at points further from the pit in order to replenish this withdrawal. Therefore, the influence of pumping extends outwards with time. The water abstracted from the pit must come from a reduction of storage within the aquifer, hence the water-table will continue to decline as long as the aquifer is effectively infinite. The rate of decline, however, decreases continuously as the area of influence expands.

The fall in the water-table at each observation point takes place in two phases. In the first phase, the water-table falls rapidly as most of the discharge is withdrawn from close to the pit, but this lasts for only a relatively short time. In the second phase, the fall in the water-table becomes much slower as the effect of pumping extends over a wider area. This trend is particularly visible at points close to the pit, extending further from the pit as the permeability is increased. Thus, when the permeability is fixed at 10m/d, the two phases are most clearly distinguishable only up to a radius of 63 metres from the pit, compared with 250 metres when the permeability is increased to 150m/d.

When water is pumped from a new pit, the initial withdrawal exceeds the rate at which groundwater flows into the vicinity of the pit. The surrounding water-table is therefore lowered. In fig. 10.19 to 10.22, the drawdown at various time intervals is plotted against distance from the face of the pit. The times chosen correspond to the end of an increment in pit radius. The water-table at the pit face is therefore equal to the maximum drawdown (DAT). In three dimensions the drawdown curves describe a conical shape known as the cone of depression.

Table 10.4 The time for maximum drawdown in the pit to be reached[1]

Radius of pit	Aquifer permeability		
	10 m/d	75 m/d	150 m/d
10.0 m	0.316	0.500	0.794
12.6 m	0.003	0.006	0.020
15.8 m	0.005	0.008	0.020
20.0 m	0.008	0.013	0.032
25.1 m	0.013	0.020	0.040
31.6 m	0.020	0.032	0.079
39.8 m	0.032	0.050	0.126
50.1 m	0.050	0.079	0.158
63.1 m	0.079	0.126	0.199
79.4 m	0.126	0.199	0.199
100.0 m	0.251	0.316	0.251

Note 1. The time is calculated in days from the start of each new increment of pit radius

For each permeability, the cone of depression increases in depth and extent with increasing time and as the pit expands. The increase in the slope of the water-table increases the flow of water towards the pit, until it balances the pumping rate. When the rate of recharge to the aquifer is sufficient to maintain the pumping rate, a new equilibrium water-table develops. However, as the pit expands this equilibrium state is destroyed and the cone of depression continues to steepen and enlarge until a steady state is reached at the maximum pit radius.

The permeability of the aquifer affects the shape of the cone of depression developed around the dewatered pit. Comparisons of the drawdown curves at the various time intervals shows that aquifers of low permeability develop tight, shallow cones of depression, whereas aquifers of higher permeability develop deep cones of wide extent. At any given point in time, therefore, the drawdown at any point is directly proportional to the permeability of the aquifer. The implication of this is that, at any given time, the volume of the aquifer dewatered as a result of pumping from a gravel pit, increases as the permeability increases. Fig. 10.23 shows the volume of aquifer dewatered plotted against time since the start of pumping. The volume calculations do not include the excavation itself. This graph gives a very clear indication of the effects of the shape of the cone of depression at different aquifer permeabilities. At a permeability of 10m/d, the volume of aquifer dewatered after one year's pumping (radius of the pit = 100 metres) is $375{,}000m^3$, compared with almost $6{,}000{,}000m^3$ at a permeability of 150m/d. At low permeabilities, the water level in the pit falls rapidly to the maximum drawdown level (table 10.4), before a significant quantity of water is removed from the aquifer. This produces the steep hydraulic gradients close to the pit. At high permeabilities, because groundwater flow into the pit is increased, the time taken for the pit to be dewatered is increased so allowing water to come from distant regions without producing steep hydraulic gradients. Therefore, if the cone of depression has steep sides, only a relatively small proportion of the total aquifer is dewatered.

Fig. 10.23 could be re-drawn, replacing time on the x-axis by pit radius. At all permeabilities an increase in pit radius leads to an increase in the amount of aquifer dewatered, although this effect is increased at a higher permeability.

Having examined the effects of permeability on the drawdown of the water-table during dewatering, the sensitivity analysis can also be

Table 10.5 The time at which recovery commenced at selected points within the aquifer[1]

Aquifer permeability	Radius of observation point					
	100.0 m	158.5 m	251.2 m	398.1 m	794.3 m	1000.0 m
10 m/d	472.8	488.9	507.9	539.9	——[2]	——[2]
75 m/d	472.8	475.4	479.9	491.9	528.9	539.9
150 m/d	472.8	474.1	477.9	486.9	513.9	517.9

Note 1. The time is calculated in days from the start of pumping (pumping ceased after 371 days).

Note 2. Drawdown was zero

used to examine the effects of permeability on the recovery of groundwater levels when pumping ceases. Fig. 10.24 to 10.26 show the drawdown at three observation points, plotted against the time since pumping started for each of the three permeability tests. Pumping ceased after 472 days, but in all cases groundwater levels continued to fall for some period of time after this.

When pumping ceases, groundwater continues to flow towards the pit because of the steep hydraulic gradients produced by dewatering. Therefore, whereas the level of water in the pit and groundwater levels close to the pit begin to rise almost immediately after pumping ceases, groundwater levels at greater distances from the pit continue to fall for some time after.

The recovery of groundwater levels is controlled to a large extent by the permeability of the aquifer. When pumping stops, the groundwater level at a radius of 100 metres, which corresponds to the final position of the pit face, begins to rise almost immediately. As the level at the pit face rises, a seepage face develops along the face, above the water level in the pit. Flow of water occurs across the seepage face and the pit then begins to fill. With the permeability at 10m/d (fig. 10.24), the rate of recovery at the pit face is relatively slow when compared with the recovery in aquifers of higher permeability (figs. 10.25 and 10.26). The flow of groundwater towards the pit, and hence the speed at which the water level in the pit rises, is directly proportional to aquifer permeability.

While the deepest parts of the cone of depression are the first to recover, groundwater levels at points further from the pit continue to fall as groundwater continues to flow towards the pit. Gradually, however, as water levels close to the pit rise, the recovery propogates outwards through the aquifer. Table 10.5 shows the time, after pumping ceased, at which groundwater levels at selected observation points began to recover. The results show that the speed at which the recovery is propogated outwards through the aquifer, is directly proportional to the permeability of the aquifer. Obviously the faster groundwater levels rise close to the pit the faster the effect is transferred through the aquifer.

An important factor which has not been considered, is the effect on recovery of the size of the pit. It is to be expected that the rate of recovery of the aquifer is indirectly proportional to the storage capacity of the pit. For a pit of large capacity, the rise in water level is correspondingly slow. Water will be drawn in from a larger

Table 10.6 Comparison of the volumes of aquifer dewatered at the end of the recovery and pumping phases

Aquifer permeability	Volume dewatered after pumping phase	Volume dewatered after recovery phase	Percentage change
10 m/d	375.163 m^3	440.180 m^3	+ 17.3
75 m/d	2,841,871 m^3	3,012,759 m^3	+ 6.0
150 m/d	5,842,906 m^3	5,832,629 m^3	- 0.2

Table 10.7 **Groundwater levels at selected points within the aquifer at the end of the pumping and recovery phases**

Drawdowns at end of pumping phase[1]						
Aquifer permeability	Radius of observation point					
	100.0m	158.5m	251.2m	398.1m	794.3m	1000.0m
10 m/d	4.400	1.884	0.870	0.223	0	0
75 m/d	4.400	2.634	1.947	1.411	0.903	0.851
150 m/d	4.400	3.213	3.092	2.813	1.954	1.[illegible]17

Drawdowns at end of recovery phase[1]						
Aquifer permeability	Radius of observation point					
	100.0m	158.5m	251.2m	398.1m	794.3m	1000.0m
10 m/d	2.876	1.650	0.819	0.233	0	0
75 m/d	1.427	1.316	1.199	1.074	0.900	0.876
150 m/d	2.035	1.996	1.956	1.913	1.852	1.842

Note 1. Drawdown in metres

area of the surrounding aquifer, so that groundwater levels at the aquifer boundary will continue to fall for a length of time proportional to the size of the pit. On the other hand, given the same aquifer parameters, a smaller pit with less storage capacity will fill relatively quickly. The deepest part of the cone of depression would also recover faster, which would then transfer its effects throughout the rest of the aquifer.

In fig. 10.27, distance - drawdown curves at the end of the recovery period have been plotted for each of the simulations. This can be compared with the same curves plotted at the end of the pumping phase (fig. 10.22). The main point to note is that at a permeability of 10m/d, a large cone of depression is still present, over 100 days after pumping has ceased. At higher permeabilities, the deepest parts of the cone of depression have 'filled out' and an equilibrium state appears to be developing with groundwater levels throughout the aquifer forming an almost horizontal water-table, although still well below the level when pumping started. This seems to be occurring faster at the highest permeability. The difference in water-table height between the pit face and aquifer boundary is 0.193 metres at 150m/d, compared with 0.551 metres at 75m/d and 2.376 metres at 10m/d.

Large parts of the aquifer remain dewatered long after pumping ceases. Recovery of groundwater levels is slow because of the large storage volume of a pit. Table 10.6 compares the volume of aquifer still dewatered at the end of the recovery period with that at the end of the pumping phase. Only at the highest permeability has there been any reduction in volume, and then only by approximately 10,000m^3. At the lower permeabilities there has been an overall increase in volume. This is due entirely to the continued fall of groundwater levels at distance from the pit after pumping ceased. Table 10.7 shows the groundwater level at selected points at the end of the pumping and recovery phases. In the aquifer of highest permeability, any earlier increase in the volume of aquifer dewatered has been overcome because of the early reversal of falling levels. With the permeability at 75m/d and 10m/d, although levels at the end of the recovery phase are rising throughout the aquifer, the reversal has occurred later, so that the groundwater level at some points is still below that at the end of the pumping phase.

Recovery implies that the water-table will eventually reach the level which existed prior to the start of dewatering. Certainly from the evidence of the simulations described above, recovery can, depending on the permeability, be very slow. Although the simulations were not continued until complete recovery was achieved, because of the long computing time required, the results do suggest that the water-table would have eventually recovered to its original level. In fig. 10.27, the drawdown curve for the highest permeability does start to approach its pre-abstraction horizontal shape. Whether the timescale would be the same under field conditions is far from certain. For the simulations, a constant daily recharge factor is used, whereas under field conditions substantial recharge occurs only seasonally. At other times of the year conditions are such that groundwater levels would normally fall. This would at least serve to delay recovery still further. Whether complete recovery ever occurs will depend on the prevailing hydrological conditions, particularly the level of groundwater recharge.

Prolonged pumping of a gravel pit under conditions of a declining water-table, eventually results in an inability of the pump to meet the head requirements of the initial pumping rate under conditions of increasing drawdown. The pumping rate gradually falls off in a manner described in fig. 10.28. This graph shows the rate of abstraction plotted against the time since pumping started. Data has only been plotted for the final pit stage (radius = 100 metres), starting at the time when the drawdown in the pit reached the specified value.

The rate of discharge of a gravel pit is equal to the sum of the rate of flow of water into the pit from the aquifer plus the rate of decrease in volume of water within the pit. In the early stages of dewatering the discharge from the aquifer will be considerably less than the discharge from the pit, because of the substantial storage capacity of the pit. When a constant drawdown is reached in the pit, the abstraction rate will be equal to the rate of groundwater flow into the pit. It is this quantity which is plotted in fig. 10.28. Once a constant drawdown has been reached, the abstraction rate is directly proportional to the permeability of the aquifer.

From an analysis of the output from the numerical model, a theoretical discharge curve can be plotted (fig. 10.29). An original discharge of $5500m^3/d$ was specified and this is shown as curve A. Curve B shows the proportion of discharge contributed by flow from the aquifer. When the dewatering of the gravel pit first starts, at time $t = 10^{-13}$, almost all the discharge is derived from storage within the pit. As time continues and the cone of depression develops, the rate of groundwater discharge from the aquifer increases. Once the drawdown reaches the specified value, discharge from the pit is equal to the discharge from the aquifer and follows the sort of relationship shown in fig. 10.29 (curve C) and fig. 10.28. Under ideal conditions, discharge would continue to decrease until equilibrium conditions were reached, when the quantity abstracted would be equal to the total recharge over the aquifer.

(ii) The effects of varying abstraction rate

Varying the initial discharge rate between $1400m^3/d$ and $7000m^3/d$ produced only very slight changes in groundwater behaviour. The main conclusion is that at a fixed permeability, there is a limiting abstraction rate above which only slight increases in drawdown occur.

Three discharge rates were selected for this series of tests, i.e. $1400m^3/d$, $4200m^3/d$, and $7000m^3/d$. As shown in fig. 10.29 and described in the previous section, the discharge only continues at these rates until the drawdown in the pit reaches the specified value. The abstraction rate then follows the theoretical curve of the form shown in fig. 10.28. Once the pit reaches a constant drawdown, therefore, the initial pumping rate is no longer the controlling factor and the abstraction rate is proportional to the permeability of the aquifer.

The main advantage of a large initial discharge rate, is that total dewatering of the pit occurs sooner. For example, at the initial pit radius of 10 metres, the specified drawdown of 4.4 metres is reached in 0.316 days at an initial pumping rate of $7000m^3/d$, compared with 3.2 days at a pumping rate of $1400m^3/d$.

At discharge rates of $4200^3/d$ and $7000m^3/d$, the pit reached its maximum extent of 100 metres radius in approximately 370 days. Because of the longer time taken to dewater the pit after each increment in pit radius, the same stage was reached in over 740 days at an initial discharge rate of $1400m^3/d$. The effect this has on groundwater drawdown is shown in figs. 10.30, 10.31 and 10.32. At the two highest initial abstraction rates, the pit is able to expand at a faster rate, so that the groundwater drawdown at all points (except the aquifer boundary) is always greater at the same time interval.

Rather than comparing the effects of the abstraction rate on groundwater drawdown at the same point in time, the results can be compared at equal stages in pit development. Figs. 10.33, and 10.34 show distance-drawdown curves plotted at two stages in the development of a theoretical pit, given an initial abstraction rate of $1400m^3/d$ and $7000m^3/d$. The two stages examined are upon reaching a constant pit drawdown of 4.4 metres at a radius of 10 metres (fig. 10.33) and 100 metres (fig. 10.34).

The respective times at which these stages were reached are 3.2 days and 774 days at $1400m^3/d$, and 0.316 days and 371 days at $7000m^3/d$.

The drawdown curves in figs. 10.33 and 10.34 indicate a similar trend to that described in the previous section. After only a short period of dewatering, very steep hydraulic gradients develop close to the pit. As the pit expands, the cone of depression expands over a wider area. The most important point to note however, is that the cone of depression is deeper and wider in extent at the lowest abstraction rate. The low discharge allows groundwater to come from more distant parts of the aquifer, whereas under heavy pumping the water is taken from where it can be obtained most readily, i.e. close to the pit. The latter produces steeper hydraulic gradients, due to the high velocities, with the result that the specified drawdown is reached sooner.

Under low rates of pumping, it is possible to dewater more of the aquifer. Fig. 10.35 shows the volume of aquifer dewatered plotted against the time since pumping commenced. On the basis of time, the volume of aquifer dewatered is directly proportional to the initial pumping rate. Taken on a stage basis, however, the relationship is reversed. The points numbered 1 to 11 on each curve represent the theoretical stages in pit development at each pumping rate; each point representing the time at which a constant well drawdown was reached at each increment in pit radius. The diagram shows that on this basis a greater proportion of the aquifer is dewatered at lower pumping rates.

Given a constant permeability, there appears to be a maximum initial abstraction rate (in this case approximately $4200m^3/d$), above which there is very little change in the shape and extent of the cone of depression. In fig. 10.35, the curve showing the volume of aquifer dewatered at an initial pumping rate of $4200m^3/d$ is almost identical to that at $7000m^3/d$. A comparison of figs. 10.31 and 10.32 shows that the time-drawdown curves are almost identical. Very high initial pumping rates can only be maintained for a relatively short period at the beginning of dewatering. Once a constant drawdown is reached, the pumping rate is dependent upon the rate of flow into the pit. At initial pumping rates of $4200m^3/d$ and $7000m^3/d$, the time at which a constant drawdown is reached in a pit of radius 10 metres is very similar, 0.5 days and 0.3 days respectively. Hence, the drawdown at all points in the aquifer is very similar. Thereafter, the discharge in both cases follows a curve of the type shown in figs. 10.28 and 10.29, and the cones of depression develop at the same rate. This is an important result, for it means that delays in the dewatering of a pit, which inevitably occur in the initial stages when ~~the~~ pumping rate is low, have no significant effect upon the shape and extent of the cone of depression. For the theoretical aquifer used in these sensitivity analyses, the limiting initial pumping rate would appear to be in the order of $4000m^3/d$, but this will be dependent upon the permeability of the aquifer. From an operator's point of view, pumping at higher rates may be advantageous. Dewatering of the pit would occur sooner, with very little or no additional adverse effect on the water-table drawdown. This should, however, be balanced against the additional costs required for pumping equipment.

10.2.4 The Expanding-pit model as a predictive tool

Understanding the flow regime within an aquifer and the factors which affect it is of great importance. Furthermore, it is advantageous to be able to predict the effect of groundwater abstraction on groundwater levels within an aquifer. To show how the numerical model can be used to predict the effects of dewatering, it was used in a series of simulations of the development of a new site in the Stanton Harcourt area.

A. Description of the proposed Watkins Farm development

The site referred to consists of the westernmost 22.5 hectares of Watkins Farm at Northmoor, and an adjoining five hectares of land owned by Christ Church College, Oxford (fig. 10.36). The land lies entirely on the lowest of the R. Thames terraces, the Floodplain - Northmoor terrace. The Amey Roadstone Corporation intend to dig gravel from this site 'dry', which means that water will be pumped from the working pit continuously. From field evidence, it has been shown in section 10.1 that the effect of this pumping will be to produce a zone of depression in the water-table around the pit being dewatered. One of the practical purposes of using the numerical model will be to predict the consequences of this on agricultural activities and on the water level in private wells which lie within the immediate area.

The Amey Roadstone Corporation plan to develop the area in three stages (or cells) as shown in fig. 10.36. After each stage has been worked out, a bund (or barrier) of gravel approximately 15 metres wide will be left between it and the next stage, and the previous stage left to fill with water. The bunds will subsequently be removed to leave the site as a continous water body.

The stages shown in fig. 10.36 will be worked in order from 1 to 3. The approximate locations of the dewatering pump for each stage of working and the routes which the pumped water will take are also shown. Water from stage 1 will be pumped into Linch Hill Brook which is at present carrying water pumped from Brown Pit (stage C). Stage 2 will initially be dewatered into stage 1 until that pit has been flooded and pumping will subsequently be into Linch Hill Brook. Stage 3 will be dewatered into stage 2 and when that pit is full the water will be pumped into the ditch by the side of Chippinghey Cottage and thus into Northmoor Brook.

B. The Prediction Analysis

In order to be able to predict the drawdown around the three stages using the numerical model, it was necessary to determine representative values for the various input parameters used by the computer program. The following values were used (the names in brackets refers to the input variables used in the program):-

1) Hydraulic conductivity (PERM) = 34.4m/d. This value was the mean of the hydraulic conductivity results determined by dye dilution at four boreholes in the Watkins Farm area (SH/12, SH/13, SH/14, and SH/21) (see fig. 4.1 for locations).
2) Storage coefficient (SUNCON) = 0.08. This value was quoted by Ridings et al (1977) from pumping tests on similar gravels in the Thames valley near Maidenhead.
3) Pit dimensions. An initial pit radius (RWELL) of 10 metres was used for all stages. The maximum (or final) pit radius (RWELLM) was determined by calculating the final area of each pit from ARC plans and converting this to an equivalent radius. The final area (and equivalent radius) for each stage is : stage 1 = 5.3 hectares (130 metres), stage 2 = 9.1 hectares (170 metres) and stage 3 = 13.4 hectares (207 metres).
4) Aquifer dimensions. Gravel thicknesses in the Watkins Farm area were determined from unpublished prospecting reports produced by the Amey Roadstone Corporation. The average depth of gravel (discounting the depth of overburden) in each of the three stages (BASE) is 4.6 metres, 4.8 metres and 4.35 metres respectively. It was decided to use a separate value for each stage, rather than calculate a mean for the whole of the Watkins Farm area, because the maximum drawdown around each stage is controlled by the depth of the pit rather than the depth of gravel

around it. The initial depth to the water-table was calculated from borehole measurements in the Watkins Farm area (depth to water-table - depth of overburden). In stage 1 the average depth to the water-table (WLEVEL) was 0.2 metres, in stages 2 and 3 it was 0.3 metres. An arbitrary outer boundary of the aquifer (RMAX) was fixed at a radius of 1000 metres for all three stages, and it was modelled as an impermeable boundary. This condition assumes that no water can be drawn from beyond the outer boundary.

5) Precipitation and recharge. The long term (1815-1975) mean annual rainfall, supplied by the Radcliffe Meteorological Station in Oxford, is 650.5 mm. An average daily precipitation value of 0.0018 metres was used. Monthly recharge values for the gravels of the Thames valley in the Oxford area were supplied by the Thames Water Authority. The mean annual recharge (1971 to 1977) was 145 mm, giving a mean daily recharge of 0.0004 metres.

Three separate runs of the expanding pit model were executed, one to represent each of the three stages of the Watkins Farm development. In each case, the initial pumping rate (QPUMP) was fixed at $5000m^3/d$. This value was chosen arbitrarily, since it was shown in section 10.2.3 that the effect of the initial pumping rate on the final drawdown is very slight. The maximum drawdown at nodes 1 and 2 (DAT) was fixed at 4.5 metres, 4.7 metres, and 4.25 metres, for stages 1, 2 and 3 respectively.

On completion of the three computer runs, the calculated drawdowns around each of the three stages were plotted on a map of the Northmoor area (figs. 10.37, 10.38 and 10.39). For each stage, the pit is assumed to be circular and it is drawn as close as possible to its actual position. It was also assumed that the initial water-table was horizontal, hence the lines of equal drawdown, which form the zone of depression around each stage, are concentric circles. Not all the lines of equal drawdown have been drawn, because drawdowns close to the working pit are so severe that it becomes impractical. It should be remembered, however, that the drawdown at the pit face (i.e. node 2) is equal to DAT (the values of which were given above). The three maps are drawn for the following time intervals after the commencement of dewatering,

1) stage 1 - 637 days
2) stage 2 - 1107 days
3) stage 3 - 1676 days

These figures appear unrealistic, but they are based upon a time between steps in pit radius which was calculated using an assumed extraction rate of $500m^3/d$ (see section 10.2.3). Any over-estimation of the extent of the drawdown zone produced by this method should be balanced against the fact that between each stage the water-table was assumed to have recovered to its pre-dewatering level, whereas in practice the drawdown zone produced by stages 2 and 3 would be superimposed upon that of the preceding stage. Therefore by using a long time interval any major discrepancy introduced by this method should be removed.

