Groundwater resource development

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Preface

In 1980 the International Decade of Drinking Water and Sanitation was declared by the World Health Organisation but tragically the news during the first half of the 1980s has been dominated by reports of devastating droughts in parts of Africa. Hence the importance to mankind of a continuous clean supply of water could not have been demonstrated more effectively. Unfortunately, however, many surface sources of water are subject to extreme temporal and spatial variations, as the African droughts testify. On the other hand, groundwater represents about 98 per cent of the world's supply of fresh water. This source is usually relatively pure and is not allowed by climatic extremes in the way that surface sources are. The challenge is to develop the groundwater resource.

In England and Wales something like 30 per cent of the total water supply is satisfied by groundwater, and in the United States this figure rises to around 50 per cent. Obviously, in those areas centred on major aquifers the figure is appreciably higher. As the demand for water increases in such developed areas, and opposition to the construction of new reservoirs mounts, then the development of groundwater schemes is likely to assume increasing importance. Furthermore a groundwater development scheme is not so capital intensive as that of a large reservoir project. Another advantage is that a groundwater scheme can be introduced gradually to keep pace with demand and a degree of flexibility is possible.

Despite the importance of groundwater supplies, there are few groundwater specialists whose sole function is to plan, develop and manage groundwater resources. Normally this is an interdisciplinary undertaking involving geologists, civil engineers, environmental scientists, mathematicians, chemists and water well contractors, for example. For this reason the book has been written so as to be of value to those involved in these disciplines in the profession of water engineering. Generally the only background knowledge required is some basic geology and hydraulics, although the mathematical modelling requires a knowledge of calculus.

The book describes the steps involved in the search for productive aquifers, their exploration, the construction and testing of water wells, water quality and pollution considerations, groundwater management and groundwater models. These represent the basic steps concerned with the development of a groundwater resource.

Material, especially for illustration, has been supplied by many firms and individuals, and due acknowledgement is given in the text wherever appropriate. In addition the authors wish to record their sincere thanks to all concerned. If any person or firm inadvertently has not been given an acknowledgement, then our apologies are humbly offered.

Chapter 1

Development of groundwater resources

1.1 Introduction

At the minimum subsistence level, most human beings generally require about 2.5 l of water every day for direct consumption. In Britain about 5 l per capita per day is required for drinking and cooking, yet the average amount of water used domestically each day by every person is around 200 l (although about 25 per cent of this is unaccounted for leakage from the distribution system). However, the total per capita usage may appear to be higher than this since the needs of industry are often accredited to domestic users and averaged out. Generally, industry requires approximately one-quarter to one-third of the public water supply, with the remainder being used in the home. Thus the total per capita usage is around 300 l per day, with some variations according to the concentration of heavy industry.

For example, in the early 1970s Manchester, in the industrial north of England, had an average demand of 317 l per capita per day, while Southampton required 268 l per capita. The quantity of water supplied to consumers in other parts of the world varies according to the habits of the local populace, the needs of industry and the availability of water. In Rome and Milan the total consumption during the same period was 440 and 545 l per capita per day, while in Istanbul and Brussels the equivalent figures were 149 and 116 l, respectively[1].

When assessing industrial usage, it should be remembered that some factories have their own private sources and that these are not taken into consideration when deriving the total per capita consumption. Nevertheless, the most striking fact to emerge from these figures is the large amount of water consumed by individuals in the developed countries.

Normally the easiest and most convenient way to meet the public demand for water is to utilize surface water resources. Unfortunately, fresh water rivers and lakes are less plentiful than may at first be imagined and, in fact, account for less than 0.01 per cent of the world's total water, and less than 2 per cent of the world's fresh water. To complicate matters further, what little fresh water there is tends to be distributed spatially and temporally in an irregular manner, while the sources that are available have often been overdeveloped or polluted. Groundwater, on the other hand, accounts for about 98 per cent of the world's fresh water and is fairly well distributed throughout the world. It provides a reasonably constant supply which is not likely to dry up under natural conditions, as surface sources may do, and is often of quite a high quality.