The important features to be ascertained from the results of the three analyses can be summarised as follows:-

1) stage 1 - the limit of influence (i.e. the radius from the centre of the pit at which drawdown is zero) is approximately 690 metres and the volume of aquifer which has been dewatered is $1,274,950m^3$. The predicted pumping rate at the end of the analysis (i.e. after 637 days) is $1403m^3/d$. Equilibrium conditions had not been reached after 144 days of dewatering at the maximum pit radius.
2) stage 2 - the limit of influence is approximately 690 metres and the volume of aquifer which has been dewatered is $1,652,961m^3$. The predicted pumping rate at the end of the analysis (i.e. after 1107 days) is $1690m^3/d$. Equilibrium conditions (i.e. constant drawdown and discharge) were reached after 1072 days.
3) stage 3 - the limit of influence is approximately 720 metres and the volume of aquifer which has been dewatered is $1,673,008m^3$. The predicted pumping rate at the end of the analysis (i.e. after 1676 days) is $1654m^3/d$. Equilibrium conditions were reached after 1529 days.

The predictions described above were achieved for a pit located in the centre of a circular aquifer of a radius of 1000 metres. What has not been taken into account is the effect on the dewatering of the original sloping water-table and the adjacent water-filled pits. Although the boundary of the aquifer to the east of Watkins Farm, which is taken to be the R. Thames, is greater than a radius of 1000 metres from any of the stages, the aquifer on the west is bounded by a series of flooded pits at a radius of much less than 1000 metres. For stage 1, the western boundary is formed by Stoneacres Lake and Willow Pool. For stages 2 and 3, the previously worked out stage will form the boundary. Consequently, it is necessary to make appropriate adjustment for the effect of these boundaries before any reliable predictions can be applied to the Watkins Farm area.

To simplify the solution of boundary problems in groundwater flow to the dewatered pits, the method of images was applied (Ferris, 1959, Jacob, 1950). An image is an imaginary well or stream introduced to create a hydraulic flow system which will be equivalent to the effects of a known physical boundary on the flow system. The cone of depression of a dewatered pit is not affected until the boundary is intersected. After that, the shape of the drawdown curve will be changed by the boundary (Viessman et al., 1972).

Detailed information about the position of an aquifer boundary is often hard to obtain and further difficulties arise in determining the actual hydraulic conditions on these boundaries. The flooded excavations to the west of the Watkins Farm excavations were modelled as a constant head boundary. A fixed potential implies that there is an infinite source of water on which the aquifer can draw and sustain the flows required to hold the groundwater potential at a fixed value. The aquifer is in hydraulic continuity with the lakes (see section 10.1), therefore they should provide a sufficient flow of water to prevent development of the cone of depression beyond that boundary.

If the lakes in the Linch Hill area cannot be depleted by the dewatering of Watkins Farm, the boundary limit requires that there shall be zero drawdown at the boundary (Ferris, 1959). To achieve this, the real and bounded aquifer is replaced by an imaginary aquifer of infinite aerial extent, and an imaginary recharging well (or, in this case, pit) is placed on the opposite side of, and equidistant from, the boundary.

The procedure for combining the drawdown curves of the real and image pits to obtain the resulting cone of depression for stage 1 of Watkins Farm is illustrated graphically in fig. 10.40. The real components of the cone of depression of the real well and the cone of impression of the image well are shown as solid lines in the region of real values. As illustrated, the image well returns water to the aquifer at the same rate as it is withdrawn by the real pit. To secure the resultant cone of depression or to evaluate the drawdown at any point in the real region, it is only necessary to add algebraically the real components of the cones of depression and impression. Consequently, this system results in zero drawdown at the boundary which satisfies the boundary conditions. The resultant cone of depression is steepened on the lakeward side of the pit and flattened on the landward side. Similar image analyses were undertaken on stages 2 and 3 of Watkins Farm, and the resultant drawdown contour maps are shown, along with that for stage 1, in figs. 10.41, 10.42 and 10.43.

There now follows a discussion of the likely effects of the predicted drawdowns on private sources and agriculture in the area of Watkins Farm.

C. Predicted effect on local water supplies

A number of private wells lie within the zone of influence of the various stages of working and these are shown on the relevant maps. The well at Lower Farm (SH/15), which is the sole water supply for the owner, will almost certainly be dewatered during each stage of the working. The amount of the drawdown will increase from stage 1 (0 to 0.5 metre) through to stage 3 (1 to 1.5 metre), as the point of dewatering nears Lower Farm. The well at Watkins Farm (SH/17) will also be dewatered during the working of each stage. The owner uses water from the well, but also has mains water installed. Other wells which may be dewatered during one or more of the three stages are at Elm Farm House, West End House (SH/26), Chippinghey (SH/16), Pencots (SH/18), and Greystones (SH/19 and SH/20). None of these sites use water from the wells as a main supply.

D. The effect on agriculture of lowering the water-table by dewatering at Watkins Farm

Whether a lowering of the water-table by dewatering will affect agriculture can be assessed by answering the following questions. In which areas can agriculture at present derive benefit from the water-table and in those areas - given a degree of water-table lowering - will this benefit be unaffected, reduced or disappear, and how can such effects be quantified?

The question is answered by:

1) an analysis of the way in which a water-table can benefit crop growth.
2) an assessment of existing water-table levels.
3) an assessment of the degree of water-table lowering postulated.

In certain situations a water-table can benefit crop growth by supplying moisture to the soil and to the plants themselves. In the Northmoor area, there is an excess of evapotranspiration over rainfall from April to August in average years, amounting in total to approximately 138 mm. This means that for sustained growth, crops are dependent upon the available soil water. In the Northmoor area, the available soil water is always insufficient to offset the climatic moisture deficit, so that crops will always suffer some moisture stress unless moisture can be supplied to the soil from the water-table (i.e. by capillarity). The exact moisture requirement that crops need from the water-table varies from between 28 to 88mm/year (Land & Water Management Ltd., 1977). This is, on average, 6% to 20% of their overall requirement, but will vary considerably from soil to soil, and from year to year. As long as the capillary fringe of the water-table can extend into the soil above the gravel, then it is likely that plants can obtain their requirement from the water-table.

Plants will not always benefit from a water-table. This is particularly so:

(a) where the soils are fine textured and the water-table is high. Capillary rise is expected to keep the topsoil moist, which especially in winter may create anaerobic conditions and give impeded drainage.

(b) where the water-table lies at least 40 cm within the gravels, capillary rise will not be sufficient to take moisture into the soil above (40 cm is taken as the maximum height to which water may rise in gravels due to capillary forces, as proved by experiments described in chapter 8).

The supply of water from groundwater will be most needed when the available soil moisture is most depleted. This will be in summer and autumn. During this period, the natural water-table drops from a maximum level in late winter to a minimum level in late autumn. At its highest level most soils are able to benefit, but as the water-table drops fewer soils can benefit due to (b) above.

E. Areas where the existing water-table can and cannot benefit crop growth

Within the zone that will be affected by dewatering at Watkins Farm, certain areas can benefit from the water-table at present. It follows from the preceding paragraphs that it will be those areas where the water-table is either in the soil profile, or less than 40 cm below the soil in the gravel.

A soil survey of the area surrounding Watkins Farm by Land and Water Management Ltd., revealed the depth to gravel. This is repoduced as a contour map in fig. 10.44. Figs. 10.45 and 10.46 show the depth below the surface of the water-table in October 1977 and January 1978, which may be taken to represent the minimum and maximum water-table levels respectively. The two areas of drawdown, one to the west and one to the north of Watkins Farm, are due to the dewatering of Brown Pit (stage B) and the pumping of the private well at Lower Farm respectively. By superimposing in turn, figs. 10.45 and 10.46 onto fig. 10.44, the areas that cannot benefit from the existing water-table level (because the water-table at present is greater than 40 cms below the top of the gravels) at the start of the growing season (fig. 10.47) and at the end of the growing season (fig. 10.48) can be mapped. It follows therefore, that those areas outside the shaded areas can benefit from the existing water-table. During the growing season there will be a gradual reduction in the area which benefits from the water-table, as the water-table falls, from fig. 10.47 to fig. 10.48.

F. Detrimental effects of the existing water-table

A water-table, if too high can impede soil drainage and adversely affect crop growth. The principal effects of imperfect or poor drainage are reduced aeration, and consequently a restricted rooting zone, as roots will not extend into waterlogged soil.

Other effects of an unsatisfactorily drained soil are puddling, where stocked, and difficulties in getting machinery on and off land. Working the soils when wet can cause structural damage and compaction.

Two of the soil types identified in the Watkins Farm area (fig. 2.2) showed evidence of impeded drainage, due to a combination of a slowly permeable (clayey) subsoil, plus a fluctuating water-table (Land & Water Management Ltd., 1977). There is an approximate correlation between the areas of imperfectly drained and poorly drained soil, and the 0.75 metre minimum depth-to-water-table contour. The imperfectly drained (gleyed brown calcareous) soils will be expected to lie wet in winter, whereas the poorly drained (calcareous gley) soils may be wet even in summer. There appears little doubt that a lowering of the water-table in these areas would be beneficial. The drainage impediment will not be so noticeable in summer, but for winter-sown crops, impeded drainage in winter will be disadvantageous.

G. <u>Areas that will be temporarily affected by the degree of lowering of the water-table that is postulated</u>

The areas that may be adversely affected by a lowered water-table are those which currently benefit from the water-table, but, given the degree of drawdown postulated would cease to do so. Some areas will not be affected, either because at present they draw no benefit from the water-table (figs. 10.47 and 10.48) or because the degree of drawdown will not be so great as to make significant difference.

By superimposing upon figs. 10.41, 10.42 and 10.43, the depth to gravel contours (fig. 10.44) and the existing depth to water-table contours (figs. 10.45 and 10.46), it is possible to indicate those areas which will not be able to draw any benefit from the water-table during the dewatering of the three stages of Watkins Farm. This is shown for both maximum (figs. 10.49, 10.51 and 10.53) and minimum (figs. 10.50, 10.52 and 10.54) water-table conditions. During the year there will be a progression from the areas shown in figs. 10.49, 10.51 and 10.53 to those shown in figs. 10.50, 10.52 and 10.54. Depending on which stage is being dewatered, the areas affected vary slightly. It is evident, by comparing figs. 10.49 to 10.54 with figs. 10.47 to 10.48, that the area of land which will not be able to benefit from the water-table during the growing season will be increased by the dewatering of Watkins Farm. It must be remembered however, that these additional areas will be only temporarily affected; once the whole site is flooded, water-table levels will rise as near as possible to existing water-table levels.

Conversely, the areas that might benefit from a lower water-table are those where the soil is imperfectly or poorly drained at present. These are located by overlaying figs. 10.41, 10.42 and 10.43 on figs. 2.2 and are indicated in figs. 10.55, 10.56 and 10.57. Comparison with figs. 10.49 through to 10.54 indicates that there will be areas which both benefit (from improved drainage), especially during winter and spring, and suffer (from less available water) in summer, from a lowering of the water-table. The relative merits depend mainly on the crop. Those sown in spring will be relatively little affected by impeded drainage in winter, whereas winter sown crops will be. Conversely, those sown in winter are not so dependent upon a high summer water-table because they are harvested in the early summer before a moisture stress is likely to become significant. Spring crops, on the other hand, will be more reliant on adequate moisture in mid-to-late-summer.

H. Conclusions and suggestions on the alternative means of working the Watkins Farm site

The numerical model has been used to predict the drawdown around the three proposed stages of the Watkins Farm development. During the dewatering of each stage drawdown of the water-table will occur. The maximum drawdown, at the end of each stage, has been mapped using an image well technique which takes into account boundary conditions west of Watkins Farm. Drawdown will be most severe immediately adjacent to each stage, where it will exceed 4 metres, but the water-table will rise parabolically away from the pits. The degree of drawdown is steepest to the west of the pits, adjacent to the lakes at Linch Hill, but more extensive to the east of Watkins Farm.

Gravel-extraction at Watkins Farm should have no long term effect on the water-table, because existing water levels will be restored after working. During the excavations some areas will benefit from a lower water-table, but others will be adversely affected.

The method of working currently proposed is to excavate the three stages in order stage 1 to stage 3. An alternative method can be proposed which appears to have certain advantages. For this method the stages are worked in reverse order, so that the first stage to be worked will be stage 3. On the completion of this stage it will be allowed to flood with water and the excavation of stage 2 will commence from the northern side of that stage. This and the subsequent stage will then be dewatered into the lake left by the previous stage. To prevent overflow, a structure to control the final lake level would be installed so that excess water is discharged into an adjacent ditch. This method of working has the following advantages:-

(i) during the working of stages 2 and 1, there will be no drawdown of the water-table on the eastern side of Watkins Farm. This will prevent the excessive dewatering of the private well at Lower Farm.

(ii) some of the water pumped from stages 2 and 1 will be recycled through the gravel bunds left between adjacent stages so that the extent of the drawdown zone will be reduced and the total discharge into the system of ditches will be less than for the first method, so reducing the risk of flooding around Watkins Farm.

A certain amount of reservation should be expressed in the interpretation of the results of the prediction analyses. It has not been possible to gauge the accuracy of the numerical model in reproducing actual field conditions. Strictly speaking the model should be calibrated against field data. There were, however, various problems which made this impossible:-

(i) lack of appropriate data. From examining data produced by the simulation analyses it is apparent that the main changes in the water-table occur soon after dewatering starts and at observation boreholes close to the pit. It was not possible to monitor the drawdown so close to a pit, right from the start of dewatering.

(ii) complicated boundary conditions. In each of the study areas, complicated boundary conditions exist which affect the drawdown pattern around the dewatered pits. It is not possible to incorporate these into the numerical model. The only simple alternatives are to either use the model with the outer boundary set at a very small radius, or to modify the results using image well techniques as described in this section.

(iii) the program models recharge and precipitation on a daily basis. To calibrate the model correctly it would be necessary to modify the program to read in individual daily values.

(iv) the model assumes initially that the water-table in the aquifer is horizontal. In the study areas the water-table is sloping, particularly in the region of existing gravel pits or lakes.

There are, therefore, certain aspects which should be considered when using the model to predict actual drawdowns. This does not, however, entirely detract from the usefulness of the model, since many of these problems can also be applied to more conventional methods of predicting drawdowns. Many of the assumptions made in using the Expanding-pit model appear perfectly acceptable, considering all the other assumptions that are normally made in groundwater analyses. Given that it is debatable whether 'perfect' solutions to groundwater problems are ever achievable, it has been shown that the numerical model described in this chapter can be used to give sensible results based on the available data and forms a good basis for further study.

CHAPTER 11

SEEPAGE INTO DEWATERED GRAVEL PITS FROM GROUNDWATER AND SURFACE BODIES OF WATER

Introduction

A dewatered gravel pit which extends below the water-table will encounter groundwater inflow, or seepage. The rate of inflow will depend upon (a) the size and depth of the excavation, (b) the time interval since dewatering began, and (c) the hydrogeological properties (particularly the permeability) of the gravels (see chapter 10, section 10.2.3).

Groundwater seepage into dewatered pits is visible as surface flow which generally emerges at, or very near to, the base of the gravels. It does not generally occur as a continuous seepage line around the entire perimeter of the pit, but rather as discrete 'springs' or as localised areas of seepage along a particular length of the pit face.

The amount of seepage entering into an excavation is an important factor in the efficient working of gravel pits. This became apparent to the author in February 1979 when Wadham-Brasenose Pit, near Hardwick, became 'flooded'. In normal times, the capacity of the dewatering pump was sufficient to discharge all the seepage which entered into the pit. Flow to a dewatering pump is normally maintained by a system of ditches which intercept and channel seepage from the pit faces towards the pump (plate 11.1 in Appendix A1). These ditches are generally excavated below the level of the base of the gravels (in the case of Wadham-Brasenose Pit, this means into the Oxford Clay), so keeping the bottom of the pit dry. In February 1979, the dewatering pump in Wadham-Brasenose Pit was unable to discharge all the seepage which was entering into the excavation. Seepage had increased most visibly along the southern face of the pit. In some areas, the seepage face was 3 metres above the base of the gravels, compared to a normal height of about 1 metre. The result was that the ditches had overflowed, flooding the base of the pit and so limiting the area available for gravel extraction to that part which was above the water-table.

The reason for this sudden increase in seepage can not be attributed entirely to higher winter recharge. Opposite Wadham-Brasenose Pit, the large Manor Farm Pit was being dewatered simultaneously. Due to problems elsewhere in the dewatering system, the water from Manor Farm Pit was being diverted into Standlake Brook. This drainage channel flows along the southern side of Wadham-Brasenose Pit (see fig. 4.3). A number of days following this, discharge into Standlake Brook from Manor Farm Pit ceased and the water level in Wadham-Brasenose Pit quickly receded to its normal level. The evidence suggests that the increase in the flow of water into Wadham-Brasenose Pit was derived predominantly from increased flow and increased seepage along Standlake Brook.

The Wadham-Brasenose example illustrates the effect excessive seepage can have on gravel working. It is by no means confined to this one area however. Similar problems were also encountered in the Ringwood area, particularly on Ibsley Airfield (the flooding of cells 4 and 5, for example, was described in chapter 10). This chapter discusses the main aspects of seepage into dewatered pits, concentrating mainly on evidence from the Wadham-Brasenose Pit. This particular pit was chosen for two main reasons. Firstly, seepage related features were particularly well developed in this pit and, secondly, early evidence from the field-work showed that seepage into the pit was derived from a number of separate sources, including the R. Windrush. In section 11.1, these sources are identified and then the evidence for seepage derived from Standlake Brook and the R. Windrush is discussed in more detail in sections 11.1.1 and 11.1.2 respectively. In section 11.2, the geomorphic evidence of seepage, i.e. piping and scouring, is discussed. Finally, the factors which are thought to control seepage are discussed in section 11.3. Supplementary evidence from sites other than the Wadham-Brasenose Pit is included in these sections to show that seepage is not a localised problem.

It was not possible to make such a detailed study of seepage in the Ringwood area, although it does undoubtedly occur in those pits investigated, for two reasons. Firstly, water pumped from the cells on Ibsley Airfield is discharged into previously worked-out cells. It is much more difficult to measure the rate of recirculation from a lake than it is from the ditches into which pit discharge is pumped in the Stanton Harcourt area. Secondly, none of the cells on Ibsley Airfield (except cell 8) were sufficiently dewatered for observations or measurements of seepage to be made within them.

Table 11.1 Electrical conductivity of water samples collected in and around Wadham-Brasenose Pit, Hardwick on 29.2.80

Sample Point	Electrical Conductivity (μmhos)	Sample Point	Electrical Conductivity (μmhos)	Sample Point	Electrical Conductivity (μmhos)
1 Spring	490	20 Channel	640	39 Channel	715
2 Spring	465	21 Spring	1550	40 Channel	725
3 Spring	495	22 Spring	1475	41 Pool	1010
4 Channel	585	23 Channel	1550	42 Spring	700
5 Spring	490	24 Spring	1375	43 Channel	785
6 Channel	510	25 Spring	1275	44 Spring	750
7 Spring	800	26 Channel	1150	45 Spring	725
8 Spring	735	27 Pool	620	46 Spring	675
9 Spring	975	28 Channel	675	47 Spring	750
10 Spring	675	29 Channel	675	48 Borehole SH/48	700
11 Spring	500	30 Channel	790	49 Borehole SH/49	800
12 Spring	460	31 Channel	685	50 Borehole SH/50	770
13 Spring	460	32 Channel	1490	51 Borehole SH/51	1480
14 Spring	490	33 Pool	1500	52 Borehole SH/52	1000
15 Channel	585	34 Channel	735	53 Borehole SH/53	780
16 Spring	545	35 Channel	695	54 Borehole SH/54	540
17 Channel	575	36 Channel	685	55 Borehole SH/55	760
18 Channel	585	37 Channel	740	56 R. Windrush	405
19 Spring	585	38 Channel	790	57 Standlake Brook	900

(Sample points refer to fig. 11.1)

11.1 Sources of seepage into Wadham-Brasenose Pit

Routine chemical analyses of samples of groundwater and of seepage entering into Wadham-Brasenose Pit, suggested that the seepage water originated from a number of different sources. Three sources have been identified - the R. Windrush, Standlake Brook and gravel groundwater. Measurements of the electrical conductivity of samples of the inflow taken from the major springs in Wadham-Brasenose Pit, suggested that the seepage from each of these sources was concentrated in different parts of the pit. Seepage entering along the western face of the pit was derived almost entirely from groundwater flow, where as along the northern and south-western faces the inflow was derived in part from the R. Windrush and Standlake Brook respectively.

Fig. 11.1 shows the location of points from which water samples were taken on 29.2.80. The electrical conductivity of each sample was measured in the field and the results are shown in Table 11.1. For comparison, the conductivity of groundwater samples taken from boreholes SH/48 to SH/55 (fig. 4.3) and of samples taken from the R. Windrush and Standlake Brook, are also included.

The electrical conductivity of the inflow taken from the springs along the western face of the pit (samples 16 to 26) varied between 545 and 1550 µmhos (mean = 1028 µmhos). The range of values obtained from the samples of groundwater taken from the boreholes to the west of the pit was 540 to 1480 µmhos (mean = 854 µmhos). It is significant that the overall increase in the electrical conductivity of the inflow from site 16 southwards to site 26 was also matched by the groundwater samples. The conductivity of the groundwater increased southwards from 540 µmhos, adjacent to the R. Windrush, to 1480 µmhos at borehole SH/51. It seems therefore, that seepage along the western face originated from two distinct areas of the aquifer. In turn this indicates the general direction of groundwater flow around the pit. The inflow at springs 16 to 20 is more characteristic of groundwater flow from the north-west, whereas the inflow at springs 21 to 26 is more characteristic of groundwater flow from the south-west, of the pit.

Along the northern and south-western faces of the pit, the electrical conductivity readings of the inflow were relatively low compared with those along the western face. In the north-western corner of the pit (springs 1 to 14), the average conductivity was only 586 µmhos. In the south-western corner (springs 44 to 47), the average conductivity was only 725 µmhos. This is consistent with the dilution

Table 11.2 **Electrical conductivity of water samples collected in and around Wadham-Brasenose Pit, Hardwick on 9.5.80**

Sample Point	Electrical Conductivity (µmhos)	Sample Point	Electrical Conductivity (µmhos)
1 Springs	430–720	12 Channel	810
2 Channel	680	13 Spring	760
3 Channel	640	14 Spring	1230
4 Channel	660	15 Borehole SH/49	850
5 Channel	740	16 Borehole SH/51	1200
6 Channel	760	17 Borehole SH/52	720
7 Channel	900	18 Borehole SH/53	580
8 Channel	880	19 Borehole SH/54	460
9 Channel	780	20 Borehole SH/55	610
10 Channel	580	21 R. Windrush	380
11 Channel	830	22 Hardwick Drain	800

(Sample points refer to fig. 11.2)

of groundwater by surface water. The springs in the north-western corner illustrate this particularly well. The conductivity of inflow at springs 1 to 5 and 11-14 varied between 460 and 500 µmhos. This compares favourably with that of the R. Windrush (405 µmhos).

Within the north-western corner of the pit, there appeared to be some higher than average electrical conductivity readings at springs 7 to 10 (i.e. 675 to 975 µmhos). These values were more characteristic of the inflow along the western face. The electrical conductivity survey was repeated on 9.5.80 (fig. 11.2 and Table 11.2), and a similar situation was observed. At sample point 1, which was a very large composite spring having a number of separate outlets, the conductivity varied between 480 and 720 µmhos. Explanations for these variations will be discussed in section 11.1.2, along with the whole question of induced infiltration of water from the R. Windrush.