Partly for these reasons, some 55 200 public water wells were constructed in Korea over a five-year period as part of a United Nations programme[2]. However, there is one significant disadvantage associated with the utilization of groundwater resources: unless the water issues naturally from the ground surface in the form of springs, it can only be abstracted by the construction of a well. If the groundwater lies some distance below the surface, then this also implies that some form of pump must be provided unless the groundwater is under artesian pressure. As a result, both the running cost of the well, in the form of power for the pump, and the initial construction cost may be quite high so that the water obtained from deep wells may be expensive compared to that derived from surface sources. This is the price to be paid for overcoming gravity while raising the groundwater to the surface. Consequently, in many parts of the world groundwater resources have been neglected in favour of the cheaper and more accessible surface supplies.

Nonetheless, the construction of a large reservoir requires a huge investment of capital initially, while the capacity of the reservoir may not be utilized fully until 20, or perhaps even 50 years after filling. Groundwater abstraction schemes, on the other hand, can be implemented stage by stage, as required, so that the actual yield of the wells is closely related to the demand. This means that the capital investment involved is controlled and initially relatively small[3]. Additionally, once the initial aquifer investigation has been completed, water wells can be sunk and brought into supply relatively quickly so that a rapid and flexible response can be made to an increase in demand. Especially in times of economic stringency these attributes can make groundwater exploitation schemes attractive and viable propositions.

Groundwater schemes offer advantages of a different type in many overseas countries. In hot climates evaporation from the surface of a reservoir can be extremely significant, accounting for perhaps one-third of the flow into the reservoir. Also surface reservoirs have resulted in large areas of slow-moving water which provide breeding grounds for mosquitos, so increasing the incidence of malaria. For the same reason there has also been a vast increase in the amount of the disease, bilharzia[4]. In developing or hot countries there are, therefore, definite advantages in ensuring that supply is closely matched to demand and that surplus water is not stored in surface reservoirs for long periods. Obviously there is little point in providing 340 l per capita of purified water when the average demand is about 5 l per capital per day[4]. In these situations groundwater exploitation provides a very suitable solution to the water supply problem.

If a groundwater resource is to be exploited, it is essential that the entire project is conducted in the most efficient and cost-effective way possible. This means that the wells must be designed so as to be efficient, and that the pumps must operate at near peak efficiency for most of the time in order to minimize pumping costs, but it also implies that every step of the operation must be carefully planned and implemented in a predetermined logical order. Rushing into a groundwater development programme will probably result in the incorrect location of wells, inefficient design of the abstraction works, high water costs and problems in managing both the wells and the aquifer. The end result may even be the abandonment of the well-field. If a projected groundwater scheme is carefully thought out in advance, however, there is no reason why a reliable supply of water should not be obtained, assuming that this is possible hydrogeologically.

The evaluation, development and management of an aquifer for water supply should proceed in stages. The simplest and least expensive phase of the investigation should be undertaken first, with the data so obtained being used to decide the way forward to the next, more expensive, phase. This approach is intended to minimize the risk of sinking costly wells in the wrong places. Although it is impossible to anticipate all the eventualities that may occur during the evaluation of a groundwater resource, a progression from one stage to another in a sensible, controlled manner should improve the chances of success. The various phases of a typical investigation could be as set out in *Figure 1.1*.