The evidence to suggest that seepage into Wadham-Brasenose Pit is derived in part from surface sources is described in the next two sections.

11.1.1 Recirculation of pit discharge from Standlake Brook

This section discusses the recirculation of water between Standlake Brook and Wadham-Brasenose Pit.

Seepage rates from drainage ditches are obtainable either by estimation or by direct measurement. Estimation, using empirical or analytical techniques, is based upon a knowledge of the relevant hydraulic properties of the materials and of the boundary conditions (i.e. depth to groundwater, ditch cross-section, and water depth). The most commonly used methods for the direct measurement of seepage are inflow - outflow techniques, ponding and seepage - meter determinations. Special methods such as the use of tracers, electrical-logging or resistivity measurement, and water-table surveys, are essentially limited to the qualitative assessment of seepage. The use of two more specialised techniques - using water temperature variations and natural hydrochemical tracers - to quantify the proportion of seepage originating from the R. Windrush will be described in section 11.1.2.

The most practical method of measuring seepage is the inflow - outflow method which has been described by Robinson and Rohwer (1959). This is the method which has been used in this study. It involves the measurement of the discharge of water into and out of a chosen section

Table 11.3 Results of inflow - outflow measurements on Standlake Brook, Hardwick on 4.7.79 and 3.9.79

Gauging Section	4.7.79		3.9.79		Maximum water velocity (m/s)
	Discharge (m^3/d)	Seepage loss ($m^3/m/d$)	Discharge (m^3/d)	Seepage loss ($m^3/m/d$)	
A	1588	-	-	-	0.44
B	1710	-	-	-	0.36
C	1881	-	1554	-	0.24
D	5656		6325	- 17.3	0.70
E	-	- 4.2	5753		0.38
F	4728		4993	- 4.1	0.25
G	2788	-	2784	-	

of channel. The difference between the inflow and outflow is then attributed to seepage through the bottom and sides of the channel between the two measuring points.

Discharge from Wadham-Brasenose Pit is pumped directly into Standlake Brook. The seepage rate from a stretch of the ditch which runs parallel to the southern face of the pit was measured on two seperate occasions (4.7.79 and 3.9.79) using the method described above. Fig. 11.3 shows the position of the various points (A to F) along the ditch at which the discharge measurements were made. The discharge values measured at each section are shown in Table 11.3 and fig. 11.3. It is generally assumed that evaporation and precipitation during the observation period has no significant effect on the seepage loss. This seems a valid assumption considering the volumes of water involved and the fact that no rain fell on the sampling days.

The results, particularly those for 4.7.79, show that leakage from Standlake Brook did not occur along its whole length, but appears to have been restricted to that stretch which flows alongside Wadham-Brasenose Pit. Between sections A and C there was a gradual downstream increase in discharge. The increase between sections A and B can be accounted for by inflow from the new cut which enters Standlake Brook downstream of section A. The small increase in discharge between sections B and C was most probably due to observation errors. The ditch at this point was heavily choked with weeds, making measurements very difficult. Between sections C and D, there was a very large increase in discharge. Immediately upstream of section D, water from Wadham-Brasenose Pit is discharged into the ditch by the extractors. By simple subtraction the estimated rate of pumping on 4.7.79 was in excess of $3775m^3/d$. At section F, the discharge decreased to around $4700m^3/d$. This indicates a net seepage loss between sections D and F of over $900m^3/d$. Expressed as a volume per unit length of ditch, the rate of water loss was $4.2m^3/m/d$. Similarly, on 3.9.79, the rate of seepage over the same stretch of ditch was over $1330m^3/d$ or $6.05m^3/m/d$.

The greatest rate of seepage from Standlake Brook was measured along the stretch of ditch which was nearest to the point of dewatering and which was also adjacent to a section of the pit-face which had not been restored with overburden. On. 3.9.79, the discharge along Standlake Brook was also measured at section E, which divided the ditch into two sections. Firstly, a stretch between sections D and E which was directly above the working-face and, secondly, a stretch between sections E and F which was directly above the restored face (fig. 11.3). The

Table 11.4 **Results of inflow - outflow measurements on Linch Hill Brook, Stanton Harcourt on 18.3.78**

Gauging Section	Discharge (m^3/d)	Seepage loss ($m^3/m/d$)	Maximum water velocity (m/s)
A	381		0.07
B	1249	2.0	0.33
C	745	0.3	0.77
D	578	0.1	0.09
E	539		0.11

seepage rate between sections D and E was $17.3m^3/m/d$, compared with only $4.1m^3/m/d$ between sections E and F.

Increases in channel seepage were also identified in the vicinity of the Brown Pits near Northmoor. Along Linch Hill Brook (fig. 1.1), the seepage rate decreased with increasing distance from the point of dewatering. On 18.3.78, seepage rates along Linch Hill Brook, downstream of the Brown Pits (stages A and B), were determined using the inflow-outflow technique. At this time, water from stage B was being discharged into Linch Hill Brook. Discharge was measured at five sections (fig. 11.4). The pumping rate from stage B was estimated at around $870m^3/d$. By far the greatest seepage rate occurred between sections B and C ($2.0m^3/m/d$), falling to only $0.1m^3/m/d$ between sections D and E.

Two factors appear to have a definite effect on the rate of seepage from Standlake and Linch Hill Brooks. These are:

1) The difference in height between the ditch and the water-table, and
2) The velocity of water in the ditch and its effect on the condition of the ditch.

The line of zero drawdown (taken from fig. 10.2) has been drawn in fig. 11.4. It shows the relationship between water-table drawdown and seepage loss. The greatest rate of seepage (between gauging sections B and C) occurred where the difference in height between Linch Hill Brook and the water-table was greatest (i.e. where the drawdown was greatest), whereas the lowest rate of seepage (between sections D and E) occurred outside the zone of drawdown where the water-table was at its natural level. This therefore suggests that the natural seepage rate for the conditions prevailing at the time was $0.1m^3/m/d$, whereas the high rate of $2.0m^3/m/d$, observed between sections B and C, was a direct consequence of dewatering.

The highest rates of seepage in the Standlake Brook occurred from that part of the ditch where the flow was highly turbulent and erosive. The maximum recorded water velocities at each section are shown in Table 11.3. These values compare with a maximum non-erosive velocity for sandy materials ranging between 0.3 and 0.75m/s (Zimmerman, 1966).

The highest velocity measured in Standlake Brook occurred at section D. This was situated approximately 7 metres downstream of the dewatering-pump outlet. Due to the high velocity at this point, the bed of the ditch had been swept clean of all fine sediment and vegetation. As a result, the underlying gravels had been exposed.

Where the dewatering-pump discharged into the ditch, a deep pool had been eroded below the normal level of the bed and the gravels exposed for at least 30 metres downstream of this point.

Downstream of section D, the velocity was considerably reduced. The sediment carrying capacity of the water was decreased, so that between sections E and F a layer of low permeability, soft mud had been deposited over the gravel bed. A thick growth of aquatic plants along this stretch, helped to reduce the velocity and so cause the deposition of more fine material. Over time, this would have had a sealing effect which would considerably reduce the seepage rate.

The amount of recirculation between a drainage channel and a dewatered pit is one factor which determines the efficiency of the dewatering system. Assuming that all the seepage from Standlake Brook re-entered Wadham-Brasenose Pit (because of the hydraulic gradient around a dewatered pit this would seem reasonable), the actual amount of recirculation can be calculated from the inflow-outflow measurements. The amount of recirculation, expressed as a percentage, is calculated as the ratio of total seepage to total pit discharge. In Wadham-Brasenose Pit, the proportion of total pit discharge which recirculated from Standlake Brook was 25% on 4.7.79 and 28% on 3.9.79. Similarly in Brown Pit (stage B), the proportion of recirculation (between gauging sections B and C) was estimated to be 58% on 18.3.78, although some of this may have been entering stage A (i.e. Willow Pool).

The total discharge from Wadham-Brasenose Pit consisted of, (a) the volume of water flowing through the network of channels which drained the base of the pit, plus (b) the volume of seepage entering the pit along the face beneath Standlake Brook. Since much of the latter has already been shown to have originated from Standlake Brook, the amount of recirculation can also be calculated by the formula:

$$Q_r = Q_{total} - Q_{section\ G} \tag{11.1}$$

where Q_r is the amount of recirculation,

Q_{total} is the total discharge from the pit into Standlake Brook,

$Q_{section\ G}$ is the discharge flowing through the channels within the pit, measured at section G (fig. 11.3).

The discharge measured at section G on 4.7.79 and 3.9.79 was 2788m^3/d and 2784m^3/d, respectively. Using the values for Q_{total} which were estimated from the inflow-outflow measurements (i.e. 3775.7m^3/d and 4771.3m^3/d respectively),

$$Q_r \text{ (4.7.79)} = 987.7\text{m}^3/\text{d or } 26\% \text{ of } Q_{total}$$

$$Q_r \text{ (3.9.79)} = 1987.3\text{m}^3/\text{d or } 42\% \text{ of } Q_{total}$$

The value estimated for 4.7.79 compares favourably with the value calculated using the inflow-outflow method (i.e. 26% and 25% respectively), but there is a discrepancy between the values for 3.9.79. This may be due to observational errors, or because a greater proportion of the seepage along the southern face was derived from sources other than from Standlake Brook (i.e. from groundwater).

A significant proportion of the total seepage into a dewatered pit is recirculated. In Wadham-Brasenose Pit, at least 26% of the total seepage was recirculated from Standlake Brook and 45% of this originated from within 30 metres of the pump outlet. To reduce recirculation and therefore increase the economy of the dewatering project, various proposals may be put forward:

1. Pumping lines should be lengthened so that water is discharged at greater distances from the pit. The advantages of this method are that it is relatively inexpensive and flexible. On completion of one dewatering project, the pipeline could easily be removed and installed at the next site.

2. Where recirculation is a particular nuisance or where the material is highly permeable, seepage could be reduced by lining the ditch. This could be done in many ways, ranging from using puddled clay to concrete linings and impermeabile membranes. This method may greatly reduce seepage losses, but depending on the material used it can be expensive.

3. Where water is discharged into a ditch, turbulence should be reduced as far as possible to avoid erosion of the bed material. The use of corrogated-sheeting as a baffle is one method which has been used by the Amey Roadstone Corporation. This measure may not be sufficient on its own, but could be used in conjunction with (1) and (2).

11.1.2 Induced Recharge from the R. Windrush

Seepage entering along the north-west face of Wadham-Brasenose Pit was derived both from groundwater and the R. Windrush. In fig. 11.5, hydrographs for the R. Windrush and boreholes SH/48 to SH/55 have been drawn for the period February 1979 to July 1980. The sharp difference in level between the river and water-table could be interpreted as showing that a head loss develops when river water moves into the aquifer and, therefore, that the hydraulic connection between river and aquifer is not good. On the other hand, the hydraulic gradient between the river and boreholes SH/54 to SH/51 indicates that there is some movement of water from the river into the aquifer, probably as a result of (or at least intensified by) the dewatering of Wadham-Brasenose Pit.

Further evidence (described in the next two sections), based upon field observations of water temperature and water chemistry, has shown that :

a) there is a good hydraulic connection between the river and the aquifer, and

b) seepage entering the pit has a component of river water.

A. Water temperature as an indicator of the infiltration of river water

Water temperature has been used in several studies of groundwater movement. Rorabaugh (1956) used thermometry to study the infiltration of Ohio river water in north-eastern Louisville, Kentucky, and many investigators, including Norris and Spieker (1962), Schneider (1962) and Winslow (1962), have used water temperature to study the directions of groundwater movement. Studies of the relationship between water temperature and the velocity of groundwater movement are described by Stallman (1963) and by Bredehoeft and Papadopulos (1965). Stallman (1965) presents an equation for computing percolation rates from surface water bodies based on the study of groundwater temperature profiles. Boyle and Saleem (1979) used temperature-depth profiles in boreholes to determine the rate of groundwater recharge.

Where infiltration of river water occurs, the temperature of the groundwater will lie between that of the river and of groundwater unaffected by the infiltration. Figs. 11.6 and 11.7 show a traverse perpendicular to the R. Windrush through boreholes SH/51 to SH/54. The lithological divisions shown at borehole SH/51 are taken from an Amey Roadstone Corporation mineral prospecting map (plan No. 723). The water temperatures were measured with a Type KLT Ott Electric Contact gauge (see chapter 4). Temperature readings were taken at intervals

Table 11.5 Temperature of groundwater and river water in the Wadham-Brasenose Pit area, compared with air temperature

Date of measurement	Boreholes								R. Windrush	Air temperature at 0900 hrs.[1]
	SH/48	SH/49	SH/50	SH/51	SH/52	SH/53	SH/54	SH/55		
11. 7.79	14.7	10.7	12.9	-	-	-	-	-	15.6	14.0
23. 8.79	15.5	-	-	12.5	11.3	12.0	14.7	-	-	12.0
28. 9.79	15.0	12.8	14.9	13.3	12.1	12.6	14.7	-	12.3	8.5
14.11.79	11.4	11.4	12.4	12.2	11.3	11.5	12.9	-	4.6	4.5
15.11.79	10.9	-	12.3	-	11.3	11.6	12.8	-	5.1	5.5
19.11.79	10.5	11.3	12.3	12.5	11.4	11.6	12.9	-	5.2	0.0
27.11.79	8.6	10.8	11.6	11.9	11.3	11.2	-	-	3.8	10.5
3.12.79	-	-	10.9	-	-	-	11.9	-	-	9.5
7.12.79	-	-	10.7	-	-	-	-	-	-	9.5
9. 1.80	5.6	7.8	8.8	8.3	9.7	9.2	5.1	-	6.0	2.0
25. 1.80	5.7	7.0	7.6	8.4	9.5	8.2	5.3	-	4.5	0.5
12. 2.80	5.9	6.3	6.7	7.9	8.9	7.9	5.5	8.7	7.[illegible]	6.5
26. 2.80	6.2	6.1	7.7	8.1	8.1	7.6	7.0	8.5	6.6	3.0
29. 2.80	6.6	6.7	8.0	8.3	8.6	7.7	7.0	8.7	7.8	7.0
11. 3.80	6.4	6.7	7.6	8.4	8.2	7.8	7.0	8.9	6.5	10.0
19. 3.80	6.2	6.2	7.2	8.0	7.7	7.3	6.4	8.3	4.9	0.0
23. 4.80	7.4	6.9	8.0	8.5	8.2	8.0	7.7	8.2	10.0	8.5
9. 5.80	8.0	7.4	9.2	8.9	8.7	8.5	8.6	9.0	9.7	11.0
23. 5.80	-	7.8	10.2	9.2	8.9	8.7	9.1	9.1	11.5	11.5
19. 6.80	-	-	-	10.1	10.3	9.9	11.4	11.0	15.3	12.0
30. 7.80	-	10.0	14.1	12.0	11.3	11.4	13.9	-	17.9	16.0

(All temperatures in °C)

1 - Measured at Farmoor Reservoir by Thames Water Authority

Mean annual air temperature (1815-1975), measured at Oxford [illegible].5°C

Table 11.6 Summary of temperature data for groundwater and river water in the Wadham-Brasenose Pit area (July 1979 - June 1980)

	Maximum		Minimum		Range	Time Lag[1]		
	Temp.	Date	Temp.	Date		Max. air - Max. water	Min. air - Min. water	Min. river - Min. water
River Windrush	15.6	11.7.79	4.5	25.1.80	11.1	+16	12	-
Borehole SH/48	15.5	23.8.79	5.6	9.1.80	9.9	27	+4	+16
Borehole SH/49	12.8	28.9.79	6.1	26.2.80	6.7	63	56	32
Borehole SH/50	14.9	28.9.79	6.7	12.2.80	8.2	63	42	18
Borehole SH/51	13.3	28.9.79	7.9	12.2.80	5.6	63	42	18
Borehole SH/52	12.1	28.9.79	7.7	19.3.80	4.4	63	65	53
Borehole SH/53	12.6	28.9.79	7.3	19.3.80	5.3	63	65	53
Borehole SH/54	14.7	23.8.79	5.1	9.1.80	9.6	27	+4	+16
Air temperature[2]	20.0	27.7.79	-5.0	13.1.80	25.0	-	-	-

(All temperatures in °C)

1 In days. A positive value means maximum or minimum water temperature occurs before maximum or minimum air/river temperature

2 Measured at 0900 hrs. at Farmoor Reservoir by Thames Water Authority

of 25 cms below the water surface in each borehole. The upper reading was taken 5 cm below the water surface to reduce the influence of the air temperature. The temperature of the R. Windrush was measured using a standard mercury thermometer.

The isotherms showing the situation on 26.2.80 and 30.7.80 (i.e. one winter and one summer) have been plotted in figs, 11.6 and 11.7. Two clear features can be distinguished from these sections, which are explained in the following paragraphs:

1) a plume of water, cold in winter and warm in summer, extended southwards from the river into the aquifer. This had the effect of creating,
2) a mound of groundwater, warmer in winter and cooler in summer, which was centred on borehole SH/52.

The plume of groundwater between boreholes SH/54 and SH/53 can be interpreted as being the result of the infiltration of river water into the surrounding gravels and its mixing with groundwater of different temperature. The zone of mixing between the two bodies of water is clearly shown in figs. 11.6 and 11.7 by steep temperature gradients around boreholes SH/53, particularly in the summer when the difference in temperature between the river and groundwater was greatest.

The temperature of shallow groundwater is approximately the same as or a little higher than the mean annual air temperature (Table 11.5). The graph in fig. 11.8 shows the temperature of groundwater (measured at a depth of 5 cm below the water surface at boreholes SH/48 to SH/55) and river water plotted against time. A significant seasonal range is evident with high water temperatures in the summer months and lower temperatures in the winter. This indicates that there is a correlation between water temperatures and air temperature. However, groundwater temperatures tend to be lower than air temperatures during the summer and higher than air temperatures during the winter months. The same relationship exists between groundwater temperature and river temperature (fig. 11.8).

A comparison of water temperatures and air temperatures (measured at Farmoor Reservoir by the Thames Water Authority) indicates that there is a time lag between the respective minimum and maximum temperatures. The time lag represents the approximate time required for water from the R. Windrush to percolate through to each borehole under the prevailing hydraulic gradient. Table 11.6 shows the dates of the maximum and minimum water temperature measured at boreholes SH/48 to SH/54 and in the R. Windrush. The times of maximum and minimum air temperatures are also shown.

In boreholes SH/49 to SH/53, the time lag between maximum air and groundwater temperatures was of the order of 2 months. In boreholes SH/48 and SH/54, the time lag was of much shorter duration, i.e. 27 days. This is concurrent with an hypothesis that a plume of warmer water infiltrated into the gravels from the river. The short time lag at borehole SH/48 was probably due to infiltration of water from Standlake Brook.

The time lag between minimum air and water temperatures is also shown in Table 11.6. Unlike the values for maximum temperatures, there is a wide variation in the minimum temperature time lags. According to Table 11.6, the minimum groundwater temperature at boreholes SH/54 and SH/48 occurred before the minimum air and river temperatures. Table 11.5 shows that there is a fairly close relationship between river temperature and air temperature in summer and autumn. In winter and spring (i.e. between mid November 1979 and April 1980), the air temperature gives a less reasonable approximation of river temperature. The large difference between air and river temperatures in winter is probably related to the origin of the river water. Outside of the drawdown zone produced by Wadham-Brasenose Pit, the river is probably recharged by groundwater which in winter will be relatively warm. Also, at its source, the R. Windrush is fed by groundwater from limestone springs which will be warmer than the air in winter.

Air temperature is generally used in computing lag times rather than river temperature (Schneider, 1962), because it is normally air temperature which controls shallow groundwater temperature. According to Schneider, during periods of above freezing temperatures, the graph of mean daily air temperature also gives a reasonable approximation of river temperature. It is obvious from Table 11.5, however, that this is not always so in winter and that more representative estimates of winter lag times would be made by using actual river temperature. The time lag between minimum river and groundwater temperatures is shown in Table 11.6. These results are inconclusive, since they suggest that at the two boreholes which would be expected to be most influenced by infiltration of river water (i.e. boreholes SH/54 and SH/48), minimum groundwater temperatures occurred before the minimum river temperature. One possible explanation is that there is not always complete mixing of the two bodies of water. The steep temperature gradient between the river water and the groundwater (see figs. 11.6 and 11.7) could prevent complete mixing of the two, although the boundary would seem from the transects to be beyond borehole SH/54. The other possible explanation is that because observations were not carried out on a daily basis, the true minimum river temperature was not recorded.

The temperature range at boreholes SH/54 and SH/48 provides further evidence that percolation of water takes place from the R. Windrush and Standlake Brook. Since the temperature of surface water is controlled very largely by air temperature, it would be expected that the temperature range of surface water will be relatively high, although less than the air temperature range because of other factors, such as groundwater seepage. Conversely, because groundwater is largely insulated from sudden changes in air temperature, its temperature range should be relatively small. Table 11.6 shows the range in temperature measured at a depth of 5 cm for the period July 1979 to June 1980 in boreholes SH/48 to SH/54 and the range in temperature of the R. Windrush. The range in air temperature (measured at Farmoor Reservoir) for the same period was 25°C. The relatively high temperature range at boreholes SH/48, SH/50 and SH/54 indicates the main areas of mixing of surface water with groundwater. The smaller temperature range at boreholes SH/51 to SH/53 seems to indicate very little influence from surface water and, therefore, that the mixing of surface water with groundwater was restricted to a narrow zone parallel to the channels.

Evidence based upon groundwater temperature suggests that the R. Windrush loses water to the aquifer, at least along the stretch of river between borehole SH/54 and Wadham-Brasenose Pit. Similar conditions also seem to exist along Standlake Brook, which confirms the findings described in section 11.1.1. An isothermal map drawn for the 28th September 1979 (fig. 11.9), shows the main areas of surface water infiltration. The temperature evidence seems to suggest that seepage from Standlake Brook is more widespread than suggested by the discharge evidence in section 11.1.1, particularly around borehole SH/48. It is probable that most of this leakage originates from the new cut (fig. 11.9) opened up to divert water from an old ditch which flows across the site of the pit. This new ditch is kept clear of weeds and fine sediment, hence seepage through the gravel perimeter should be high (see section 11.1.1).

If the isothermal map in fig. 11.9 is compared with the groundwater contour map drawn on the same day (fig. 11.10), it is apparent that any seepage from the R. Windrush downstream from borehole SH/54 will flow towards Wadham-Brasenose Pit under the influence of the steep hydraulic gradient caused by dewatering. In the next section, the chemical analysis of water entering the pit is used to determine whether a river water component can be recognised.

B. The identification of induced recharge of river water using natural tracers

Introduction

This investigation was carried out to determine the river water component of groundwater in the gravels adjacent to the R. Windrush near Wadham-Brasenose Pit. The method used involved the multi-element monitoring of groundwater from boreholes and seepage points within the pit and comparison of the observed variations with the chemistry of two end-member components, i.e. river water and gravel groundwater.

The technique used is based on that described by Edmunds et al (1976 b) for the estimation of the induced recharge of river water into Chalk boreholes at Taplow, Berkshire. During a 14 day constant rate pumping test, between 67% and 77% of the total discharge was found to be derived from the R. Thames. Similarly, this technique has been used by Edmunds (1972), and Edmunds et al (1976 a) to study induced recharge into Thames alluvial gravels at Dorney, Bucks. Edmunds et al (1976 a) estimated that 36% (± 10%) of the total discharge from a number of pumped wells adjacent to the Thames was river-derived.

There are very few applied geochemical investigations which use natural tracers. Apart from those mentioned above, Pinder and Jones (1969) investigated the chemistry of total run-off to determine the contribution of groundwater to peak discharge, and Broadhead and Mackey (1972) made estimates of induced recharge by chemical methods. The experiments described here are thought to be the first attempts to investigate the induced recharge of river water during gravel pit dewatering using this method.

Selection of natural tracer constituents

The success of using natural tracers depends upon the ability to differentiate between the chemistry of each component source. In this

Table 11.7 Summary of the chemical analyses of water samples collected from the Wadham-Brasenose Pit area

	Ca^{2+} (mg/l)		Mg^{2+} (mg/l)		Na^{+} (mg/l)		K^{+} (mg/l)		Total Alkalinity (mg/l)	
	MEAN	RANGE	MEAN	RANGE	MEAN	RANGE	MEAN	RANGE	MEAN	RANGE
R. Windrush	106	122- 90	4.2	4.8-3.8	10.4	16.25-7.5	3.3	5 -1.175	214	241-198
Borehole										
SH/48	137	170 -91	4.0	4.4-3.6	12.1	15 -7.5	3.3	5.0-1.5	220	256-154
SH/49	215	230-200	4.7	5.5-3.9	18.4	23.8-13.0	3.[illegible]	5.0-1.5	231	257-205
SH/50	145	167-104	4.9	5.3-4.8	9.2	10.6 -7.0	4.8	11.3-1.1	237	249-219
SH/51	347	415-255	12.1	19.8-9.4	15.9	20.0-10.0	5.8	13.8-2.1	524	820-283
SH/52	180	225-110	5.2	5.9-4.6	9.8	15.0 -7.2	3.7	5.1-1.1	211	344-117
SH/53	161	190-135	4.3	5.2-3.7	9.3	12.5 -6.8	3.9	6.3-1.3	258	368-224
SH/54	126	195- 85	6.6	13.2-4.0	12.5	18.3 -8.8	[illegible].2	11.3-1.7	314	600-193
SH/55	181	202-160	6.5	7.3-5.6	12.8	14.4-11.3	4.7	7.5-2	247	283-211
Seepage 1	157	210-105	5.0	5.9-4.3	10.5	15.6- 7.5	3.0	6 -0.55	238	276-195
Seepage 2	211	245-182	4.7	5.0-4.4	16.8	20.6-12.6	5.4	10 -2.2	212	229-188
Seepage 3	455	455	8.25	8.4-8.1	12.5	13.8-11.3	2.5	3.75-1.25	289	317-262
Ditch 1	204	217-190	4.7	5.2-4.4	11.1	11.9 -9.4	3.4	1.75-1.1	242	259-224
Ditch 2	160	172-147	3.85	4.0-3.7	10.0	10.0	2.5	3.75-1.25	236	256-216

Table 11.7 Summary of the chemical analyses of water samples collected from the Wadham-Brasenose Pit area

	SO_4^{2-} (mg/l)		Electrical Conductivity (µmhos)		pH		Total Fe (mg/l)	
	MEAN	RANGE	MEAN	RANGE	MEAN	RANGE	MEAN	RANGE
R. Windrush	56	100- 32	459	510-380	7.9	8.1-7.7	0.5	0.8-0.1
Borehole								
SH/48	156	187-125	697	710-680	7.4	7.6-7.2	1.3	1.4-1.2
SH/49	240	330-150	830	800-850	7.0	7.0	1.7	2.3-1.0
SH/50	140	192- 87	745	770-720	7.0	7.2-6.9	6.4	7.1-5.6
SH/51	450	620-100	1297	1480-970	6.8	7.0-6.4	12.1	16.8-6.1
SH/52	256	290-225	796	1000-660	7.0	7.4-6.8	4.9	6.3-2.9
SH/53	154	180-112	662	780-580	6.9	7.1-6.7	1.7	1.8-1.7
SH/54	47	61- 30	505	550-460	6.9	7.2-6.4	4.3	6.2-2.5
SH/55	162	237- 87	690	820-570	7.6	7.9-7.2	5.0	7.6-2.4
Seepage 1	183	325- 60	679	950-510	6.9	7.2-6.4	3.1	5.2-0.5
Seepage 2	311	362-250	877	1010-760	7.1	7.2-7.0	1.4	4.7-0.2
Seepage 3	795	800-790	1373	1510-1230	6.9	7.0-6.8	20.7	32.0-9.4
Ditch 1	285	375-187	857	900-800	7.5	7.6-7.1	0.9	1.3-0.5
Ditch 2	137	165-110	670	750-590	7.6	7.6	0.9	1.3-0.5

study only two potential sources are considered. These are:

1. alluvial gravel groundwater, and
2. water from the R. Windrush

According to Edmunds et al (1976 b). ideal natural tracers should fulfill two criteria. Firstly, each tracer should have a characteristic composition for each component source and within each source this composition should show low variability. Secondly, the tracer concentration should differ between the component sources.

A survey of the potential contributing sources was carried out to determine those constituents which might be most suitable for use as natural tracers. Samples of groundwater and river water were collected at intervals from July 1979 to June 1980 for chemical analysis. Samples were also collected from the main areas of seepage within Wadham-Brasenose Pit. Seepage sample 1 was taken from the north-west face of the pit, which is nearest to the river. Seepage samples 2 and 3 were collected from the southern and western faces respectively. The results of the chemical analyses are summarised in Table 11.7.

Only those constituents for which precise analytical methods were available were studied. Many of the differentials between natural tracers in geochemical environments are small (Edmunds et al, 1976 b), hence continuity and consistency in both sampling and analysis are important. Pumped samples were obtained from each borehole using the Rock and Taylor automatic water sampling machine (described in Appendix A2). River samples were taken adjacent to the stage-board (fig. 11.10) from the bank. The waters were analysed for Ca^{2+}, Mg^{2+}, Na^{+}, K^{+}, total Fe, SO_4^{2-}, HCO_3^{-}, electrical conductivity and pH. Ca^{2+} and Mg^{2+} were analysed by atomic absorption spectrophotometry, Na^{+} and K^{+} by flame photometry, and total Fe, HCO_3^{-} and SO_4^{2-}. by colorumetric methods. Electrical conductivity and pH were measured in the field using standard meters. Although it would have been possible to use stable isotopes (O^{18} and C^{13}) as tracers, these were not considered because of the expense involved in having the samples analysed and the difficulty in interpreting the results. The results of the chemical analyses (Table 11.7) show that the groundwater exhibited great variability both spatially and within each borehole through time. Such non-systematic variations are bound to occur under natural conditions. They probably reflect lithological variations within the gravels and the complicated pattern of groundwater flow around a dewatered pit.

For all elements considered, the groundwater showed far greater variability than the river. An investigation in detail of the geochemical and other controls on the levels of the potential tracers is outside the scope of this study. However, the high Ca^{2+} and HCO_3^{2-} contents of the groundwater are related and probably due to the solution of the oolitic limestone gravels through which the groundwater flows. The relatively high concentration of Mg^{2+} in the groundwater is more difficult to explain, but may be due to the presence of high magnesian calcite within the oolitic material.

Sulphate in groundwater is usually described as being due to the dissolution of gypsum ($CaSO_4 \cdot 2H_2O$) and anhydrite ($CaSO_4$) within an aquifer. There is no evidence, however, for the occurence of naturally-derived gypsum or anhydrite in the gravels of the Stanton Harcourt area. An alternative proposal is that the sulphate ions were produced by the oxidation of iron sulphides in the gravels. A similar proposal was put forward to explain hydrochemical relationships in the Great Basin of Nevada and California by Winograd and Thordarson (1975). The source of the iron sulphide minerals is probably the Jurassic rocks of the Cotswolds, from which the gravels are derived. Numerous formations throughout the Jurassic succession in the Cotswolds contain iron concretions, ferruginous nodules, ferruginous oolitic limestone, and dispersed iron grains (Richardson et al, 1946).

Surface infiltration which flows towards the water-table will be well-oxygenated through contact with the air. Any iron sulphides in the unsaturated zone or uppermost part of the zone of saturation which this water encounters will then undergo the following oxidation reaction:

$$FeS_2 + \frac{15}{4} O_2 + \frac{7}{2} H_2O \longrightarrow Fe(OH)_3 + 4H^+ + 2SO_4^{2-} \quad (11.1)$$

This reaction produces sulphate ions and also ferric hydroxide. It is this ferric hydroxide which forms the iron-staining seen throughout the gravels in the Stanton Harcourt area. Indeed, in some places the precipitation of ferric hydroxide has occurred to such an extent that it has produced an iron-cemented conglomerate. The subsequent reduction of ferric hydroxide in the saturated zone leads to the production of Fe^{2+} ions according to the following reaction:

$$2Fe(OH)_3 + 4H^+ = 2Fe^{2+} + 5H_2O + \tfrac{1}{2}O_2 \quad (11.2)$$

This reaction indicates that any increase in H^+ ions or decrease in the dissolved oxygen content of groundwater leads to the solution of $Fe(OH)_3$

and reduction of Fe^{3+} to Fe^{2+} (Langmuir, 1969). Therefore, the oxidation of iron sulphides which produces H^+ ~~ions~~ and uses up dissolved oxygen (eqn. 11.1) facilities the solution of Fe $(OH)_3$. This would explain the relatively high total Fe concentrations found in many of the groundwater samples (Table 11.7).

The characteristic smell of hydrogen sulphide (H_2S) in many groundwater samples from the Stanton Harcourt and Ringwood areas suggests that sulphate reduction is an active process. This process may occur where organic matter is in long continued anaerobic contact with water containing sulphate ions, or in association with sulphate-reducing bacteria (Schoeller, 1959).

Starting with cellulose, the complete process of sulphate reduction is represented by Olmsted et al (1973) as follows:

a) Depolymerisation of cellulose (possibly originating from the blocks of peat which are found within the gravels) to sugar

$$(C_6H_{10}O) \longrightarrow X \cdot C_6H_{10}O_5 \qquad (11.3)$$

b) Hydrolysis of the sugar to methane and carbon dioxide

$$C_6H_{10}O_5 + H_2O \longrightarrow 3CH_4 + 3CO_2 \qquad (11.4)$$

c) Reduction of sulphate ions

$$SO_4^{2-} + 2CH_4 \longrightarrow H_2S + CO_3 + H_2O \qquad (11.5)$$

$$SO_4^{2-} + CH_4 \longrightarrow HS^- + HCO_3^- + H_2O \qquad (11.6)$$

Reaction 11.5 shows that production of H_2S takes place as a by-product of the sulphate reduction process. An important point in considering sulphate reduction is that for each chemical equivalent of sulphate ion reduced, one chemical equivalent of carbonate or bicarbonate is formed (reactions 11.5 and 11.6). This process, therefore, may also be partly responsible for the high HCO_3^- content of groundwater in the area around Wadham-Brasenose Pit. At borehole SH/51, where the smell of H_2S was very strong, the HCO_3^- content was as high as 820 mg/l.

Where sulphate reduction is considerable, hardening of the water may also occur (Olmsted et al, 1973). Hardening is a base exchange reaction in which Na^+ ions dissolved in water are replaced by Ca^{2+} and Mg^{2+} ions (Schoeller, 1959).

The processes of hardening and sulphate reduction can be examined using the $Na^+ : (Ca^{2+} + Mg^{2+})$ and $SO_4^{2-} : HCO_3^-$ ratios respectively. Mean values have been calculated for boreholes SH/48 to SH/55, the

Table 11.8 Mean Na^{+} : $(Ca^{2+} + Mg^{2+})$ and SO_4^{2-} : HCO_3^{-} ratios for the River Windrush and groundwater sources

Source	Na^{+} : $Ca^{2+} + Mg^{2+}$ ratio	SO_4^{2-} : HCO_3^{-} ratio
R. Windrush	0.094	0.33
Borehole SH/48	0.086	0.71
Borehole SH/49	0.084	1.04
Borehole SH/50	0.061	0.59
Borehole SH/51	0.044	0.86
Borehole SH/52	0.053	1.21
Borehole SH/53	0.056	0.57
Borehole SH/54	0.094	0.15
Borehole SH/55	0.068	0.66
Seepage 1	0.065	0.77
Seepage 2	0.078	1.47
Seepage 3	0.027	2.75

R. Windrush and the three main seepage points in Wadham-Brasenose Pit. The results are shown in Table 11.8.

The Na^+ : (Ca^{2+} + Mg^{2+}) ratios for the R. Windrush and borehole SH/54 are identical. This is further evidence that the groundwater in the vicinity of borehole SH/54 was composed almost entirely of induced recharge from the river. Along the transect between boreholes SH/54 and SH/51, there was a gradual reduction in the Na^+ : (Ca^{2+} + Mg^{2+}) ratio from 0.094 to 0.044. This indicates that the river water becomes harder as the residence time increases and more Na^+ ions are replaced by Ca^{2+} and Mg^{2+} ions.

If, as Olmsted et al (1973) suggests, hardening and sulphate reduction occur simultaneously, a similar trend should be indicated by the SO_4^{2-} : HCO_3^- ratio. The range of values is quite wide, however, and the ratio actually increased between boreholes SH/54 and SH/51, because of an increase in the SO_4^{2-} concentration along the transect (Table 11.8). The following process may explain this apparent anomaly. River water which enters the aquifer has a relatively high dissolved oxygen content. As it flows through the gravels, it causes the oxidation of iron sulphide minerals, according to reaction 11.1, and an increase in the sulphate concentration of the groundwater. As the water continues to flow through the aquifer, more and more of the dissolved O_2 is consumed until eventually, reducing conditions are established. This favours the growth of anaerobic bacteria and the degradation of organic matter, which is the basis of the sulphate - reducing process.

It is probable that the change from oxidising to reducing conditions takes place in the vicinity of borehole SH/51. The evidence to support this is that (a) the smell of H_2S was strongest at borehole SH/51, (b) there was a reduction in the SO_4^{2-} : HCo_3^- ratio between boreholes SH/52 and SH/51, and (c) the total Fe concentration was highest at borehole SH/51, which is indicative of a reducing environment.

It is apparent that some care should be exercised in the choice of tracers. Only those ions having a high geochemical mobility, that is, those which form minerals with relatively high solubilities and which are least likely to be removed in water-rock reactions should be chosen (Edmunds et al, 1976 b). For this reason Ca^{2+}, Mg^{2+}, Na^+, total Fe, SO_4^{2-} and HCO_3^- should be discounted, because there is evidence to suggest that each may undergo some chemical reaction within the aquifer. It was decided, therefore, that rather than using individual ions, greater reliability would be achieved by examining groups of related ions.

Estimation of percentage induced recharge

Estimation of the proportion of river water in groundwater is possible if the composition of the two end members can be satisfactorily defined. Sampling of the R. Windrush has enabled its composition to be defined adequately. Definition of gravel water composition presents certain problems since lateral variations occur. The dewatering of Wadham-Brasenose Pit had been in operation for a number of years prior to this study, so it is highly likely that the composition of groundwater adjacent to the river and to the pit was already modified to some extent by mixing with induced recharge from the river. In order to define gravel water, therefore, samples from borehole SH/49 have been used. This borehole was chosen because:

a) it is sufficient distance from the R. Windrush to assume negligible alteration by river water, and
b) the isothermal and groundwater contour maps show no indication of river water in this borehole.

Two parameters were finally chosen as tracers. These were the total cation concentration and electrical conductivity. The four major cations (i.e. Ca^{2+}, Mg^{2+}, Na^{+} and K^{+}) have been shown to undergo exchange reactions. However, this should not affect the overall balance of total cations. Conversely, the anions have been shown to undergo quite complex reactions within the aquifer and for this reason the total anion concentration has been discounted. Electrical conductivity has been included because it is generally a good indicator of water quality and gives a good indication of the total dissolved solids content (Hem, 1970). It is also a useful parameter because of the ease in collecting conductivity measurements.

Fig. 11.11 (a) is a summary diagram to illustrate the variations in range of total cation concentration and electrical conductivity between the R. Windrush and gravel groundwater at borehole SH/49. Although the river water and groundwater show a range in values for both parameters, the differential between each is sufficiently large for their use as tracers.

Within the range in composition so defined, a graph has been constructed from which the proportions of river water and gravel groundwater in other water samples can be estimated (fig. 11.11 (b)). The hatched lines represent confidence limits given by the range in total cation concentration and electrical conductivity around the mean (solid lines).

Table 11.9 Proportions of water derived from the River Windrush in groundwater in the area of Wadham-Brasenose Pit

	Total Cations		Electrical conductivity		Mean % river water contribution
	Mean Concentration (mg/l)	% river water contribution	Mean (µmhos)	% river water contribution	
River Windrush	123	100	459	100	100.0
Borehole SH/49	241	0	830	0	0.0
Borehole SH/50	164	65	745	23	44.0
Borehole SH/51	381	0	1297	0	0.0
Borehole SH/52	199	36	796	9	22.5
Borehole SH/53	179	53	662	46	49.5
Borehole SH/54	151	76	505	88	82.0
Borehole SH/55	205	31	690	38	34.5
Seepage 1	175	56	679	40	48.0
Seepage 2	478	0	1373	0	0.0

The percentage contribution of river water at various sources between the river and Wadham-Brasenose Pit, using mean values for both parameters, is shown in Table 11.9. Borehole SH/48 and seepage sample 2 have not been included since they are so distant from the river that the possibility of river water penetrating so far is remote. Also, earlier evidence has shown that leakage from Standlake Brook is more important. The results for both total cation concentration and electrical conductivity are in good agreement. An increase in the proportion of river water can be detected in groundwater in those boreholes nearest to the R. Windrush. For example, the estimated percentage of river water decreased steadily away from the river along the transect between boreholes SH/54 and SH/51. It must be concluded that the induced recharge was a direct consequence of dewatering at Wadham-Brasenose Pit, which produced a steep hydraulic gradient directed away from the river and towards the pit. However, the total depletion in the flow of the R. Windrush must be a certain amount greater than that indicated by the results in Table 11.9. Total depletion is made up of two components, namely induced recharge into the aquifer plus the amount of groundwater flow which under normal circumstances would have discharged directly into the river, but is now diverted by the changes in groundwater flow patterns.

The mean river water contribution in borehole SH/54 was around twice that in borehole SH/50 and at seepage point 1. All three points were approximately equidistant from the river. According to Todd (1959), induced recharge depends upon the rate of pumping, distance from the river, natural groundwater movement and the permeability of the aquifer material. Assuming that the other factors are constant, permeability appears to be the most important factor in controlling the amount of seepage from the R. Windrush. The permeability of the gravels was measured in the area of Wadham-Brasenose Pit using the single well dilution technique (see chapter 6). The results for boreholes SH/50 and SH/54 are 4.30m/d and 6.33m/d respectively. The difference does not seem sufficient to explain the variations in mean river water content.

An additional factor which may influence the amount of seepage from the R. Windrush is channel morphology. Observations show that seepage from the river was directly proportional to channel depth. The R. Windrush, in the vicinity of Wadham-Brasenose Pit, flows through a series of quite pronounced meanders. Fig. 11.12 shows five cross-sections of the river taken between borehole SH/54 and the pit (see fig. 11.10). Borehole SH/54 is situated on the outer bank of one meander (section 1), and borehole

SH/50 is situated on the inner concave bank of the next meander downstream (section 3). Although there was no evidence of scour in the right bank of section 1 to suggest that this was the reason for the higher rates of seepage, the river at section 1 was up to 0.5 metres deeper than at section 3.

The variation in the percentage of river derived water in groundwater between November 1979 and June 1980 is shown in fig. 11.13. The most outstanding feature was the large variation in values at each source except borehole SH/54. The proportion of river derived water at seepage point 1 (i.e. the north-west face) ranged between 6% and 88% over this period. It is possible that seepage from the R. Windrush is concentrated in zones of higher permeability gravels (i.e. layers of 'open-work' gravel or frost wedges). As a result of this and the short distance from the river to the pit, the retention time of the river water will be reduced and mixing of the river water with the groundwater will be poor. It is possible, therefore, that because of the heterogeneous nature of the gravels, samples of groundwater collected from the pit face will indicate large variations in the proportion of river-derived water depending upon the permeability of the gravels exposed at that particular time. These variations, which occur over short distances, are illustrated by measurements of electric conductivity taken on the 29.2.80. Samples of seepage were taken from a short length of face adjacent to the R. Windrush (samples 1 to 13 in fig. 11.1). The range in electrical conductivity was 460 to 975 µmhos, representing values of 0 to 99% river derived water.

The large variation in the river water component at seepage point 1 is also thought to be the result of the dewatering of a temporary extension to Wadham-Brasenose Pit. Between February and March 1980, the following sequence of events were recorded (fig. 11.14),

1) 12.2.80 - a section of overburden was removed along a stretch of land parallel to the R. Windrush.
2) 26.2.80 - the new section was excavated to a depth below the water-table. Seepage was already well developed along the face nearest to the river.
3) 29.2.80 - the new section was extended to point A (fig. 11.14). Measurements of the electrical conductivity of samples of seepage water from the new extension indicated a great variation in the percentage river water component (see section 11.1)

4) 19.3.80 - the new section was excavated to its maximum extent, represented by point B.
5) 23.4.80 - working in the new section had ceased and the extension was completely back-filled with overburden. Contouring and restoration of the northern face of the pit was completed.

The rapid changes in the river water component of groundwater during the new phase of excavation indicates that flow patterns were quickly established around the dewatered pit. If the above sequence of events is compared with the curve for seepage point 1 in fig. 11.13, it can be seen that the maximum river water contribution was recorded on the 19.3.80, i.e. when the temporary section was at its maximum extent. It should also be noted that on the same date, the water-table at borehole SH/50 was at its lowest recorded level. By the 23.4.80, when excavation of the new section had ceased, the percentage river water component was considerably reduced. This evidence shows that the extension of the dewatered pit parallel to the river, resulted in a rapid increase in the amount of induced recharge, although this was quickly reduced again when the excavation was restored. It is felt that this hyposthesis better explains the observed variations in the proportion of river water at seepage point 1, although the effects of variations in permeability on seepage routes must be a contributory factor.

11.2 Piping and scouring caused by seepage of groundwater into dewatered pits

Erosion by piping or scouring was found to be recurrent feature in the majority of dewatered gravel pits examined during this study. The types of erosion feature, their distribution and their possible mode of formation are discussed in this section.

11.2.1 Types and mechanisms of piping and scouring

Seepage of groundwater into dewatered gravel pits generally occur as a steady flow through the interstices of the gravel with little surface disturbance of the material. However, where seepage is particularly intense, erosion of the gravel can occur. This can lead to scouring and erosion at the base of the gravel face and, in extreme cases, to the formation of pipe-like channels. These pipes appear to be similar in nature to those commonly described in alluvial soils (Fletcher et al, 1954). The ones observed in Wadham-Brasenose Pit varied in size, the largest having an outlet which was approximately 0.5 metres in diameter.