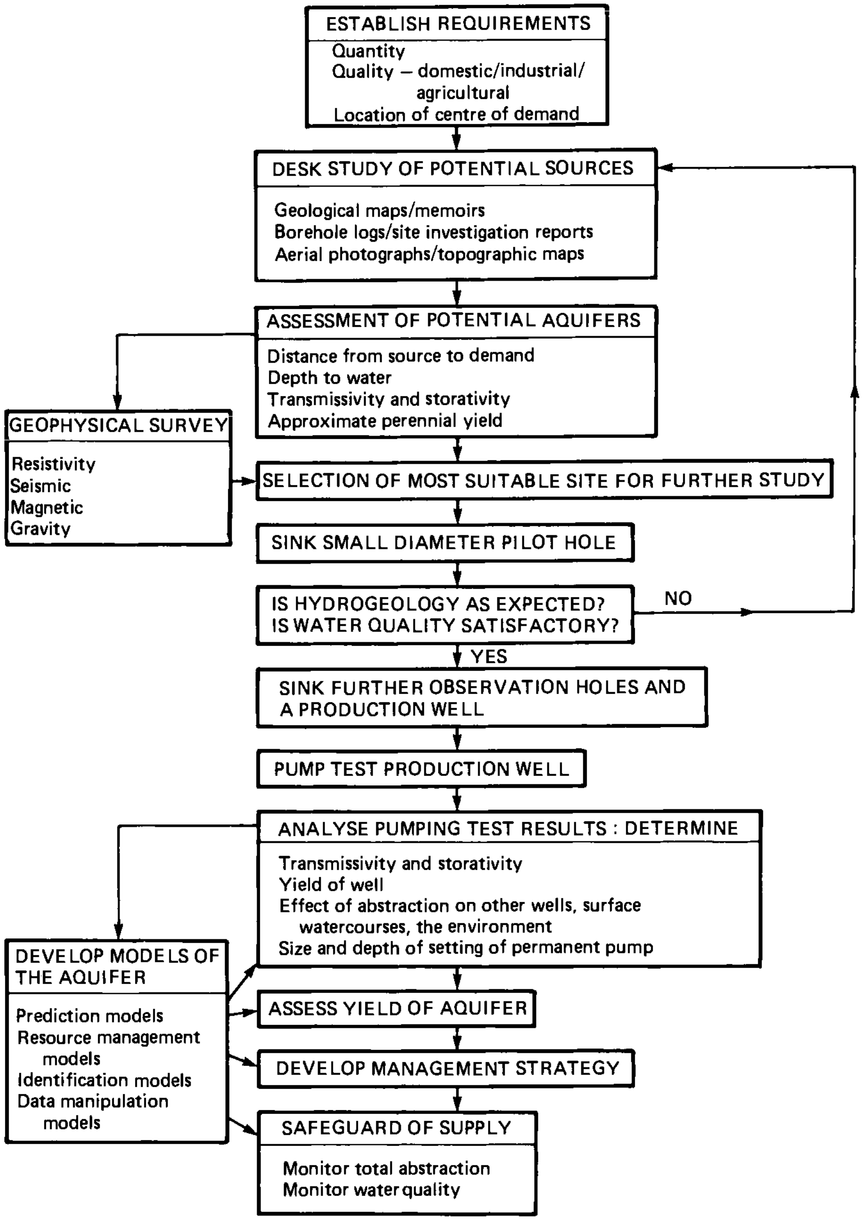

Figure 1.1 Steps in the evaluation, development and management of an aquifer

1.2 Establishment of the requirement

One of the first steps in an investigation must be to establish the quantity and quality of the water required from the groundwater development. The quantity may be governed by a deficit in existing supplies, or by a projected increase in the domestic, industrial or agricultural demand. The nature of the demand and the ultimate destination of the water can be important, since this may determine the quality of the water to be supplied. For instance, to satisfy the needs for a growing population something like 200 l per capita per day of potable water must be made available. This means that a high-quality source must be located, namely an aquifer that is unpolluted, not saline or highly mineralized, and which is unlikely to be affected by contamination. On the other hand, if the water is for certain industrial uses then a source of relatively poor quality may suffice. Water which is unacceptable for drinking purposes may be adequate for use as a coolant.

1.3 Desk feasibility study

This study is undertaken in order to locate the most promising areas for economic development and to establish the economic viability (or otherwise) of the alternative sources. Only after such a study would more expensive investigative techniques, such as pump-testing, be implemented. Obviously, consulting geological maps and memoirs provides a much cheaper means of establishing the broad geology of an area than does sinking boreholes. This assumes, of course, that maps of the area are available and that they are reasonably accurate. Nevertheless, care should be taken to ensure that money is not spent on a field investigation of an area which has been the subject of a detailed study at some time in the past and for which adequate records exist.

A desk study involves assembling all the relevant data concerning a particular area. The general objective is to determine:

1. The location, extent, and thickness of any potentially suitable aquifers.
2. The distance between the centre of demand and the various aquifers. As this distance increases, so does the capital cost of constructing the supply pipeline. The pumping and maintenance costs that have to be borne throughout the life of the project also increase with the distance between the source and the consumer.
3. The depth to water at the various potential well sites. Both the initial construction costs and the subsequent pumping costs can be reduced if the wells are as shallow as possible. This must be considered in conjunction with (2).
4. The transmissivity and storativity of the various aquifers. Whenever possible, wells should be located in areas with a large coefficient of transmissivity, since this facilitates flow to the wells and results in a higher yield.
5. The approximate perennial yield of the various wells and well-fields. This may be estimated from meteorological and hydrological data.