Piping in gravel pits is related to areas of concentrated seepage. In Wadham-Brasenose Pit, the most extensive piping occurred along the south-western face, which is adjacent to Standlake Brook (plate 11.2)[1]. Evidence has already been given in section 11.1.1 to show that considerable leakage occurred from this ditch. Although no measurements of discharge were possible, visual evidence suggests that the greatest proportion of this leakage re-entered the pit through the pipes.

The dark colouration of the gravels which can be seen around the seepage points in plate 11. 2 is due to the precipitation of orange-coloured iron compounds from groundwater entering into the pit. Iron in groundwater chemically combines with oxygen in the air to form precipitates of iron oxides and hydroxides. These deposits commonly occur as an orange slime both around springs within the pits and also where seepage enters into drainage ditches.

Piping is a form of erosion which involves the removal of subsurface material (Masannat, 1980). The initiation of piping therefore requires a threshold erosive force that can overcome the erosion-resistance of the gravel. The main erosive force is the seepage force of flowing water. When a pit face is subject to a steep hydraulic gradient, as in the south-western corner of Wadham-Brasenose Pit where the water level in Standlake Brook was considerably higher than the base of the pit, water will flow into the gravels from the point of higher total head (i.e. Standlake Brook) to the point of lower total head (i.e. the base of the pit). As it does so it will experience a gradual reduction in head. This will be accompanied by an erosive seepage force which will tend to remove particles in the direction of flow (Masannat, 1980). According to Freeze and Cherry (1979), the seepage force is directly proportional to the hydraulic gradient.

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1. Plates 11.2 to 11.6 , which are referred to in this chapter, are included in Appendix A1.

Piping may also result from subsurface erosion at the discharge point of water seeping through a pit face. Scouring occurs where the seepage velocity at the discharge point of the water is large enough to produce surface erosion and removal of the gravel. Once initiated, it will tend to proceed progressively backwards along zones of high permeability and low resistance to scour. In extreme cases this may eventually lead to the formation of subsurface pipes. Once the outlet of a pipe is formed, subsurface erosion will extend the pipe progressively backwards by the continuous removal of dislodged particles. The large pipe in plate 11.3 had reached a very advanced stage in its development. Progressive erosion had increased the volume of the pipe until the roof collapsed, resulting in the formation of a steep gully running down the face of the pit.

The most common type of pipe observed in the study areas had a roughly circular cross-section. However, a second type was observed in Wadham-Brasenose Pit which differed both in size and shape (plates 11.4 to 11.6). These pipes were lenticular in cross section, being 0.5 metres high at the centre, and 2 metres in width. The roof, which extended at least 10 metres into the face of the pit, sloped down steeply towards the back forming a cave-like structure. Water was ponded in the base of the pipe behind a threshold of gravel scree, through which water from the pool was seeping. The threshold, which ran along the bottom of the face, appeared to be formed of loose material that had slumped down from the sides of the pit rather than from material eroded from the pipe.

The mode of formation of the lenticular-shaped pipes is difficult to determine accurately. Four of these pipes were observed, situated along the north-west face of Wadham-Brasenose Pit, although they were subsequently destroyed by gravel excavation. The three pipes shown in plate 11.6 were less well developed than that shown in plate 11.4, being merely hollows at the base of the face. It is not certain whether their development was arrested by accumulation of the gravel scree in front, or whether this was somehow linked to their formation. However, their position within the seepage zone does suggest that their formation was linked to the action of groundwater. Their size is incompatible with the present volume of seepage observed from them, which suggests that they were formed at an earlier stage in the excavation of the pit when the volume of seepage was greater. The lack of debris within the pipe tends to indicate that the seepage force at one time was large enough to remove all the eroded material.

It is suggested here that the lenticular-shaped pipes developed initially as scours at the base of the face during an early stage of the excavation when groundwater inflow was greater. The scouring gradually proceeded backwards into the face to form a pipe. The lenticular-shape was possibly controlled by the presence of a horizontal lens of more pervious or less erosion-resistant gravel. As the amount of groundwater inflow decreased through time, it would tend to become concentrated in preferred horizons. As a result, the main pipe would continue to enlarge, whereas the formation of the others, having reduced seepage, would be arrested. The three less well developed pipes in plate 11.6 may have become abandoned in this way. Further reductions in seepage would cause even the largest pipe to be abandoned. It is concluded, therefore, that the gravel scree accumulated after the formation of the pipes and is not a prerequisite for their formation.

11.2.2 Conditions required for piping and scouring

Pipes are a relatively rare feature in most gravel pits, despite the fact that scouring and subsurface erosion seems to be a relatively fast process. For example, in the temporary extension to Wadham-Brasenose Pit (fig. 11.14), pipes were formed within one week of the excavation being opened. Obviously one reason for their scarcity is that excavations along a working face will disrupt and prevent their complete formation. The majority of pipes in the Wadham-Brasenose Pit were found in areas of the pit where extraction had ceased. On the other hand, the restricted distribution of piping and scouring indicates that certain hydrological and/or sedimentological conditions are required to initiate piping erosion.

Piping in gravel pits mainly takes place where the hydraulic gradients and seepage are greatest, i.e. adjacent to surface channels. This water will then tend to become concentrated in the more pervious zones where it is most likely to attain a seepage velocity high enough to dislodge and transport particles. Lenses of open-work gravel, or even frost wedges, for instance, usually have a higher permeability than the gravels in general (see chapter 7), so would form ideal zones for erosion. Open-work gravels are characterised by being poorly sorted and by having a high void ratio, with a sand content that is free to move. Subsurface removal of these sands could easily occur, which would produce an unstable structure with collapsing properties. It was noted that the debris on the floor of most pipes in Wadham-Brasenose Pit consisted predominantly of gravel-size particles with very little sand present.

Thus, two types of erosive processes are thought to occur within Wadham-Brasenose Pit. The first, thought to result from channelised subsurface erosion, is induced by steep hydraulic gradients and develops near surface channels where seepage towards the pit is greatest. This type of piping is particularly active, because of the constant supply of water, and in some cases may lead to gully erosion of the gravel face. The second type may occur anywhere within the pit where the seepage force is large enough to scour sand and gravel particles at the face of the pit. A pipe may form by progressive erosion back into the face. When the seepage velocity falls during the course of dewatering these pipes may become abandoned.

The main conditions necessary for the development of piping erosion in gravel pits can be summarised as follows:

a) the presence of erodible and permeable lenses of gravel,
b) a rapid supply and concentration of water,
c) a hydraulic gradient that develops sufficient seepage force, and
d) a period of stability along a suitable face (i.e. unaffected by excavations).

Piping is potentially quite dangerous. Gullying, which develops where piping erosion is intensive, may lead to undercutting and collapse of a gravel face. In certain circumstances this may have unfavourable results. Gullying had already taken place in the south-western corner of Wadham-Brasenose Pit and appeared to be extending into the face which is directly below Standlake Brook. A major collapse along this face may cause a total failure of the bank and a breach of the channel. The flow would then be diverted into the pit, making dewatering extremely difficult. Similarly, piping erosion in the bunds formed between dewatered pits and lakes, where there is a large head differential, such as between cell 8 and cell 3/6 on Ibsley Airfield (fig. 1.1), could possibly lead to failure of the retaining wall in places and flooding of the pit.

11.3 Factors controlling the distribution of seepage into gravel pits

Seepage tends to occur at discrete points around gravel pits. Fig. 11.1 shows the distribution of major springs in Wadham-Brasenose Pit on 29.2.80. Each arrow represents a point where groundwater was observed flowing into the pit. The concentration of springs in certain areas or along particular faces is typical of all the gravel pits visited during the study. In Wadham-Brasenose Pit, for example, three areas of concentration can be distinguished:

a) a group of large, closely spaced springs, with relatively large discharge, in the north-western corner of the pit,

b) a group of relatively small springs, with relatively low discharge, which are more widely distributed along the western face of the pit, and

c) a group of large, closely spaced springs, with relatively large discharge, which are concentrated in the south-western corner of the pit.

No major springs were observed along the eastern or southern faces of Wadham-Brasenose Pit, which had been restored with top soil. This suggests that the restoration of gravel faces, even when the pit is still being worked, may reduce total groundwater inflow considerably and therefore reduce dewatering costs.

The qualitative classification of springs and seepage used above, is based upon a visual assessment of the discharge rate from each spring and the area of slope covered by seepage. Moist areas of face with little flow of water were classified as small springs. Large springs refer to those areas where water was visibly flowing, often from scour-holes or pipes. As a general guide, the relative volume of discharge in a particular area could be gauged by the côncentration of iron oxide and iron hydroxide deposition.

Two sets of factors appear to control the distribution of major springs in gravel pits; these are geological and hydrological. In simple terms, the geological factors control the occurrence and position of individual springs, whereas their concentration in certain areas is largely controlled by hydrological factors.

A typical gravel deposit contains layers or lenses of material having different permeabilities. Lenses of openwork gravel or frost wedges may serve as preferred routeways for groundwater flow. The major proportion of seepage and the largest springs would, therefore, tend to occur wherever these features are found. This would explain why large springs are unevenly distributed around gravel pits.

In Wadham-Brasenose Pit, the main concentrations of springs - in the north-western and south-western corners - occurred along faces which were adjacent to surface channels, i.e. the R. Windrush and Standlake Brook respectively. These areas were typified by steep hydraulic gradients, which in places had produced scouring and piping. The large discharge from these springs was derived to a great extent from seepage from the surface channels.

A transient flow system develops around a dewatered pit as the water-table declines. As a result, the rate of inflow decreases as a function of time, until a steady-state is reached and the water-table has reached equilibrium (see section 10.2.3). Thus, when an excavation is first dewatered, seepage will tend to occur over a large proportion of its perimeter. However, with increasing time and as the rate of inflow decreases, only the most permeable horizons will still be transmitting water. The length of the effective seepage face will shrink.

Fig. 11.2 shows the distribution of major springs in Wadham-Brasenose Pit on 9.5.80. In comparison with the same map for 29.2.80 (fig. 11.1), it is apparent that there has been a reduction in the number of springs along the western face. Although seasonal factors should be taken into account, it is suggested here that the reduction is partly due to the natural reduction in groundwater inflow as described above. Rather than the discharge of each spring decreasing proportionally with time, groundwater flow is diverted to the largest springs.

There has been no significant reduction in the number of springs in the two areas of greatest concentration. In these areas groundwater flow is in a steady rather than transient-state. Steady-state flow occurs when conditions at the boundaries of the flow region remain unchanged through time. The local flow system around Wadham-Brasenose Pit is bounded on the north by the R. Windrush and on the south by Standlake Brook. Since both these have an adequate flow of water, they can be assumed to represent fixed head boundaries (i.e. they have an infinite source of water on which the aquifer can draw so that the groundwater potential remains fixed). It is therefore suggested here that the large, permanent springs in the north and south-western corners of the pit are supplied by a constant flow of groundwater which at least in part, consists of recharge from the R. Windrush and Standlake Brook.

CHAPTER 12

THE INTERACTIONS BETWEEN GRAVEL LAKES AND GROUNDWATER

12.1 Introduction

Where gravel excavations extend below the natural water-table, lakes will develop on cessation of dewatering. Since it is unlikely that these lakes have a surface inlet, the abandoned pits fill up through effluent seepage of groundwater and through precipitation falling directly into the pit. Similarly, because gravel lakes do not generally overflow or have surface outlets, this inflow either moves down-gradient through the lake boundary, so returning to the aquifer, or returns to the atmosphere by evaporation. It is clear, therefore that gravel lakes are generally in close connection with the surrounding groundwater body, and so form an integral part of a dynamic groundwater-flow system.

Lakes into which groundwater enters and from which water leaves by seeping through the bottom and banks of the lake, are termed seepage lakes (Born et al, 1974). Seepage is used here in the sense of water moving across the interface between a porous medium and either a water body or air. It indicates that such lakes are groundwater dominated.

The subject of groundwater/surface-water relationships around gravel lakes has received very limited attention. The work of Peaudecerf (1975) in France, and of Wrobel (1980) in Germany, are notable exceptions. However, information on this topic is sparse in Britain.

Gravel lakes are important hydrologically because they influence the hydrodynamic behaviour of the groundwater. In particular they affect groundwater levels and the direction of groundwater flow. They may also constitute a preferred routeway for the flow of groundwater, although as a result of sealing of the bed and banks of the lake, they may eventually constitute an obstacle to groundwater flow by restricting outflow on the downstream side. In terms of the water balance of an area, the increased loss of water through evaporation from a large number of lakes, particularly in summer, must be noted.

Probably of greatest importance is the effects of gravel lakes on water quality, particualrly where the lakes are used for recreational purposes. The waters of gravel lakes can quickly become populated by aquatic plants, particularly algae, and bacteria, and the chemical and biological characteristics of the water may become considerably modified relative to the surrounding groundwater. In a completely open system, these changes may eventually affect the quality of the groundwater downstream of the lake.

Gravel lakes are particularly vulnerable to pollution. This has implications not only for the quality of the lake water, but, where there are significant transfers of water between the lake and the aquifer, it may also lead to pollution of the groundwater system downstream of the lake and possible deterioration of any water-supplies abstracted from that part of the aquifer.

The purpose of this chapter, is to examine the interactions between gravel lakes and the groundwater system. In section 12.2, the influence of gravel lakes on groundwater levels and flow is examined, and the effects of sealing of the bed and banks of the lake are discussed. In section 12.3, estimates of the rate of water movement through gravel lakes are made. Finally, in section 12.4, the changes in lake water quality and its effects on groundwater quality are determined. These points will be examined principally with reference to the gravel lakes in the Ringwood area, although supplementary evidence will also be drawn from those in the Stanton Harcourt area.

12.2 The relationships between gravel lakes and groundwater

The nature of the influence of a gravel lake on groundwater levels, the direction of groundwater flow and the transfer of water between the lake and the aquifer, depends on many factors. The main ones and the ones which will be considered here, are:

(a) the extent of sealing of the sides and the bottom of the lake,

(b) the effect of surface outlets, and

(c) the shape, dimensions and orientation of the lakes, with respect to the general direction of groundwater flow.

One other factor which, although important, is not dealt with here in any great detail, concerns the position of the lake in the groundwater flow system. Born et al (1974) described three types of lake based on the pattern of groundwater flow around them (fig. 12.1). In the upstream section of an aquifer, lakes may contribute water to the groundwater zone by way of influent seepage. These are termed recharge lakes by Born (fig. 12.1b). Conversely, lakes in the downstream zone of an aquifer, gain groundwater through the entire lake boundary, and are termed discharge lakes (fig. 12.1a). In areas of lateral groundwater flow, however, lakes may gain water on the upstream side and, at the same time, lose water on the downstream side. These are generally termed flow-through lakes (fig. 12.1c). Gravel lakes in the Ringwood area fall into this third category.

Fluctuations of the flow system around gravel lakes are not uncommon. For some time following the removal of the dewatering pump, when the excavation begins to flood, there is still a steep hydraulic gradient towards the pit (see section 10.2.3). In such circumstances, the lake clearly acts as a discharge point for groundwater. Conversely, where a lake receives discharge, i.e. from the dewatering of other pits, there may well be an appreciable increase in the water level of the lake above the level of the surrounding groundwater table. This situation would induce discharge of water into the aquifer.

The position of a gravel lake in the groundwater flow system can be determined by studying the hydraulic gradient in the immediate vicinity of the lake and its relationship to the water level in the lake (i.e. see the groundwater contour maps in chapter 10). An example is shown in the following section.

12.2.1 Groundwater flow around gravel lakes

The groundwater flow configuration around Spinnaker Lake on 1 February 1979 is shown in fig. 12.2. This situation is typical of the gravel lakes in the Ringwood area. The direction of groundwater flow was inferred from water-table observations around the lake. The lake level was continuously monitored using an autographic recorder (see chapter 4). This map shows that groundwater levels are higher on the eastern side of the lake than they are on the western side. The difference is approximately 0.5 metre. Groundwater therefore flows from east to west through the lake. From the orientation of the groundwater contours, the influent and effluent parts of the lake can be clearly defined.

The groundwater contours are deflected around gravel lakes. The removal of aquifer material during the extraction phase produces a zone of very high permeability within the groundwater system. Groundwater is drawn in from all sides towards the excavation, because it constitutes a preferred routeway for the flow of water. In effect, a greater amount of groundwater is channelled through the lake cross-section than through the equivalent thickness of gravel over the same area.

12.2.2 The influence of groundwater on gravel lake levels

In a homogeneous and isotropic aquifer, with no sealing of the lake banks, effluent seepage into a gravel lake would be balanced by influent seepage from the opposite side of the lake into the aquifer. Under such conditions, the initial lake level would be the mean of the pre-extraction groundwater levels at the upstream and downstream shores (fig. 12.3). Although ideal circumstances very rarely exist, particularly when dealing with unconsolidated sand and gravel aquifers, this concept enables estimates of the lake level, immediately after the completion of the extraction phase, to be made.

The initial, unaffected lake level should rise to the height of the pre-extraction water-table which would have occurred at a point approximately half way between the upstream and downstream banks. The points where the line representing the pre-extraction hydraulic gradient intersects the surface of the lake would, on a map, plot as a line which approximately bisects the lake. This line is termed the 'Kippungslinie' by Wrobel (1980) (fig. 12.3). The Kippungslinie is, therefore, the continuation of the equipotential line which is equal to the height of the lake surface.

Any subsequent shift in the position of the Kippungslinie represents an overall change in lake-level relative to the original hydraulic gradient. The main factors which appear to influence the long-term lake levels are sealing of the lake boundary and artificial abstractions or additions of water. It would therefore seem that this is a relatively simple, indirect method of determining the presence and extent of lake sealing; a process which has generally been very difficult to determine in the past.

It is possible to construct a series of Kippungslinien for any lake at different time intervals and compare their position with the predicted post-extraction Kippungslinie, which will always be at the mid-point of the lake. It would then be possible to determine the current state of the lake (including the intensity of sealing) and its effect on the groundwater system.

Table 12.1 **Predicted and actual lake levels under high- and low-water conditions for four lakes in the Ringwood area**

Predicted level = hp
Actual level = ha
Difference (ha-hp) = d

Spinnaker Lake:

HW (5. 4.79)	hp = 21.30 m;	ha = 21.55 m;	d = 0.25 m
LW (6.12.78)	hp = 20.65 m;	ha = 20.97 m;	d = 0.32 m

Ellingham Lake:

HW (5. 4.79)	hp = 19.43 m;	ha = 19.61 m;	d = 0.18 m
LW (6.12.78)	hp = 18.65 m;	ha = 19.10 m;	d = 0.45 m

Cell 1, Ibsley Airfield:

HW (5. 4.79)	hp = 22.18 m;	ha = 22.03 m;	d = -0.15 m
LW (4. 9.78)	hp = 21.35 m;	ha = 21.19 m;	d = -0.16 m

Ivy Lane Lake:

HW (28. 5.79)	hp = 20.28 m;	ha = 20.78 m;	d = 0.50 m
LW (11.10.78)	hp = 20.00 m;	ha = 20.43 m;	d = 0.43 m

The major problem in calculating the position of the Kippungslinie was in determining the pre-extraction hydraulic gradient in the area now occupied by the lake. All the lakes studied were formed prior to this study, so there was not a sufficient period of groundwater observations available. The hydraulic gradients were therefore extrapolated from adjacent areas which were similar topographically, and geologically, and which were uninfluenced by gravel extraction.

Figure 12.4 shows a cross-section through Spinnaker Lake from east to west. The exact depth of the excavation has not been surveyed, but from information provided by the gravel operators, the average depth must have been between 4 and 6 metres. During the groundwater year 1978 to 1979, the minimum and maximum lake levels occurred on 6 December 1978 and 5 April 1979 respectively. The pre-extraction hydraulic gradient across Spinnaker Lake was estimated to be approximately 0.0024. Using the observed groundwater level at borehole R/23 on 6 December 1978 and 5 April 1979 as a base point, the pre-extraction water-table has been plotted. Assuming, firstly, that the lake boundary is free from silting, the lake surface would rise to the height of the pre-extraction water-table at the mid-point of the lake. This is shown for the low and high water conditions in fig. 12.4 (A and B respectively). The point at which the pre-extraction water-table intersects the mid-point of the lake is the predicted or uninfluenced Kippungslinie. At low water, the predicted lake level is 20.65 metres O.D. and at high water it is 21.30 metres O.D. The actual levels on the same days were 20.97 metres O.D. and 21.55 metres O.D. respectively. The point at which the actual or observed lake level intersects the pre-extraction water-table is the actual Kippungslinie. Figure 12.4 indicates that since Spinnaker Lake was formed, the Kippungslinie has been displaced upstream. Moreover, the low water Kippungslinie has been offset further than that at high water. Figure 12.5 shows that the actual lake level is at a height which is slightly more than midway between the upstream and downstream groundwater levels. It also shows that the difference is greater in summer and autumn, when water levels are lowest.

Table 12.1 shows the predicted and actual lake levels at high and low water for four lakes studied in the Ringwood area. The predicted levels were determined using the method described above for Spinnaker Lake. The equivalent predicted and actual Kippungslinien for each lake are shown in plan view in figs. 12.6, 12.7. 12.8 and 12.9 (a and b).

Table 12.2 The predicted and actual length of lake boundary comprising the inflow section of four lakes in the Ringwood area

Lake	Predicted length of inflow boundary	Actual length at high water	Date of high water	Actual length at low water	Date of low water
Spinnaker Lake	950 metres	750 metres	5.4.79	700 metres	6.12.78
Ellingham Lake	1000 metres	900 metres	5.4.79	750 metres	6.12.78
Cell 1, Ibsley Airfield	444 metres	750 metres	5.4.79	778 metres	4.9.79
Ivy Lane Lake	1275 metres	900 metres	28.5.79	350 metres	11.10.78

The actual Kippungslinien for each of the four lakes studied, had been displaced varying amounts from the predicted Kippungslinien. The amount of change is measured by the height difference (represented as d) between the actual and predicted lake levels, as indicated in Table 12.1.

In each of the lakes, except cell 1 on Ibsley Airfield, the actual Kippungslinien had been displaced upstream relative to the predicted Kippungslinien. This means that the actual lake levels were higher than the predicted levels. Generally speaking, the greater the distance between the actual and predicted Kippungslinien, the greater difference there was between the actual and predicted lake levels.

Under certain circumstances, such as occurs at Ivy Lane Lake at low water, the Kippungslinie was displaced so far upstream that it almost coincided with the upstream bank (fig. 12.9a). Since the Kippungslinie divides the lake into two sections - an inflow (or upstream) section, incorporating the whole lake boundary where effluent seepage is predominant, and an outflow (or downstream) section, incorporating the area where influent seepage is predominant - displacement of the actual Kippungslinie from its predicted position produces an increase or decrease in the area through which groundwater inflow takes place. The change in the length of the inflow area will be proportional to the distance between the actual and predicted Kippungslinien.

In table 12.2, the predicted and actual lengths of groundwater inflow area for the four lakes studied in the Ringwood area have been calculated. Apart from cell 1 on Ibsley Airfield, the results indicate that there had been a reduction in the inflow areas of these lakes. The greatest reduction was in the Ivy Lane Lake, particularly at low-water. Inflow was then restricted to only a small area along the northern shoreline, the remainder of the lake boundary forming part of the outflow area (figs. 12.9a and b).

In order to consider those changes outlined above more closely, three lakes will be studied in greater detail. These are the Ivy Lane Lake, cell 1 on Ibsley Airfield and Stoneacres Lake near Stanton Harcourt.

A. The hydrology of Ivy Lane Lake, showing the influence of lake sealing

The groundwater/lake conditions on the 11.10.78 and 28.5.79 are shown in figs. 12.9 a and b respectively. As these diagrams show, there were substantial seasonal variations in the nature of the groundwater/ lake interactions.

On the 11.10.78, the lake was at its lowest recorded level (20.43 metres O.D.). In comparison, the predicted level was 20.00 metres O.D. The actual Kippungslinie had been displaced towards the northern shore of the lake, greatly reducing the inflow area. Therefore, groundwater flow to the east and to the south of the lake was diverted around the lake. Since there appeared to be very little flow across the eastern or southern boundary of the lake, they may be considered to be impermeable boundaries. Most of the outflow from Ivy Lane Lake appears to have taken place across the western boundary.

Figure 12.9b shows the high-water situation on 2[illegible].5.79. The direction of groundwater flow around the lake shows that the groundwater/ lake interaction had changed considerably from that in fig. 12.9a. The main feature to note is the extension of the inflow zone; so that groundwater inflow occurred along the whole of the eastern boundary. The indication is that in winter and spring, most of the groundwater flowing west passes through the lake, rather than being diverted around it.

The expansion and contraction of the groundwater contributing area appears to be seasonal. In the summer, the lake level falls more slowly than the surrounding groundwater-table (fig. 12.10). This may be accentuated where natural outflow from the lake is restricted, such as by the effects of lake sealing. Therefore, as groundwater levels fall relative to the lake level, the inflow area, i.e. that part where there is a positive hydraulic gradient between the water-table and the lake, will contract.

It is possible to imagine extreme cases, in which the height of the lake surface is greater than the height of the water-table, both upstream and downstream. In such circumstances, all throughflow will cease, since the hydrostatic pressure in the lake will be higher than that in the surrounding aquifer.

A completely 'open' lake increases the flow of groundwater through that part of the aquifer by acting as a preferred routeway. This, and the fact that the lake surface is horizontal, causes the upstream groundwater levels to fall and the downstream levels to rise in relation to their pre-extraction levels. This is shown diagramatically for Ivy Lane Lake in fig. 12.11. Taking the pre-extraction hydraulic gradient in the area now occupied by Ivy Lane Lake to be 0.0011 and the groundwater level at borehole R/5 to be 20.80 metres O.D. (the actual level on 11.10.78), the pre-extraction height of the water-table at borehole R/32, downstream of the lake, would have been 19.45 metres O.D. (fig. 12.11a). On the formation of Ivy Lane Lake, the surface of the lake would rise to the height of the pre-extraction groundwater-table at the mid-point of the lake, i.e. 20.0 metres O.D. (fig. 12.11b). Upstream of the lake, this would produce a zone of drawdown, i.e. the water-table would be lower than the pre-extraction level. Similarly, downstream of the lake, there would be an area in which the water-table would be higher than the pre-extraction level.

The actual situation which was observed on 11.10.78 is shown in fig. 12.11c. The actual lake level was 0.[illegible] metres higher than the predicted level. This resulted in a reduction of the upstream zone of drawdown. Downstream of the lake, however, there was a marked fall in groundwater levels (fig. 12.12). The groundwater level at observation borehole R/32, for instance, was almost 2 metres below the lake level. It is proposed that the differences between the actual and predicted water levels were due to sealing of the downstream boundary of the lake. This would restrict the natural flow of water through the lake and produce an increase in lake level. As the lake level rises the water-table upstream is raised also, while downstream, because outflow from the lake is restricted, the water-table is lowered. It is suggested therefore, that sealing of the lake boundary is a dominant factor in groundwater-lake interactions and the most probable reason why the actual surface levels of not only Ivy Lane Lake, but also Spinnaker Lake and Ellingham Lake are higher than their predicted levels (see table 12.1).

B. The hydrology of cell 1 on Ibsley Airfield

The actual level of cell 1 on Ibsley Airfield, in contrast to the situation described for the lakes elsewhere in the Ringwood area, was slightly lower than the predicted level both at high and low-water. This suggests that sealing was not a major influence in this particular lake. However, the fact that the actual Kippungslinien had been shifted so far downstream of the predicted line is not a true reflection of the difference in height between the actual and predicted lake levels (d). It is also a function of the low pre-extraction hydraulic gradient across the airfield. For the same values of d, the actual Kippungslinie will be displaced further upstream or downstream from the predicted line in an area of low hydraulic gradient than in an area of high hydraulic gradient (fig. 12.13). This diagram also illustrates another important point. Other things being equal, lake levels will tend to be highest in areas of lowest hydraulic gradient.

The restriction of groundwater inflow into cell 1 may explain why the actual lake levels were lower than the predicted levels. Between cells 1 and 2 (the lake to the north) is a narrow bund which was constructed from overburden material. Being of lower permeability and more compacted than the gravels, seepage through the bund will be reduced relative to a normal gravel bank. This will result in lower than predicted lake levels, since it has previously been assumed that groundwater inflow takes place equally around the whole upstream boundary.

C. The hydrology of Stoneacres Lake, Stanton Harcourt, showing the influence of surface outlets

A surface outlet from a lake may considerably modify the relationship between actual and predicted lake levels. Figure 12.14 shows the displacement of the actual Kippungslinie from the predicted Kippungslinie, on three particular dates, at Stoneacres Lake. Considering the year 1979, for which complete lake level and groundwater level records were available, it was found that for the first six months of the year, actual lake levels were lower than predicted. In the second half of 1979, actual levels were higher than predicted. The most probable reasons for this relationship are the combined effects of the two surface outlets (A and B on fig. 12.14) and sealing of the downstream bank.

During the first half of 1979, predicted lake levels varied between 63.26 metres O.D. and 62.79 metres O.D. Actual lake levels over the same period varied between 62.79 metres O.D. and 62.65 metres O.D. The actual Kippungslinien were therefore displaced downstream. The heights of the two surface outlets A and B were 62.35 metres O.D. and 62.66 metres O.D. respectively. Outlet A was a 31 cm diameter concrete pipe which drained into a ditch at the north-eastern corner of the lake. Outlet B was a 15 cm diameter pipe which discharged into Linch Hill Brook at point C (fig. 12.14).

The predicted winter lake levels were greater than the heights of both surface outlets. Under such circumstances, outflow through both outlets should occur. From the results of a current-metering exercise carried out on 18.3.78, the discharge through outlet A was approximately 225 m^3/d. The lake level on this date was below the height of outlet B.

Surface discharge probably accounts for a large percentage of the total outflow from Stoneacres Lake. Sealing of the downstream banks will also increase the outflow through the surface outlets, by raising lake levels relative to the height of the outlets. The total flow of water through the lake, however, would tend to remain the same. The effect of surface outlets, therefore, is to lower lake levels, relative to the predicted levels. This would occur especially in winter, when the predicted lake levels are above the height of the surface outlets. The lowering can occur to such a degree, that on certain days the actual Kippungslinien plot downstream of the lake (fig. 12.14). On these occasions, the lake surface was lower than the groundwater-table both upstream and downstream of the lake. All outflow across the lake boundary must therefore cease, so that outflow only occurs through the surface outlets.

Between August and November 1979, the predicted level of Stoneacres Lake varied between 62.29 metres O.D. and 62.07 metres O.D. Actual lake levels over the same period varied between 62.55 metres O.D. and 62.32 metres O.D. The actual Kippungslinien were therefore displaced upstream of the predicted Kippungslinien. The predicted lake levels are below the height of both surface outlets. Conversely, the actual lake levels are, in general, above the height of the lower of the two outlets, i.e. outlet A. This suggests that sealing of the downstream lake boundary was an important factor. Sealing of the lake boundary causes lake levels to rise by reducing influent seepage. This can occur to such an extent that on some dates the actual Kippungslinien plot upstream of the lake. Then, not only was the actual lake level higher than the predicted level, but it was also higher than the upstream water-table. Under such circumstances all groundwater inflow must cease.

To summarise, when predicted lake levels are higher than the surface outlets, the outlets are the dominating factor. This produces lower actual lake levels. When the predicted lake levels are lower than the height of the surface outlets, the effects of lake sealing predominate. This produces higher actual lake levels.

12.2.3 The causes of lake sealing

The sealing of the lake boundary occurs as the result of diverse phenomena. Perhaps the most important mechanism is the clogging action of redeposited fine-grained material. This will most commonly be silts and clays, although fine-ground material produced by chemical or even biological processes may also be involved. Two other mechanisms which may be important, depending on the hydrological environment, are biological clogging and chemical clogging.

Without an intensive study, it is very difficult to decide which of these three possible sealing mechanisms is most important, or to predict the rate at which these processes occur. Obviously, in different lakes there will be variations in the mechanisms and rates of sealing. We have seen in section 12.2.2, how sealing of the downstream boundary of a gravel lake is an important factor in controlling lake and groundwater levels. Even without knowing the exact process of its formation, it appears no less necessary to discuss the most probable mechanisms responsible for this sealing.

It is necessary to define exactly what is meant by the term sealing. In the context of this study, it is the process by which the inter-granular spaces of the porous material forming the boundary of a lake become clogged. This reduces the effective permeability of the material and reduces the rate of seepage through the banks.

The three types of sealing mechanism will now be discussed.

A. Clogging by suspended material

Particulate material in suspension is probably the dominant factor in reducing influent seepage from lakes. Depending upon the size of the particles, the nature of the bank formation, and the flow velocity, the sediment may either be trapped on the surface, so creating an impervious mud film, or may be carried into the gravels, blocking the pore spaces within a few centimetres of the lake/gravel boundary.

The rate of outflow across the lake boundary is a function of the resistance or straining efficiency of the boundary surface. The straining efficiency will increase with time. During the first stages in the flooding of an excavation, a large proportion of any suspended material will pass into the aquifer. The larger particles, however, will be intercepted at the surface, so reducing the size of the pore spaces. Eventually, most of the suspended material will be intercepted and a thin film of sediment will develop over the lake boundary.

In coarse gravel aquifers, the initial straining efficiency of the boundary material will be relatively low. Suspended material will be carried into the aquifer. Sedimentation will eventually take place, but at a much greater depth. It may occur, for instance, when the flow encounters a layer of material having a lower porosity, causing clogging at the interface of the two layers.

Ives (1964) describes a mechanism for clogging at depth by suspended sediment, which he suggests takes place during the artificial recharge of aquifers. The attractive molecular force between suspended particles and grains in the aquifer material, may deposit suspended particles as small as one hundredth of the pore size. Deposition of these particles will initially increase the straining efficiency of the material by enlarging the surface area of the granular material. At the same time, larger particles bridge over the gaps and trap other particles. As deposition continues, the flow paths between the grains are straightened and reduced in size, increasing the flow velocity. Particles are then swept through the pores, without adhering to other grains. Thus, the straining efficiency is reduced and deposition is transferred further down into the aquifer material.

The rate at which the lake boundary becomes sealed with suspended sediment, depends to a great extent upon the quality, and particularly the particulate matter content, of the lake water. The origin of the majority of the suspended material which causes clogging will be the excavation itself. During the extraction phase, a large proportion of the fine fraction of the sand and gravel will be left behind in the pit. This will be the case particularly where wet-digging takes place. Silt and clay will then tend to be washed out of the gravel as it is excavated, being deposited at the bottom of the pit.

The overburden, which is removed prior to excavation, is a further source of fine sediment. The overburden is often used in restoration work (i.e. bank formation) prior to the flooding of gravel pits. Heiple (1959) investigating the straining efficiency of sedimentary material, showed that the straining efficiency increased as the particle size decreased. So not only does the use of overburden increase the supply of fine sediment, but it will also increase the rate of sealing, where it is used in bank restoration.

For clogging to take place, the particulate matter must be suspended in the lake water. In relatively shallow gravel lakes, windy conditions will increase turbidity by the resuspension of fine material. There will be a tendency for this material to be deposited on the downstream boundary, by the natural throughflow of the lake. Similarly, the upstream boundary will be kept free from clogging by the flow of water into the lake, which will wash-out any deposited particulate material.

From the above discussion, the rate of sealing by suspended material, seems to depend upon:

(a) the particle-size of the aquifer material,
(b) the concentration of particulate matter in the lake water,
(c) the rate of through-flow, and
(d) the age of the lake or the straining efficiency.

For a given grain-size of aquifer material and flow rate, the rate of sealing will be proportional to the straining efficiency and the concentration of particulate matter. The straining efficiency depends upon the quantity and form of the pores in the aquifer material. At the surface it will increase throughout time, whereas at depth the straining efficiency will first increase and then decrease.

The difference between the predicted and actual lake levels (table 12.1) gives an indication of the degree of lake sealing. On this basis, Ivy Lane Lake appears to be most seriously affected. This lake is used as a silt lagoon. Water from the adjacent washing plant, which contains considerable quantities of silt and clay. is pumped directly into the lake. Much of the coarser material quickly settles out around the discharge point, but the finer material will be carried out into the lake to be deposited more slowly. It therefore seems only a matter of time before Ivy Lane Lake becomes so clogged with sediment, that all through-flow of water ceases, and the lake will become isolated from the surrounding aquifer.

B. Biological clogging

Algae are the most important and conspicuous form of biological clogging agent. In most lakes, algae will grow, particularly in the summer. Extensive beds of rooted aquatic vegetation also colonise many of the more established gravel lakes. Bacterial flora may also grow, both on the surface of the strata in contact with the lake water and also within both the lake and aquifer deposits.

Algae occur in the lakes in two main forms, (a) those present in the lake water, which may be treated as a form of suspended solid, and (b) benthic algae, which grow over the surface of the lake boundary.

An extensive growth of algae over the lake boundary can cause clogging of the gravels in its own right, but the algae may accentuate this action by trapping or adsorbing particulate or dissolved organic matter. Algal mats formed in this way can be found around many gravel pits and lakes. Algae in dewatered pits grow around groundwater springs. In places it forms a thick mat which completely covers the gravel surface and is closely associated with orange-coloured particles of iron precipitates.

The rate at which biological clogging proceeds appears to be relatively fast. Algae soon becomes established in dewatered pits, and they quickly colonise the shallower parts of most gravel lakes. In summer, when lake levels are lower, the algal mats become exposed and dry out to form hard crusts around the lake shore-line. Algal growth appears to be independent of lake-use. Colonisation takes place equally quickly in undeveloped gravel lakes and those which, for example, are used in fish farming, where the nutrient status may be higher. The latter may, however, have more long-term effects on the rate of biological activity.

C. Chemical clogging

Chemical reactions which take place in the lake waters can lead to clogging of the lake boundary. Blair (1970) lists three types of reaction which may produce clogging in gravel lakes:

(1) calcium carbonate precipitation, as a result of changes in the calcium carbonate saturation when carbon dioxide comes out of solution,

(2) precipitation of insoluble ferric compounds, as a result of the mixing of reducing and oxidising waters, and

(3) the swelling and dispersion of semi-colloidal particles of clay under the effect of ion exchanging reactions, which may occur when the lake water contains large amounts of organic matter or exchanging ions (eg. Na^+ and H^+).

These reactions take place either through the interaction of dissolved chemicals in the lake water with those in the gravels, or by the mixing of lake water with groundwater which has different chemical properties. Since sands and gravels tend to be chemically unreactive, they will be largely unaffected by chemical reactions with the lake water. On the other hand, where the gravel banks are covered with overburden, chemical reactions may take place with any clay minerals contained (see point 3 above).

The greatest amount of chemical clogging will result from the chemical reactions which take place upon the mixing of groundwater with lake water, in particular the precipitation of insoluble ferric compounds.

Iron deposits are commonly seen in dewatered pits, around seepage points where groundwater containing dissolved iron comes into contact with the air (see chapter 11). Iron, present in the dissolved form as ferrous ions, chemically combines with oxygen in the air until all of the iron is precipitated. This is generally in the form of ferric hydroxide ($Fe(OH)_3$) - a mixture of amorphous material and goethite, with small amounts of haematite (Langmuir, 1969). As the ferric iron is precipitated, it forms a thin layer of soft, gel-like floc, which coats the individual grains of sand and gravel. When the excavations are flooded, these accumulations may remain and facilitate clogging of the gravels.

Iron in lake waters occurs in the reduced ferrous state and in the oxidised ferric state. Which of these predominate in any particular lake largely depends upon the chemical characteristics of the lake water. In acid to neutral water, low in dissolved oxygen, iron occurs in the soluble ferrous state. These conditions generally exist in the hypolimnion of stratified, eutrophic lakes (Cole, 1979). At pH 7.5 to 7.7, a threshold is reached where ferric iron, in the form of $Fe(OH)_3$ is precipitated. This change from the reduced to the oxidised state also takes place with the introduction of oxygen. In well-oxygenated lakes therefore, ferric hydroxide is likely to be the iron compound most constantly present. Various organic substances stabilise ferric hydroxide, so that a considerable part of the ferric hydroxide found in lake water is absorbed on plankton and on dead sestonic particles which will gradually sink to the bottom.

The inflow and mixing of groundwater which, in the normal pH range found in gravels (pH 5 to 7) contains iron chiefly in the dissolved ferrous state, with well-oxygenated lake water, facilitates those chemical reactions which cause a rapid conversion of the ferrous iron to the ferric state and precipitation of ferric hydroxide. Being insoluble in lake water, the particulate material will behave in the same way as suspended sediment and may therefore lead to clogging and sealing of the downstream lake boundary. Similar phenomena have been observed in drainage ditches and streams which receive discharge from dewatered gravel pits. Orange-coloured flocculations of precipitated ferric hydroxide accumulate on the sides and bottom of the channels.

The rate at which chemical clogging takes place is obviously difficult to determine. However, from evidence derived from dewatered pits, the recipitation of iron hydroxides from groundwater seepage is visible within days after the appearance of a spring. Large accumulations rapidly form to produce quite significant deposits. In the temporary extension to Wadham-Brasenose Pit, which was described in chapter 11, distinct signs of iron hydroxide accumulation became apparent around seepage points after about two weeks. The rate at which the iron precipitates accumulate, will be proportional to the concentration of available iron in the groundwater.

No direct evidence for the occurrence of any other form of chemical clogging has been observed in the excavations studied. Suitable conditions for the precipitation of calcium carbonate may exist in the Stanton Harcourt area, where the calcium content of groundwater is high (100-300 mg/l). Similar chemical reactions have taken place in the past. In some borehole samples from Stanton Harcourt, grains of sand and gravel, cemented by calcite, have been found. Chemical evidence from cell 3/6 on Ibsley Airfield, which is discussed in section 12.4, also suggests that decalcification of the lake water takes place, with subsequent precipitation of calcium carbonate. The rate at which these reactions occur is difficult to determine, although it is probably seasonal, being dependent on the photosynthetic activity of algae and macrophytes.

12.2.4 The influence of lake shape and size on lake/groundwater interactions

The interactions between gravel lake and groundwater are influenced by the surface shape and size of the lake.

Figure 12.15a shows the influence of lake orientation on groundwater flow. The amount of disturbance of the original equipotential lines is an indication of the size of the effect of gravel lakes on groundwater flow. An excavation elongated with its long axis perpendicular to the groundwater flow direction has relatively little effect on flow conditions. On the other hand, a lake elongated parallel to the general flow direction, has a far greater disturbance on the flow pattern.

Cross-sections through the lakes, A^1-A^2 and B^1-B^2 in fig. 12.15b, illustrate the influence of lake shape on groundwater levels. Given completely open conditions (i.e. no sealing of the lake boundary), the lowering of levels on the upstream side and the raising of levels on the downstream side relative to pre-extraction levels, is greater around lakes orientated parallel to the flow direction than around lakes orientated perpendicular to the flow direction. It also shows that the radius of the upstream drawdown zone (R) is proportional to the length of the lake axis which is parallel to the direction of groundwater flow.

Sealing of the downstream lake boundary leads to an increase in lake level. Where the topography and slope of the land surface reflects that of the original groundwater surface, excessive sealing may lead to overtopping of the downstream banks and consequent flooding of the land down-gradient of the lake (fig. 12.16). The hypothetical situations in fig. 12.15b show that the difference in height between the original water-table at the downstream boundary of the lake and the predicted lake level is greatest for lakes which have their longest axis parallel to the groundwater flow direction. Over-topping of the downstream banks therefore, appears to be more of a risk for large lakes orientated parallel to the flow direction. The effects of sealing and the risk of flooding can therefore be minimised by excavating large gravel pits as a series of small individual cells (fig. 12.17 a and b).

In the Ringwood area, the development of large lakes trending NE/SW may lead to undesirably high lake levels at their south-western margins. If one considers the development of a single lake on Ibsley Airfield, it can be calculated that the highest lake levels would be approximately 22 metres O.D. This would mean that the water-level at the south-western margin would be up to 1.25 metres above the present water-table. With sealing of the downstream banks, the actual lake level could be increased by 0.5 metres or more above the predicted level. This indicates that over-topping of the downstream banks and flooding of the area around Ellingham Cross could be a problem (ground levels around Ellingham Cross are shown in fig. 12.18). The risk of overtopping would obviously be greater in winter or after particularly heavy storms, when input to the lake would be at a maximum.

In practice, the flooding problem may be overcome by constructing strong embankments around the margins of the lake, or by incorporating a surface outlet or spillway to control the final lake level. Alternatively, the problem may be reduced by constructing smaller lakes, in which case the east-west water level would decrease in a series of steps (fig. 12.17b). This is, in fact, what is now taking place on Ibsley Airfield. At present, eight cells have been excavated in the north-eastern section of the airfield, some of which have since been incorporated to form larger lakes (fig. 12.18).

12.3 Water movement through gravel lakes

Water flows into and through gravel lakes as long as there is a hydraulic gradient between the upstream water-table, the lake surface and the downstream water-table. Assuming that the lake level is constant, inflow to the lake must be balanced by the outflow. Therefore, disregarding evaporation, a measure of lake throughflow can be made by an estimation of lake inflow using the Dupuit equation. This is of the form:

$$Q = -\frac{K}{2x}\left(h^2 - ho^2\right) \qquad (12.1)$$

where Q is the discharge through the lake boundary per unit width, K is the hydraulic conductivity, h and ho are the heights of the water-table above a datum at two known points, and x is the horizontal distance separating them.

Certain assumptions, which are implicit in equation 12.1, limit the applicability of the results (Dupuit, 1863). These assumptions are:

a) that the hydraulic gradient is equal to the slope of the water-table and does not vary with depth.

b) that groundwater flow is horizontal and uniform everywhere in a vertical section.