After these parameters have been considered, it should be possible to decide whether or not any of the aquifers are capable of yielding the water required at an economic rate. Additionally, the most promising sites can be identified for field investigation.

1.4 Field investigation

The purpose of a field investigation is to prove the actual hydrogeology of the areas selected for further study, to establish the potential yield of the wells and the aquifer as a whole, to investigate the quality of the groundwater and generally, to confirm the results of the desk study. Until field tests have been conducted, it can never be taken for granted that the conditions on site will be the same as those anticipated.

Despite their undoubted usefulness, field investigations are expensive so it is imperative that such investigations are undertaken logically and in a way which will minimize costs. Thus, the investigation should commence by sinking a small diameter pilot or observation hole at the most promising site. If the data from the pilot hole suggest that the site is unsuitable, then the hole can be abandoned without incurring a very high cost. This would not be the case if the field investigation was initiated by sinking a very large diameter production well, on the mistaken assumption that the site would prove to be suitable. On the other hand, if the results from the pilot hole are favourable, a production well can be sunk nearby and the pilot hole used as an observation hole when the well is pump tested. However, before undertaking a lengthy pumping test, either the pilot hole and/or the production well can be tested using an air lift technique. This involves forcing air down a delivery tube located in the well. The air displaces water from the well, so effecting a pumping action. This gives an indication of the rate of flow to the well and whether or not the penetrated strata can be justifiably termed an aquifer. Sometimes it may transpire that the formation is of low permeability, very thin, or is absent from the geological sequence at the location in question. Under these circumstances a new site must be selected for investigation, bearing in mind the lessons learnt from the abandoned site.

The quality of the water obtained from the pilot hole must also be taken into consideration when assessing the viability of a particular site. If the water is of unacceptable quality, and its treatment would prove uneconomic, then the site has to be abandoned. Sometimes, however, water quality can vary significantly with depth as a result of the stratification of the flow through an aquifer. Water, for example, in thick aquifers, often becomes more saline with increasing depth. Consequently, this may govern the depth from which water may be abstracted[5]. In some situations, pumping the well for a lengthy period may bring about an improvement in quality.

Prolonged well pumping tests involving the use of several observation holes are expensive to conduct and should generally only be undertaken when it seems likely that the site will be suitable for water supply. However, these tests provide information not just on the potential yield of a borehole, but also on the drawdown that can be expected, the transmissivity and storativity of the aquifer, the most suitable size and depth of setting for the permanent pump, and the effect of abstraction on the environment, surface watercourses and other wells in the area. Clearly, pumping tests supply a lot of data that are of value in completing the well and deriving appropriate management strategies. Such data are essential if the consequences of abstraction are to be evaluated and the perennial yield and quality of water pumped from the well are to be maintained.

1.5 Safeguard of supply

Once an aquifer has been developed as a source of water, especially potable water, it is necessary to protect the supply and to ensure that no harmful activities, which could jeopardize its continuity or quality, take place in the vicinity of the groundwater catchment. This is basically a management function and may involve monitoring the recharge area of the aquifer to detect undesirable activities, such as the dumping of domestic or industrial wastes. Changes in land use should also be noted. The routine testing of water quality should be undertaken to ensure that the chemical and biological quality of the water is stable and satisfactory. Sudden variations in quality should be investigated, since this may be the first indicator of the onset of groundwater pollution, or the failure of the management strategy.

The management of an aquifer may be relatively easy in the early stages of its development, but becomes progressively more and more difficult in later stages as the amount of water abstracted increases. Ultimately, the pumpage may equal or even exceed the perennial yield of the aquifer. Under these conditions it is very difficult to operate the resource in a manner that does not, in some way, adversely affect the quality, quantity, or the economics of the water supply.

The objective of any management strategy, as far as a groundwater resource is concerned, must be to ensure that an acceptable supply of water can be economically and continuously maintained at a rate approximately equal to the demand. Often there may be more than one way of achieving this objective, in which case some form of computer modelling or optimization technique may be adopted. These are capable of evaluating various operational strategies with regard to some criteria such as the least cost or the maximum yield. In the latter case some limit on the drawdown in the aquifer may have to be specified. Although overpumping may be permissible on a short-term basis, in general it is undesirable because it may leave some wells dry, it may take groundwater levels some time to recover with the result that increased pumping costs are incurred, or it may cause a permanent deterioration in water quality within the aquifer. Groundwater modelling techniques may also be of value in determining the nature and significance of potential problems associated with various management options. These problems may include saline intrusion or induced infiltration as a result of surface–groundwater interconnection. It is important to realize that surface water and groundwater are part of the same resource, part of the hydrological cycle and as such are inter-related[6].

1.6 Approximate cost of groundwater development schemes

At some time during the planning of a groundwater development scheme the question of cost must arise. This may occur when a preliminary feasibility study is conducted concerning the possible exploitation of a groundwater resource, although the question of cost is likely to re-occur all the way through a groundwater development programme. Of particular concern may be how the cost of underground water will compare with that from surface sources, the overall cost of the scheme, the number of wells and observation holes that can be afforded, and so on. Questions of this type will be almost endless, while answers may be scarce and can vary from day to day as new geological, hydrogeological or technological information is obtained as the investigation progresses.

Frequently it is very difficult to assess in advance the cost of a groundwater development scheme, since there may be a large number of unknown variables. Nevertheless, some estimate of cost is essential in order to judge the viability of a scheme and to avoid becoming involved with an uneconomic enterprise. It must be appreciated from the outset that a groundwater exploitation project is expensive to implement. Groundwater does not necessarily represent a cheap solution to a water shortage, although it may if conditions are favourable. Often water derived from a surface source is less expensive because of greater accessibility and reduced running costs. A deep production well of large diameter can cost anything up to £500 000 including equipment and fittings. An entire groundwater development scheme is a multi-million pound investment. It must be appreciated also that there are many hidden costs that may easily be overlooked. The total cost consists not just of the expense of constructing the appropriate number of water wells, but also of sinking exploratory boreholes, observation holes and undertaking well development and pumping tests. Many large schemes have incorporated up to 80 observation holes, which may represent about 10 per cent of the total expenditure (*Table 1.1*). Developing, testing and equipping a well may double the actual construction cost. Instrumentation and telemetry may account for about 2 to 5 per cent of the total cost of the scheme, while compensation (for land, etc.) may represent around 5 per cent. The construction of a

TABLE 1.1. Details of multiple borehole schemes (1976 prices) (after Water Research Centre[7])

Aquifer type	*Yield of scheme*					
	45 Ml/day		*90 Ml/day*		*135 Ml/day*	
	Chalk	*Permo-Triassic Sandstone*	*Chalk*	*Permo-Triassic Sandstone*	*Chalk*	*Permo-Triassic Sandstone*
50 per cent probability yield of aquifer (Ml/day)	3.3	2.2	3.3	2.2	3.3	2.2
Number of boreholes required for scheme yield	14	20	27	41	41	61
	Cost (£000's 1976 Q3)					
Construction cost	203	275	365	567	567	841
Additional costs:						
A. Acidization cost (£2500 per borehole)	35	—	67	—	103	—
B. Test-pumping cost (£6000 per borehole)	84	120	162	246	246	366
C. Pump/rising main/ switchgear cost (£8000 per borehole)	112	160	216	328	328	488
D. Headworks chamber cost (£2000 per borehole)	28	40	54	82	82	122
Cost of abstraction boreholes (i.e. construction cost and additional costs)	462	595	864	1222	1326	1817
Ancillary costs:						
1. Exploratory boreholes (number required)	(2)	(2)	(2)	(3)	(3)	(4)
Construction cost (£6000 per borehole)	12	12	12	18	18	24
Acidization and/or test-pumping of exploratory boreholes	17	12	17	18	26	24
2. Observation boreholes (number required)	(28)	(40)	(50)	(60)	(60)	(80)
Construction cost (£2000 per borehole)	56	80	100	120	120	160
TOTAL COST	547	699	993	1378	1490	2025

pipeline to connect the wells to the distribution system may be 80 to 300 per cent of the total cost of the abstraction boreholes. For further details of costs see reference 7.

The viability of marginal aquifers can change as a result of variations in inflation, exchange rates and the price of power, equipment and raw materials. The same factors make it difficult to quote typical costs since within a period of a few years the prices often look ridiculously small. None the less, average and total costs relating to 1976 are listed in *Table 1.1*. Obviously some calculation—mental or otherwise—must be performed to translate these into the present-day equivalent. As a rough guide between 1976 and 1985 costs have increased by about a factor of 3.