In effect, these assumptions neglect the vertical components of groundwater flow and, as such, should be treated with caution. In the drawdown zone upstream of a gravel lake, the hydraulic gradient is steep and vertical components of flow will occur. The Dupuit assumptions are therefore poor approximations to the actual flow conditions, except at the time of low water levels when the drawdown effect and the slope of

Table 12.3 The predicted and actual inflow into four lakes in the Ringwood area at low and high lake levels

Lake	Low water				Proportion of actual to predicted inflow	Water Renewal Time (days)	Area of lake (ha)
	Actual inflow		Predicted inflow				
	1	2	1	2			
Spinnaker Lake	3.36	2,353	8.5	8,114	29%	425	22
Ellingham Lake	15.63	11,726	21.17	21,168	55%	42	12
Cell 1, Ibsley Airfield	4.85	3,773	3.95	1,754	215%	42	5
Ivy Lane Lake	1,87	646	3.99	5,090	13%	1,177	17

Lake	High water				Proportion of actual to predicted inflow	Water Renewal Time (days)	Area of lake (ha)
	Actual inflow		Predicted inflow				
	1	2	1	2			
Spinnaker Lake	4.40	3,307	8.5	8,047	41%	302	22
Ellingham Lake	20.91	18,819	23.22	23,218	81%	27	12
Cell 1, Ibsley Airfield	4.98	3,732	4.10	1,821	205%	57	5
Ivy Lane Lake	3.30	2,970	5.81	7,413	40%	288	17

1. Inflow per unit length of inflow boundary ($m^3/m/day$)
2. Inflow through the total length of inflow boundary (m^3/day)

the free surface is reduced. Under these circumstances, the calculations based on the Dupuit equation compare favourably with those based on more rigorous methods (Freeze and Cherry, 1979).

Table 12.3 shows the estimated values of actual inflow, for the four lakes studied in the Ringwood area, using equation 12.1. Values are given for inflow per unit length of bank and for the total inflow. The latter were computed as the product of the inflow per unit length of bank and the total length of inflow bank upstream of the Kippungslinie (see Table 12.2). The total inflow results compare favourably with those given by Wrobel (1980) for two gravel lakes (area of both approximately 14 ha) in the Munich area of West Germany, which averaged 3456 m^3/day and 4320 m^3/day respectively.

Original throughflow rates (i.e. before sealing, etc.) can only be estimated roughly. The actual Kippungslinien of all four lakes have been displaced varying distances upstream or downstream of the predicted Kippungslinien. The area therefore, from which groundwater originally entered the lakes is very different from that observed now. The original inflow values were calculated however, using estimates of the predicted lake level and the predicted length of inflow section, which are shown in Tables 12.1 and 12.2 respectively. These results are also shown in Table 12.3.

Inflow rates have been estimated for both low and high water conditions. The latter may be less good approximations to the actual rates, because of the increase in importance of vertical components of flow. The general trend is that inflow rates at high water are greater than those at low water. Since the same value of K (i.e. 77.05 m/d, which is the average value determined by point dilution in the Ringwood area) has been used throughout, the difference is a function of the increased hydraulic gradient in winter.

For Spinnaker Lake, Ellingham Lake and Ivy Lane Lake, the actual inflow rates were less than the predicted rate. This is due to the decreased hydraulic gradient between the upstream groundwater levels and the lake surface. As a result of the sealing of the downstream lake boundary, lake levels have risen relative to the surrounding water-table. Also, the Kippungslinien of all three lakes have been displaced upstream of the predicted line, so reducing the effective length of the inflow boundary.

For cell 1 on Ibsley Airfield, the actual total inflow rate was greater than the predicted rate, although the values of inflow per unit length of boundary are very similar. This seems to suggest that throughflow is uninhibited, and that little downstream sealing of the lake has occurred. The actual total inflow is greater than predicted, because the Kippungslinie has been displaced downstream (fig. 12.8), so increasing the length of the inflow boundary (table 12.2). However, it is possible that the actual inflow rate is slightly inaccurate. It was indicated in section 12.2.2, that the northern boundary of cell 1 is constructed from overburden. This will be of lower permeability than the surrounding gravels. An average value for the permeability of the gravels has been used in calculating the inflow rates, which will therefore overestimate the inflow rate into cell 1.

The differences between the estimates of total inflow at low and high-water are related to the position of the Kippungslinien. At low-water the Kippungslinien were displaced further from the theoretical Kippungslinien. In the case of cell 1, because the displacement was downstream, the total inflow at low-water was greater than at high-water. For the other three lakes, because the displacement was upstream, the total inflow at low-water was less than at high-water.

Considering Spinnaker Lake, Ellingham Lake, and Ivy Lane Lake only, it is possible to relate the estimates of total inflow to the history of the excavations. The low rate of inflow into Ivy Lane Lake is related to the accentuated degree of sealing, which was described in sections 12.2.2 and 12.2.3. Gravel extraction started on the sites of the Spinnaker and Ivy Lane Lakes in 1952 and 1956 respectively. Neither excavations were dewatered. The excavation of Ellingham Lake began in 1965, and this site was dewatered. It seems, therefore, that the inflow rate is proportional to the age of the lake and is related to the methods used to extract the gravel. Digging the gravel from below standing-water, as already suggested in section 12.2.3, washes-out much of the fine material and accentuates the sealing process.

It is concluded that the throughflow rates of the Spinnaker, Ivy Lane and Ellingham Lakes are proportional to the degree of sealing, which in turn is a function of the method of extraction and the age of the excavation. The proportion of the actual to predicted inflow is expressed as a percentage in Table 12.3. This value can be used as a measure of lake sealing, i.e. the lowest values relate to the greatest amount of clogging.

Estimates of the water-renewal time for each lake are shown in Table 12.3. The water-renewal time is calculated as the ratio of lake volume to the actual lake throughflow rate. Its reciprocal is the lake flushing rate. The values calculated for the Ringwood lakes compare favourably with those given by Wrobel (1980) for German gravel lakes, i.e. 97 to 217 days.

The flushing rate is important in determining the critical nutrient load which would produce a change from oligotrophy to eutrophy (Vollenweider, 1976). A study of the phosphorus supply of some Nova Scotian lakes led to the conclusion that lakes with shorter water-renewal times were less vulnerable to pollution (Kerekes, 1975). It seems, therefore, that lakes such as Ivy Lane Lake which are heavily sealed and have large water-renewal times (i.e. low flushing rates) are most susceptible to pollution. This could then be transmitted into the aquifer and in some circumstances may lead to groundwater quality problems at sources downstream of the lake. The relationships between lake water quality and groundwater quality is discussed in the proceeding section.

12.4 Lake-water quality and its effect on groundwater quality

This section deals with the thermal, chemical and biological properties of gravel lakes, and their effect on groundwater quality, particularly downstream of the lakes. The first part deals with the relationship between lake temperature and groundwater temperature, and its use as an indication of the degree of lake sealing. The second part discusses the chemical and biological properties of gravel lakes, their influence on groundwater quality in the downstream area, and the possible causes and effects of lake eutrophication. Examples will be drawn from the Ringwood area.

12.4.1 Water temperature

Between March 1978 and May 1980, the temperatures of the lakes in the Ringwood area varied between $0^{\circ}C$ and $>20^{\circ}C$, whereas groundwater temperatures only ranged between $3^{\circ}C$ and $17^{\circ}C$. Groundwater temperature is dependant to a certain extent on the position of the borehole in the aquifer relative to the lakes. The temperature of groundwater downstream of the lakes fluctuates over a wider range than the upstream groundwater temperatures. For example, the range in temperature of groundwater upstream of Ivy Lane Lake, Spinnaker Lake, and cell 3/6 on Ibsley Airfield was 3.75°C, 3.95°C, and 5.6°C, respectively. The equivalent downstream range in groundwater temperatures was 6.35°C, 12.3°C, and 13.7°C. This is due to mixing with lake-derived water which, being exposed to the atmosphere, is subject to wider temperature fluctuations than groundwater.

A cross-section through the aquifer upstream and downstream of cell 3/6 on Ibsley Airfield (fig. 12.19) shows clearly the influence of the lake on groundwater temperatures on three selected dates. In winter, the groundwater immediately downstream of the lake is cooled whereas in summer it is warmed, relative to groundwater elsewhere in the aquifer. The greatest influence is restricted to a fairly narrow zone of the aquifer around borehole R/52. This particular borehole is positioned between cell 3/6 and cell 8, which, at the time of the observations, was being dewatered. A considerable quantity of water was observed flowing into cell 8 through the intervening bund. The temperature evidence suggests that most of this water was outflow from cell 3/6.

The effect of cell 3/6 on the annual range in groundwater temperature is shown in fig. 12.20. During the observation period 12.7.79 to 21.5.80, the temperature of groundwater upstream of cell 3/6 varied between 7.7°C and 12.5°C. A short distance downstream of the lake (at borehole R/52) the groundwater temperature varied between 3.0°C and 16.7°C. A further 600 to 1000 metres down-gradient of borehole R/52, the temperature range of the groundwater decreased towards the natural (i.e. upstream) groundwater range. This trend was also noted around the other lakes studied.

The curves of maximum and minimum groundwater temperature in fig. 12.20 show that in winter, lake water was warmed as it flowed down-gradient through the aquifer, whereas in summer it was cooled, by mixing with groundwater. The curves are asymmetric, which shows that the water was warmed faster in winter than it was cooled in summer. The effects of the lake on groundwater temperatures therefore, were felt over a wider area in summer than in winter.

The temperature range of groundwater downstream of gravel lakes provides an independent check on the amount of lake sealing. Immediately downstream of a lake where outflow is not restricted by sealing, the groundwater temperature should correspond closely to the temperature of the lake water. Conversely, downstream of those lakes which are heavily sealed, groundwater temperatures should correspond more to the overall regional

Table 12.4 Observed groundwater temperatures at selected sites in the Ringwood area (March 1978 to May 1980)

Lake	Observation well	Groundwater Temperature		
		Max. (°C)	Min. (°C)	Range (°C)
Cell 3/6	R/6 (upstream)	12.6	7.0	5.6
Ibsley Airfield	R/52 (downstream)	16.7	3.0	13.7
Spinnaker	R/23 (upstream)	13.15	9.2	3.95
Lake	R/21 (downstream)	18.3	6.0	12.3
Ivy Lane	R/34 (upstream)	12.25	8.5	3.75
Lake	R/32 (downstream)	13.35	7.0	6.35

Table 12.5 Water quality of cell 3/6, Ibsley Airfield, and groundwater upstream and downstream

Chemical constituent	Upstream Groundwater	Cell 3/6 Ibsley Airfield	Downstream groundwater	
	R/6		R/52	R/22
Ca^{2+} (mg/l)	40.45	26.25	35.67	40.33
Na^{+} (mg/l)	10.44	10.19	10.33	12.17
K^{+} (mg/l)	4.72	4.00	4.65	3.83
Mg^{2+} (mg/l)	2.16	2.23	2.93	2.37
Fe (total) (mg/l)	1.55	0.2	13.00	2.2
Total alkalinity (mg/l)	49.67	70.00	20.33	73.00
SO_4^{2-} (mg/l)	35.00	20.00	37.50	24.0
PO_4 (total) (mg/l)[1]	1.00 - 0.1	0.6 - 0.3	0.8 - 0.1	4.6(?) - 0.1
pH	6.15	8.36	6.45	6.23
Conductivity (µmhos)	277	198	233	250

1. Mean concentrations are shown for all constituents except PO_4, for which the range in observed values is given.

temperature, as expressed by the upstream groundwater temperature.

In Table 12.4, the maximum and minimum groundwater temperatures, observed from March 1978 to May 1980, at selected observation wells upstream and downstream of three lakes in the Ringwood area are shown. The temperature of groundwater upstream of the lakes was similar in all three cases, and probably reflects the natural temperature range of groundwater. The downstream groundwater temperatures however, varied considerably between the lakes. This is an indication of the amount of interchange of water between lake and aquifer. The greatest temperature range was observed at borehole R/52, downstream of cell 3/6, indicating that there is a substantial exchange of water between the lake and aquifer and, therefore, that very little sealing of the lake boundary has occurred. The smallest temperature range was observed at borehole R/32, downstream of Ivy Lane Lake. This range was similar to the regional groundwater temperature range, indicating that very little outflow from the lake occurred. This is probably the result of excessive sealing. The temperature range of groundwater at borehole R/23, downstream of Spinnaker Lake, indicates that there was a substantial exchange of water between the lake and the aquifer. These results confirm the suggestions made in sections 12.2.2 and 12.3, i.e. Ivy Lane Lake was quite effectively sealed from the aquifer, whereas Spinnaker Lake, although showing some indications of sealing, was still in hydraulic continuity with the surrounding aquifer. Cell 3/6, being the youngest of the lakes studied, showed no signs of sealing.

12.4.2 Chemical and biological characteristics

The chemical and biological properties of lake water may differ significantly from the groundwater entering the lake. These changes occur during the water's storage within the lake. This section discusses the factors which affect the water quality transformation process, and the influence on groundwater quality downstream of the lakes. The problems of eutrophication will also be discussed.

As an example of the changes in water quality brought about by the passage of groundwater through a gravel lake, Table 12.5 lists the mean values of selected water quality parameters for cell 3/6 on Ibsley Airfield and for groundwater upstream and downstream of the lake. Eight complete sets of samples were collected for water quality analysis between April 1978 and April 1980. The concentration of many of the chemical constituents analysed, is modified during flow through cell 3/6. This is also reflected in changes in the pH and electrical conductivity of the lake water relative to the groundwater upstream.

The increase in pH of the lake water was probably linked to the decrease in the Ca^{2+} content. Groundwater, containing relatively large amounts of $Ca(HCO_3)_2$, is in equilibrium with CO_2 (Cole, 1979). Removal of the CO_2 in the lake, by the process of photosynthesis, disrupts this equilibrium and leads to the precipitation of $CaCO_3$ and the formation of hydroxyl ions. All of this leads to an increase in ph. This can be shown by the following reaction:

$$Ca(HCO_3)_2 \rightleftharpoons \underset{\text{precipitated}}{CaCO_3} + H_2O + \underset{\text{assimilated}}{CO_2} \qquad (12.2)$$

The precipitation of $CaCO_3$ may lead to the formation of calcareous incrustations. This may be an additional factor in the sealing of some lakes (see section 12.2.3), particularly those in the Stanton Harcourt area where the groundwater is highly calcareous.

Lake eutrophication and the widespread proliferation of algae are brought about by excessive concentrations of phosphorus and nitrogen in lake water (eg. Jones, 1972). Various attempts have been made to quantify this relationship. According to Hamm (1975), the critical concentration of total phosphate for the commencement of excessive algae production is 0.02 mg/l. In cell 3/6, and the other lakes in the Ringwood area, and in groundwater, phosphate contents considerably higher than this critical level were observed.

Nitrate concentrations were not measured (because of the lack of suitable laboratory equipment), but phosphate levels in both the groundwater and lake water were greater than Hamm's critical concentration for algae production. The phosphate results given in Table 12.5 were measured as orthophosphate and are expressed in mg/l P. Although the limit of detectibility was only 0.1 mg/l, groundwater concentrations up to 1.0 mg/l and lake water concentrations up to 0.6 mg/l were recorded.

The phosphate concentrations of groundwater and lake water on Iblsey Airfield appear to be very high, but they are within the limits recorded for the R. Avon at Ringwood. A report on the water quality of river samples taken at Ringwood (grid reference SU 146 056), by the Wessex Water Authority, between 1970 and 1980, indicate a mean phosphate level of 0.53 mg/l. Values ranged between 0.1 mg/l and 4.3 mg/l. These high levels are thought to be due to sewage effluents and/or agricultural runoff (Cox, 1981, personal communication). The high concentrations over Ibsley Airfield are probably derived therefore, from surface waters which are contaminated with agricultural runoff. The airfield itself is in agricultural use, where there are no excavations, and nearby Dockens Water drains the higher heathlands to the east. The phosphates would reach the water-table, and later the gravel lakes, either directly by surface infiltration and leaching of phosphates in the soil or indirectly by seepage from surface water such as Dockens Water.

The phosphorus content of cell 3/6 was found at times to be greater than in the upstream and downstream groundwater. Unless there is an additional contribution of water to the lake, the concentration in the lake should be equal to, or less than, the mean concentration of the inflows (Gibson, 1981, personal communication). If the concentration in cell 3/6 really is greater than the mean inflow concentration, this can either be because:

a) there are peculiar hydrological conditions whereby high phosphorus concentrations are attained at times when the flow through the lake is greatly reduced, but the mean phosphorus concentration is much less than this maximum; or
b) the lake is a net exporter of phosphorus and is out of equilibrium with the groundwater.

Arguments can be put forward to support either case. The maximum upstream and downstream groundwater phosphorus content is greater than the maximum lake content. It is possible that there is a time lag between the groundwater phosphorus content and the lake content. This may be partly due to sealing of the lake boundary, which would reduce the outflow rate and increase the retention time of thelake water. This would then make the water more susceptable to changes in chemistry which take place wholly within the lake.

Stundl (1981) observed a pronounced tendency of eutrophication in flooded excavations which were less than 3 metres deep. This was largely due to frequent resuspension of bottom sediments. It could be suggested, therefore, that the relatively high nutrient loading observed in cell 3/6 (and other gravel lakes) results from the resuspension of colloidal dispersions of clay deposited on the lake bottom. Some of the most eutrophic lakes in East Africa are ones where the bottom sediments get stirred by wind (Perrot, 1981, personal communication).

The high phosphate content of cell 3/6 may also be the result of the decay of organic material by bacteria. It is known that living planktonic organisms, which are common in lakes, excrete organic phosphorus compounds. These soon become colloidal and can account for most of the filterable phosphorus in lake waters (Lean, 1973).

The effect of high nutrient loadings is to promote rapid growth of algae, phytoplankton, and macrophytes. Algae are particularly abundant in gravel lakes. Around the shoreline they form extensive mats which cover the submerged gravel surface. A variety of macrophytes flourish in gravel lakes. On some of the smaller ones, they have become so well established that they form floating mats which completely cover the lake surface, particularly in summer.

Biological activity has important effects on lake chemistry. It has already been shown that algae are responsible for the precipitation of $CaCO_3$ and the increase in pH, but their growth also leads to the consumption of nutrients (particularly Fe, NO_3, SiO_2, and PO_4) within the lake, and the consequent reduction in electrical conductivity of lake water relative to groundwater.

A major feature of cell 3/6 was the reduction in iron content relative to groundwater inflow. This was probably due to the precipitation of insoluble ferric hydroxides in the lakes as a result of the oxidation of ferrous compounds in groundwater (see section 12.2.3) and the increase in pH. According to Cole (1979), at ph 7.5 to 7.7, a threshold is reached where iron (in the form of $Fe(OH)_c$) is precipitated automatically,

Downstream of cell 3/6, there is a marked increase in the total Fe content of the groundwater. These high values are probably due to the reactivation of iron compounds which are normally found in the lake sediments and in the underlying gravels. Brownlow (1979) described a well in Mississippi which had a total Fe content of 11mg/l and a pH of 6.3. He believes that the Fe content is high for such a pH value, and suggests therefore that the water is O_2 depleted. Within the bottom sediments, the process of organic matter decomposition leads to a reduction in oxygen content. This would facilitate the reduction of any ferric iron to the more soluble ferrous form, which would then be carried downstream by the lake outflow. Wrobel (1980) found that around the Fasaneric-See - a gravel lake in the Munich area - the O_2 saturation of the groundwater fell from 68.5% upstream of the lake to 0% downstream, and that oxygen-deficient groundwater could be traced for up to 700 metres downstream. This aids the dissolution of iron minerals in the aquifer material. The iron content of the groundwater was 0.05 mg/l upstream and 1.59 mg/l downstream of the lake.

There was a general increase in the major ion content of groundwater downstream of cell 3/6 (i.e. at borehole R/52), relative to the lake-water. This was reflected by an increase in electrical conductivity and a substantial decrease in pH. Such conditions were only maintained for a short distance however. By the time the groundwater reached borehole R/22, the concentration of many of the major ions (eg. K^+, Mg^{2+}, total Fe and total alkalinity) had diminished. Only the Ca^{2+}, Na^+ and PO_4 contents and electrical conductivity were increased by flow down-gradient.

The high total alkalinity and total Fe content of groundwater at borehole R/52 appears to be linked. Figure 12.21 shows the total alkalinity and total Fe concentrations downstream of cell 3/6. Cole (1979) notes that in the anoxic bottom sediments of eutrophic lakes, soluble ferrous bicarbonate ($Fe(HCO_3)_2$) is often plentiful. It is thought, therefore, that the high total alkalinity may be due to a high concentration of HCO_3 in the lake outflow. It should also be noted that between boreholes R/52 and R/22, there was a reduction in both the total alkalinity and the total Fe content of the groundwater. An increase in the oxygen content of the groundwater down-gradient probably leads to the re-precipitation of ferric compounds. The high increase in the total alkalinity at borehole R/2 is more difficult to explain. It may be related to an increase in the Ca^{2+} content. The mean Ca^{2+} concentration at borehole R/2 was 115 mg/l compared with only 36 mg/l and 33 mg/l at boreholes R/52 and R/22.

12.5 Conclusions

The discussion given above shows that considerable interaction takes place between the groundwater and gravel lakes.

In lakes which are not sealed, the water level approximates to the pre-extraction groundwater level at the point half-way between the upstream and downstream banks. This simple relationship enables predictions to be made of the range of lake levels to be expected using groundwater level observations.

With time, the deposition of silt and clay in the lake, augmented by chemical and biological changes, leads to a gradual sealing of the downstream lake boundary. This reduces the interaction between the groundwater and the lake and, in certain circumstances where sealing is particularly intense, leads to isolation of the lake from the groundwater system. The most obvious results of sealing are higher than predicted lake levels, brought about by reduced outflow. For example, the hydrograph for Ivy Lane Lake (fig. 12.12) shows that the lake level, far from being mid-way between the upstream and downstream groundwater levels, was much closer to the former. At times, the lake level was higher than the upstream groundwater level with the result that inflow of groundwater ceased. Consistent evidence for lake sealing was provided by lake throughflow rates and groundwater temperatures. Low rates of throughflow (i.e. for Ivy Lane Lake) were caused by clogging of the downstream lake boundary. This in turn, increased the water renewal time (or flushing rate). The temperature of groundwater, downstream of open lakes (eg. cell 3/6 on Ibsley Airfield), was greatly modified by mixing with the outflow. It was cooled in winter and warmed in summer, relative to upstream groundwater. Where the downstream groundwater temperature was little changed from that upstream (eg. downstream of Ivy Lane Lake), sealing of the lake may have occurred, preventing outflow of water.

The difference between the predicted and actual lake levels varied seasonally, being greater at times of low water (i.e. in summer and autumn). McBride and Pfannkuch (1975) showed by numerical modelling and field observations that most seepage into or out of lakes tends to be concentrated near the shore. The rate of seepage was also found to be greatest at the shore. This is probably due to coarser sediments around the edge of the lake.

Where sealing through siltation and biological clogging takes place, it seems reasonable to propose that this will be concentrated in the deeper parts of the lake which are submerged throughout the year. In the littoral zone of a lake, the effects of siltation and biological clogging will be reduced. Wave-action will remove much of the fine material which causes siltation of the banks, and will also prevent or restrict the growth of algae. Also, in summer when large parts of the shore are exposed, any layer of fine sediment which has accumulated will dry-out, crack, and disintegrate. This will diminish its sealing effect. This process was observed in the Ringwood area, around Spinnaker Lake, and is probably very effective in restoring high seepage rates in winter. At high water, therefore, a greater proportion of the littoral zone is submerged. This will facilitate higher seepage rates. At low water, seepage is restricted to the deeper parts of the lake which are more sealed. In this case, seepage rates will be lower and the difference between actual and predicted lake level accentuated.

One effect of an open gravel lake, is to produce a drawdown in the water-table upstream of ~~the~~ lake and a zone downstream where the water-table is increased in level. This is contrary to the popular belief that after dewatering ceases, groundwater levels quickly return to their pre-extraction level. Clearly, in the vicinity of recently flooded excavations this will not be the case.