The equations given in *Table 1.2* may be of value in obtaining an estimate of the cost of a groundwater development scheme. The original publication[7] lists the sources of information and the assumptions used in deriving the cost models. It also breaks down the total cost into smaller components. The reader is advised to study these before applying the models to a proposed development, since no simple equation can be

TABLE 1.2. Cost models for single and multiple boreholes (after Water Research Centre[7])

Cost of a single borehole—no screen or pack. (1976 costs)

$$\mathrm{COST} = 0.851 \times \mathrm{DEP}^{0.49} \times \mathrm{DIAM}^{0.64} \times \mathrm{CASLEN}^{0.21} \tag{1.1}$$

where COST is total construction cost including setting up, drilling, casing and grouting (£000's). Mean value 13.3.*
DEP is depth of borehole (m). Mean value 126.
DIAM is diameter of borehole (m). Mean value 0.616.
CASLEN is length of casing (m). Mean value 32.5.

Cost of a single borehole—with screen and pack. (1976 costs)

$$\mathrm{COST} = 1.94 \times \mathrm{DEP}^{0.62} \times \mathrm{SCRTYP}^{-0.44} \quad \text{if drilled diameter 0.5–0.8 m} \tag{1.2}$$

where COST is total construction cost including setting up, drilling, casing, grouting, screen and packing (£000's). Mean value 26.9.*
DEP is depth of borehole (m). Mean value 77.1.
SCRTYP = 1 for a screen made of stainless steel or rubber-coated steel with a pre-formed pack, or
= 2 for a mild steel slotted screen.

$$\mathrm{COST} = 3.00 \times \mathrm{DEP}^{0.42} \times \mathrm{DIAM}^{0.62} \times \mathrm{CASLEN}^{0.14} \quad \text{if drilled diameter 0.8–1.0 m} \tag{1.3}$$

where DIAM is drilled diameter (m).
CASLEN is length of casing (m), and the other variables are as Equation (1.2).

Cost of multiple borehole schemes—no screen or pack. (1976 costs)
The model is based on borehole depths of 80 to 165 m in consolidated aquifers. See also *Table 1.1*.

$$\mathrm{COST} = 2.08 \times \mathrm{DIAM}^{1.26} \times \mathrm{CASLEN}^{0.63} \times \mathrm{NOBHS}^{1.16} \tag{1.4}$$

where COST is total cost of setting up, drilling, casing and grouting the boreholes (£000's). Mean value 72.0.*
DIAM is mean borehole diameter (m). Mean value 0.586.
CASLEN is mean casing length per borehole (m). Mean value 30.9.
NOBHS is number of boreholes. Mean value 6.69.

Cost of multiple borehole schemes—with screen and pack. (1976 costs)
No data available. Use repeated application of the single borehole equation.

* The cost of development, test pumping, installation of the pump and associated equipment and the construction of the headworks chamber and instrumentation house are not included in the above models

applicable to all situations in all parts of the world. In fact most of the base data were obtained from two major aquifers in Britain—the Chalk and the Triassic Sandstone. However, in the preliminary stages of an investigation any estimate of costs may be extremely useful, and the models are included for this reason.

1.6.1 Updating costs

The authors of the work described[7] intended that the costs derived from the models should be updated to allow for inflation by using a suitable index. One such index is the *Monthly Index of Average Earnings—Construction.* This is published by the Central Statistical Office (CSO) monthly and annually[8,9]. Since many of the activities involved in groundwater development work are labour-intensive this index may serve as a useful guide, but others are available.

The costs quoted in connection with the models relate to the third quarter of 1976 (1976 Q3), at which time the index just referred to had a value of 241. The equivalent cost in 1985 (Quarter 1) would be obtained as follows:

$$\text{Cost at 1985 Q1 prices} = \text{Cost at 1976 Q3} \times \frac{\text{Index at 1985 Q1}}{\text{Index at 1976 Q3}} \tag{1.1}$$

However, the base of the index is changed from time to time. As a result the values below relate to January 1976 = 100. Therefore, an approximate indication of present-day costs can be obtained from the models simply by using the following multiplication factors, or more accurately by using Equation (1.1).