The sealing of gravel lakes influences the water-table by increasing the upstream groundwater levels, so reducing the area of drawdown, and by lowering downstream groundwater levels. In areas where seepage of water into excavations is a problem to dewatering, artificial sealing of the pits (using puddled clay or impermeable membranes, for example) may be considered to reduce the amount of recirculation. However, in areas where the natural water-table is shallow, sealing is disadvantageous and attempts should be made to prevent it. It may cause the level of the eventual lake to rise above the height of the downstream bank and hence cause flooding. In such areas the orientation of the excavations are particularly important. They ought not to be too elongated in a direction parallel to the groundwater flow direction, since this produces a correspondingly wider zone of influence on groundwater levels. Alternatively, excavating the area in a series of smaller cells may be advantageous.

Lake sealing will take place faster and to a greater degree in eutrophic lakes. Eutrophication, although desirable to wild fowl, should be avoided when the lake is planned for recreational purposes. This is because it commonly results in algal blooms (with attendant odour problems), nuisance rooted aquatic plant growth, sediment infilling (which reduces the usable water surface and helps in sealing the lake), and gradual loss of amenity. Eutrophication may also lead to a diminution of groundwater quality downstream from the lake. It is important to avoid eutrophication in areas where no alteration of groundwater quality should occur (i.e. upstream of public sources). It is recommended therefore that a chemical examination of the groundwater in the upstream area of the proposed gravel lake be undertaken. Of particular interest are those components which are important for algal development, namely PO_4, NO_3, Fe, and perhaps SiO_2. Problems could also occur where sewage effluent or fertilisers are infiltrating to the groundwater.

In areas where the possibility of eutrophication is great, the effects of incorporating a surface outlet should be examined. The increased flushing rate, produced by the increased discharge through the surface outlet, will remove nutrients more efficiently and will therefore be important in reducing the risk of eutrophication and pollution. A disadvantage of having a surface outlet, however, is that it will increase the zone of drawdown upstream. This must be off-set against the loss of amenity brought about by eutrophication.

Gravel lakes fulfill a recreational function, serving as sites for water-based sports as well as for fishing and fish-farming. It is important therefore, when planning their after-use, that attention should be paid to the hydrological factors and how these will affect and be affected by the proposed uses. For example, Stundl (1981) found that the nutrient concentrations of gravel lakes increased markedly with attemps to maintain intensive fisheries, so leading to eutrophication, and that faecal coliform counts tended to rise following their use for recreation. It is therefore strongly recommended that planning of water-based recreational facilities on gravel lakes should be accompanied by hydrological and limnological investigations in the early stages, and that separate areas or lakes should be designated for specific uses, in preference to allowing a wide range of activities over a large body of water.

SECTION V.

CHAPTER 13

CONCLUSIONS

In each of the chapters concerned with the analysis of data, the main conclusions have already been identified and discussed. In the first section of this chapter, these conclusions are drawn together in an attempt to present a model of gravel pit development, with emphasis on its effect on groundwater. This will provide a set of general guidelines which could be applied by gravel operators in areas where gravel extraction is proposed. In the second and final section, some brief retrospective comments on the project and some suggestions for future work in gravel pit hydrology are outlined.

13.1 Model of gravel pit development

One of the primary aims of the thesis was to study the behaviour of groundwater in gravel aquifers, paying particular attention to the responses to gravel extraction and dewatering. In the introduction to the thesis (chapter 1), it was noted that the project broke down into three main topics. Firstly, an analysis of the groundwater characteristics of gravel deposits. Secondly, an analysis of the effects of dewatering on groundwater, and thirdly an assessment of the effects of gravel pit restoration (i.e. in this case, the interactions between gravel lakes and groundwater). The intention is to discuss the major conclusions from each section separately.

Much of the difficulty in previously assessing the effects of dewatering was caused by the lack of precise information on the main input parameters. A feature of this study has been the emergence of saturated hydraulic conductivity as an important factor in determining the effects of gravel pit dewatering. Most previous gravel pit developments have been undertaken on a speculative rather than on a scientific basis. It is important now that the permeability of the gravels are considered in the planning of future developments, particularly where there is likely to be conflict with water interests.

The method of single-well dilution has proved to be a very effective method of determining hydraulic conductivity. This method is to be recommended, especially for use in gravel aquifers where traditional pumping tests are more expensive and often prove unreliable.
The advantages of the dilution method are that it is quick and, despite the initial outlay on equipment, relatively inexpensive. On a large development such as Ibsley Airfield, the whole area could be monitored in a very short time. More importantly, however, the results are comparable with those determined from pumping test analysis.

The results from the Ringwood and Stanton Harcourt areas show that the permeability of gravels varies widely, both laterally and vertically. Values ranging from 309.57 m/d to 22.95 m/d (mean = 77.05 m/d), and 78.61 m/d to 4.30 m/d (mean = 23.39 m/d) were recorded in the Ringwood and Stanton Harcourt areas respectively. The results of simple correlation and factor analysis suggest that, in terms of grain-size, the hydraulic conductivity of gravel deposits is inversely proportional to the percentage silt plus clay content of the gravels. Since gravel operators commonly use grain-size analyses during prospecting as one way of estimating the likely potential of a gravel area, the results could also be used as a qualitative indicator of gravel permeability.

Using a multiple regression equation based on grain-size components calculated by factor analysis, the highest permeability values in the Stanton Harcourt area were estimated for gravel samples taken from frost-wedges and lenses of open-work gravel. This is an important point, because observations have suggested that zones of increased permeability act as preferred routeways for groundwater flow.

Seepage into dewatered pits via large springs, occurs at discrete points around a pit boundary. In some cases, erosive features such as piping and gullying have formed, particularly in proximity to surface water channels where recirculation of pit discharge is great. It has been concluded that the presence of such erosive features may be linked to the concentration of high velocity seepage in frost-wedges, lenses of open-work gravel, or other zones of higher permeability.

As an introduction to investigating the effects of gravel extraction on groundwater, an extensive water-level monitoring programme was undertaken in both study areas. Hydrographs and groundwater contour maps have emphasised the complex nature of gravel aquifers and has highlighted deficiencies in the orthodox theory of recharge. A seasonal trend of groundwater fluctuations was apparent (even in dewatered areas), a feature of which was the rapid response of the water-table to individual rainfall events.

Two types of instantaneous response to rainfall have been identified, each of which differs in terms of the size and shape of the hydrograph. The most important distinguishing feature, however, is that each type of response occurs under different hydrological conditions. The type 1 responses generally occur in late-spring and summer, when there is a soil moisture deficit. Conversely, the type 2 responses, which tend to produce greater fluctuations in groundwater level, mainly occur in winter when there is a moisture surplus. Recharge therefore occurs throughout the year, and is not restricted to the winter months when the soil is at field capacity, as suggested by the orthodox theory of recharge (i.e. vertical recharge through air-filled pores).

It has been proposed that there are two different recharge mechanisms, each of which is predominant at different times of the year. The type 1 responses are thought to result from recharge to the water-table <u>via</u> preferred routeways or from the interception by the boreholes of a static perched water-table. The latter is a temporary feature and is associated with periods of heavy rainfall when soil moisture deficits exist. What remains unclear is the significance of vertical drainage in recharging the permanent water-table. The type 2 responses are thought to be due to the displacement of water already in the soil. This is thought to reflect recharge to the permanent saturated zone. During periods of persistent precipitation, expecially during the winter half of the year, it is possible that both saturation zones exist either separately or combined, depending on the current hydro-meteorological conditions. The use of stable isotopes (O^{18} and C^{13}) could be useful for any future work on the mechanisms of recharge in gravel aquifers.

Longer-term fluctuations in groundwater level have been observed in some boreholes (eg. R/4 and SH/1) which have been shown to be the result of gravel pit dewatering. Dewatering produces a cone of depression, in which the height of the water-table is reduced varying amounts around the excavation. The drawdown is greatest at the pit boundary, where a seepage face develops, but decreases parabolically with increasing distance from the pit. The exact shape of the cone of depression tends to be asymmetrical, depending upon the shape of the pit and the slope of the water-table, generally being elongated in the direction of groundwater flow. Therefore, the effects of the drawdown on other water interests will be felt over a wider area down-gradient of the pit. The zone of drawdown is not a static feature however. It has been observed to contract slightly in winter as a result of prolonged recharge and the exact zone of influence changes as the centre of pumping moves.

Although the shape and extent of the cone of depression is determined to some extent by antecedent groundwater conditions, computer modelling has shown that the hydraulic conductivity of the gravels and the method of gravel extraction are also important determining factors.

The effect of hydraulic conductivity on groundwater drawdown has been analysed using a numerical model of an expanding pit. This model cannot only be used to predict the drawdown around a dewatered pit, but it can also be used to estimate the influence of various hydrological parameters. The following relationships with hydraulic conductivity (K), for example, were observed:

a) the speed at which an aquifer is dewatered increases in direct proportion to K.
b) the shape of the cone of depression in profile is dependent upon K, i.e. at low K. dewatering tends to produce a tight, shallow cone, whereas at higher K, it forms a deep, wide cone.
c) the drawdown at any point between the pit and the line of zero drawdown, and therefore the volume of aquifer dewatered, is directly proportional to K.
d) the speed at which the water-table recovers to its pre-extraction level after dewatering ceases is directly proportional to K.
e) the rate of flow from the aquifer into the pit, and therefore the rate of pumping, is directly proportional to K.

The initial rate at which an excavation is dewatered by pumping, does not seem to have an important influence on groundwater levels in the long term. There is a critical pumping rate, above which only slight increases in drawdown occur. A high initial pumping rate can only be sustained whilst there is sufficient storage within the pit itself. Once a pit has been dewatered, the rate of pumping is dependent upon the rate of inflow from the aquifer and is therefore directly proportional to the hydraulic conductivity of the gravels. Under conditions of a declining water-table, the rate of pumping will decrease through time until equilibrium conditions are reached.

It will be of benefit to gravel operators therefore, to aim for an initial rate of pumping that is near the critical level. This will have two advantages. Firstly, the pit will be dewatered faster and secondly, the effect on groundwater levels will be minimised. At initial rates of pumping below the critical level, it has been shown that the zone of drawdown is deeper and wider in extent than at higher pumping rates. The low discharge allows groundwater to come from more distant parts of the aquifer, whereas at higher rates the water is drawn from that part of the acuifer nearest the pit. On this basis, a greater proportion of the aquifer will be dewatered when pumping is at the lower level. The critical abstraction rate can be determined from the expanding pit model, given that reliable estimates of hydraulic conductivity are available, and that the pit dimensions and the aquifer dimensions are known.

Whether the drawdown of the water-table is a problem depends to a large extent upon local circumstances. Those areas which will be adversely affected by a fall in the water-table are those where agriculture currently benefits from a high water-table or where there are important public or private water sources.

Groundwater in gravels can benefit crops only when capillary rise is sufficient to transfer water from the water-table to the root zone. In the Stanton Harcourt area, this has been shown to be when the water-table is within the soil or if in the gravels, within 30 to 36 cm of the root zone. The areas therefore which will be adversely affected by dewatering are those where the water-table falls below the height of capillary rise.

The lowering of the water-table can, in certain cases, be of benefit to agriculture. A water-table, if too high, can impede soil drainage and adversely affect crop growth, or, if the land is stocked with animals or farmed with heavy machinery, lead to puddling. There seems little doubt therefore that a lowering of the water-table by dewatering would be beneficial in such areas. It is important to point out that these effects would only be temporary, i.e. they would only be apparent during dewatering. When dewatering ceased, or when the point of abstraction was moved so that the area in question was no longer within the zone of influence, groundwater levels would recover slowly towards their original height.

Recovery implies that the water-table returns to its pre-extraction level when the influence of dewatering is removed. Although observations suggest that this is true over much of the zone of influence, recovery is not a rapid process nor does it proceed at the same rate throughout the affected area. The speed of groundwater recovery has been shown to be directly proportional to K, but inversely proportional to the radius of the pit. When dewatering ceases, groundwater levels close to the pit begin to rise almost immediately, as the pit begins to flood. More importantly however, at greater distances from the pit, the water-table continues to fall as groundwater continues to flow towards the pit. It will not be until the pit is completely flooded that groundwater levels throughout the area of influence will have totally recovered.

The way in which a gravel pit is worked has an important influence on gravel pit hydrology. The method of wet-digging produces no discernible cone of depression, although there will be a slight drawdown in the water-table confined to a narrow radius around the pit. The major factors, where dewatering occurs, are the size of the excavation and the way in which the pit discharge is treated.

A substantial arguement can now be put forward in favour of limiting the size of gravel excavations and for using the cell-system of working. The results of tests using the expanding pit model have shown that the volume of aquifer dewatered is directly proportional to, and that the rate of recovery is inversely proportional to, the size of the pit.

The system of working a designated area as a series of small cells reduces the drawdown zone by improving recharge conditions around the pit. Pumping of the pit discharge into a previously worked-out cell provides additional surface storage and increases recharge to the aquifer by leakage through the cell boundary. This will maintain the water-table at relatively high-levels, except in a small area immediately adjacent to the pit being dewatered.

Dewatering schemes are only effective when they remove all seepage into the pit, enabling dry-working of the gravel. Seepage into gravel pits is derived from normal groundwater flow plus, in some cases, induced recharge from surface channels. In any dewatering scheme, some thought must be given to the problem of how to dispose of the water pumped from the pit. The basis of most pumping schemes is to remove all water from the immediate vicinity of the pit in order to prevent recirculation of the water back into it.

Local drainage channels are often given first priority when considering the disposal of pit discharge, since they effectively remove water from the site. It has been observed however, that such ditches can lose water to the surrounding gravels due to seepage through their boundary. Most of this seepage appears to recirculate into the same pit from which it was pumped. For example, 25 to 28% of the discharge from Wadham-Brasenose Pit, near Stanton Harcourt, was recirculated from Standlake Brook. This could be reduced by extending the pumping line, so that the water is discharged into the ditch at a point further from the pit; by restoring the faces adjacent to the ditch with material of lower permeability; by sealing the bed of the drainage channel along the section nearest to the pit; or by using methods which reduce turbulence in the channel and prevent erosion of the bed of the ditch. Some thought should also be given to whether local drainage systems can transmit large volume of extra water without risk of flooding.

The advantage of the cell-system of working is that it reduces the area of groundwater drawdown by increasing recharge. One disadvantage, however, is that it increases the rate of recirculation. On Ibsley Airfield, gravel bunds were left between adjacent cells in such a way that facilitated rapid rates of recirculation through them. It is suggested, therefore, that some thought should be given in future to the size and permeability of these bunds. Where the gravels prove to be highly permeable, it is suggested that the bunds should be sealed or constructed of lower permeable material (i.e. overburden), in order to reduce the amount of recirculation. Sealing would appear to be the more acceptable method and would be easier to implement. Where possible, a layer of overburden (or possibly Oxford Clay in the Stanton Harcourt area) should be constructed along the face between the pit and the adjacent lake, since it is through this face which the greatest amount of seepage has been observed to occur. It should be constructed on the lake-ward side of the bund and could, therefore, form part of the normal restoration scheme. It is not recommended that all sides of the cells be treated in this way, since this in itself may cause later problems; some transfer of water between the lake and aquifer is necessary to prevent overtopping when the lake is allowed to flood.

In areas where there is no suitable material for sealing the bund walls, an alternative method of reducing recirculation must be found.

On Ibsley Airfield, new cells are opened up directly adjacent to the previous cell, so that a narrow bund is immediately formed between the two. Seepage could be reduced by working the cells in reverse, i.e. starting from the point furthest away from the previous one, therefore maintaining the greatest width of gravel between the two for the longest possible time. Also, individual cells on Ibsley Airfield are being excavated in series from east to west, i.e. in the direction of regional groundwater flow. Lakes are formed therefore, up-gradient of the most recently worked cells, so that seepage from the flooded cells will flow westwards under the influence of the natural hydraulic gradient and eventually into the currently worked cell. On the other hand, if the cells were worked in the opposite direction, the lakes would be formed down-gradient of the current cell and the greatest proportion of the seepage would flow westwards and not back into the dewatered cell.

It is usually widely accepted that the effects of gravel extraction on groundwater are temporary and that on completion of dewatering, the original groundwater pattern will be restored and there will be no detrimental effects on the overall flow pattern or groundwater levels. Whilst over most of the area affected by dewatering that is no doubt the case, observations of the interactions between gravel lakes and groundwater has shown that localised, but long-term modifications of the hydrogeological regime may occur around large lakes.

The removal of an area of gravel, to be replaced by a lake, changes the shape of the groundwater surface in a limited area surrounding the excavation. In the Ringwood area, there has been a considerable increase in the hydraulic gradient over an area where a large number of lakes have been developed. This has altered the overall flow pattern in the area, which in turn has had a detrimental effect on the total quantity of water available downstream of the lakes. It has been shown that this is probably the result of the gradual sealing of the lake boundaries, which has reduced the hydraulic continuity between the lakes and the surrounding aquifer.

The level to which a lake will rise after dewatering ceases can be estimated from the pre-extraction hydraulic gradient. The predicted level is equivalent to the height of the original hydraulic gradient at the mid-point of the lake. Actual lake levels which are considerably higher than this, are thought to be direct evidence of lake sealing. The corroborative evidence of reduced throughflow rates and downstream groundwater temperatures which indicate little mixing with lake water, have been given to support this hypothesis.

The increase in lake level as a result of sealing of the lake boundary, produces an increase in upstream groundwater levels and a decrease in downstream groundwater levels. Where the regional surface slope is low, it may also lead to over-topping of the banks and flooding of the surrounds. These are important factors to be considered when planning pit restoration schemes, since the consequences may affect water interests around the lake.

Sealing of a lake is mainly caused by the clogging action of suspended sediment, although chemical and biological clogging may also be significant under certain circumstances. The after-use of a lake will influence the rate of sealing. Those which have been restored with overburden or are used as silt lagoons (eg. Ivy Lane Lake, Ringwood), will quickly show evidence of sealing and may have an adverse effect on the quality and quantity of groundwater downstream. Close liason with the water authorities prior to the commencement of excavation can ensure that silt lagoons, for example, are positioned in the most advantageous position.

The build-up of chemical nutrients in a gravel lake, originating either from the inflow of polluted surface water or, for example, as a result of intensive fish-farming, may eventually lead to eutrophication. Chemical evidence has shown that the quality of groundwater is altered by passage through a lake and that this may be transferred into the aquifer downstream of it. There is a risk, therefore, that downstream groundwater supplies may become polluted by the outflow from eutrophic lakes, although this will depend upon the distance downstream of the source. Observations have shown that the chemical changes produced by the flow of groundwater through a lake, are considerably reduced after passing through 100 to 200 metres of gravel.

The conclusions have shown that the effects of gravel pit dewatering are dependent upon two factors. Firstly, the local hydrological environment and, secondly, the method of gravel working. The most important hydrological factors are the hydraulic conductivity of the gravels, the direction of local groundwater flow and the position of the proposed pit in relation to surface water bodies. It is recommended, therefore, that an initial study of the hydrology of an area should be made prior to the excavation of gravel pits, not only to assess what the likely effects of dewatering will be, but also to determine a satisfactory restoration scheme for the worked-out pits.

As part of the initial study, a certain amount of groundwater monitoring and experimental work should ideally be carried out. Obviously, this will not be possible in all cases, because of difficulties with access and the time factor which is involved. The minimum amount of information which should be gathered would show the likely range of groundwater levels to be expected under normal conditions and the local groundwater flow patterns. Much of this information can be collected with little additional expense. During the initial prospecting phase, when boreholes are being drilled, plastic tubewells of the type described in chapter 4 could be easily installed. These have proved to be completely adequate for groundwater monitoring purposes. It will not be possible to install boreholes during every prospecting phase, because of the uncertainties of whether the site will be acquired and eventually worked. However, where the site has been purchased or where there is known to be some groundwater problems, some thought should be given to the installation of monitoring points. The number and position of these boreholes should be decided prior to drilling, with regard to the site and the nature of the information required.

Where a more detailed study is required, the Expanding-pit numerical model, developed in this thesis, has proven to be a very useful tool in predicting the effect of dewatering on groundwater levels. In turn, it can also be used to predict the likely effects of dewatering on agriculture or water sources. The major advantage of this model is the flexibility it allows to incorporate as many conditions as necessary into a single solution.

It is hoped that the results of the intensive studies undertaken in the Ringwood and Stanton Harcourt areas, along with the recommendations made here, will form a basis on which future dewatering schemes can be based.

13.2 Lines of further enquiry

By virtue of the fact that this is the first study of its kind in this field, much of the work has been involved with laying the foundations for future work. It was stated in the introduction to this thesis that a major aim of the work was to identify the main effects of dewatering. An obvious area for further work is at the more specific scale of study.

Some initial work on the interactions between gravel lakes and groundwater suggest that the relationships are more complex than originally believed. Sealing of the gravel lakes has been shown to be an active process, involving three mechanisms of clogging - by suspended sediment, chemically and biologically. Some additional work on the actual processes of sealing, including the rates at which they operate and under what conditions any one process is dominant, still needs to be undertaken. Similarly, only brief details of the changes in water quality produced by gravel lakes has been outlined. Much more work still needs to be done on the chemical processes which operate in new gravel lakes, on the way in which changes in lake quality may affect groundwater quality, and the effects of various types of after-use on lake quality. The causes of eutrophication in gravel lakes and the rate of biological production are two particularly important areas of lake water quality which require further study, since this can affect their use. In some gravel pits near Newport Pagnell, for instance, some problems have been experienced with lack of food production in gravel lakes which are being used to rear wild-fowl.

Another aim of the work was to develop a model of gravel pit dewatering. Although this has been achieved through the development of the Expanding-pit numerical model, there is still much scope for further work in this area. An important stage in the development of an adequate aquifer model is to check the model behaviour against field data. The author initially planned to go through this process using the Watkins Farm development, but because of various delays through planning applications, the site was never developed during the course of the study. It is only through this process of model calibration that any short-comings in the present model can be identified.

A recurring problem during this study has been the lack of historical evidence, on topics such as groundwater levels, from a period of time before gravel excavations began in the two study areas. In retrospect, this has made it virtually impossible to draw any qualitative comparisons between groundwater patterns before and after the start of gravel extraction. It is recommended therefore that any future workers in this field should aim to study the 'before and after' situation, by carefully chosing a field area where there is no previous history of gravel extraction or dewatering. This will require close co-operation with the gravel operators. In addition, the results of the computer modelling have shown that the greatest effect on groundwater levels occurs at short distances from the pit. One problem with the borehole networks used in this study was that this important area was very largely neglected. As a result this work has concentrated mainly on the effects of dewatering on a macro- or regional scale. An extension of the present work would be to concentrate on the micro-scale, i.e. the area immediately adjacent to the pit itself.

Studies on the dewatering of excavations are very scarce and are mainly descriptive. This work has looked at the effects in one particular environment (i.e. gravel) and, even then, not at every possible scale of study. Objective, comparative work on different types of excavations, for example, limestone quarries, is needed before a general model of the effects of dewatering on groundwater can be advanced. Thus, within this subject there is a wealth of research yet to be undertaken. It is hoped that this thesis points the way and suggests techniques with which these other areas of study may be analysed.