Year		*Average earnings index (new series) construction* (January 1976 = 100)
1976		106.5
1977		118.3
1978	Annual average	132.1
1979		151.2
1980		180.7
1981		204.1
1982		223.5
December 1982		235.7

Figures for the period after December 1982 are not available at the time of writing. These can be obtained from references 8 and 9, by estimation, or by the reader using his or her own earnings as a guide.

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Chapter 2

Groundwater: fundamentals

2.1 The origin and occurrence of groundwater

The principal source of groundwater is meteoric water, that is, precipitation (rain, sleet, snow and hail). However, two other sources are very occasionally of some consequence. These are juvenile water and connate water. The former is derived from magmatic sources whilst the latter represents the water in which sediments were deposited. This was trapped in the pore spaces of sedimentary rocks as they were formed and has never been expelled.

The amount of water that infiltrates into the ground depends upon how precipitation is dispersed, namely, on what proportions are assigned to immediate run-off and to evapotranspiration, the remainder constituting the proportion allotted to infiltration/percolation (see *Figure 4.3*).

The retention of water in a soil depends upon the capillary force and the molecular attraction of the particles. As the pores in a soil become thoroughly wetted the capillary force declines so that gravity becomes more effective. In this way downward percolation can continue after infiltration has ceased but as the soil dries, so capillarity increases in importance. No further percolation occurs after the capillary and gravity forces are balanced. Thus, water percolates into the zone of saturation when the retention capacity is satisfied.

2.1.1 The water table

The pores within the zone of saturation are filled with water, generally referred to as phreatic water. The upper surface of this zone is therefore known as the phreatic surface but is more commonly termed the water table. Above the zone of saturation is the zone of aeration in which both air and water occupy the pores. The water in the zone of aeration is commonly referred to as vadose water. Meinzer[1] divided this zone into three belts, those of soil water, the intermediate belt and the capillary fringe (*Figure 2.1*). The uppermost or soil water belt discharges water into the atmosphere in perceptible quantities by evapotranspiration. In the capillary fringe, which occurs immediately above the water table, water is held in the pores by capillary action. An intermediate belt occurs when the water table is far enough below the surface for the soil water belt not to extend down to the capillary fringe. The degree of saturation decreases from the water table upwards, saturation occurring only in the immediate neighbourhood of the

Zones and sub zones

Aeration	Soil water	Vadose water	Hygroscopic	Percolation	Discontinuous capillary saturation	
	Intermediate		Pellicular		Semi-continuous capillary saturation	
	Capillary fringe		Capillary		Continuous capillary saturation	Water table
Saturation	Phreatic zone	Phreatic water	Ground-water	Seepage	Unconfined groundwater	

Figure 2.1 Zones and sub-zones of groundwater

water table. However, where the water table is at shallow depth and the maximum capillary rise is large, moisture is continually attracted from the water table due to evaporation from the ground surface. Hence the soil is saturated, or nearly so.

The water table fluctuates in position, particularly in those climates where there are marked seasonal changes in rainfall. Thus permanent and intermittent water tables can be distinguished, the former marking the level beneath which the water table does not sink whilst the latter is an expression of the fluctuation. Usually water tables fluctuate within the lower and upper limits rather than between them, especially in humid regions, since the periods between successive recharges are small. The position at which the water table intersects the surface is termed the spring line. Intermittent and permanent springs similarly, can be distinguished.

A perched water table is one which forms above a discontinuous impermeable layer such as a lens of clays in a formation of sand, the clay impounding a water mound. As such a perched water table occurs in the zone of aeration above the true water table (*Figure 2.2*(*a*)).

2.1.2 Aquifers, aquicludes, aquitards and aquifuges

An aquifer is the term given to a rock or soil mass which not only contains water, but from which water can be readily abstracted in significant quantities. The ability of an aquifer to transmit water is governed by its permeability. Indeed the permeability of an aquifer usually is in excess of 10^{-5} m/s.

By contrast, a formation with a permeability of less than 10^{-9} m/s is one which is regarded as impermeable and is referred to as an aquiclude. For example, clays and shales are aquicludes. Even when such rocks are saturated they tend to impede the flow of water through stratal sequences.

According to De Wiest[2] an aquitard is a formation which transmits water at a very slow rate but which, over a large area of contact, may permit the passage of large amounts of water between adjacent aquifers which it separates. Sandy clays provide an example.

A rock which neither transmits nor stores water is called an *aquifuge*.

An aquifer is described as unconfined when the water table is open to the

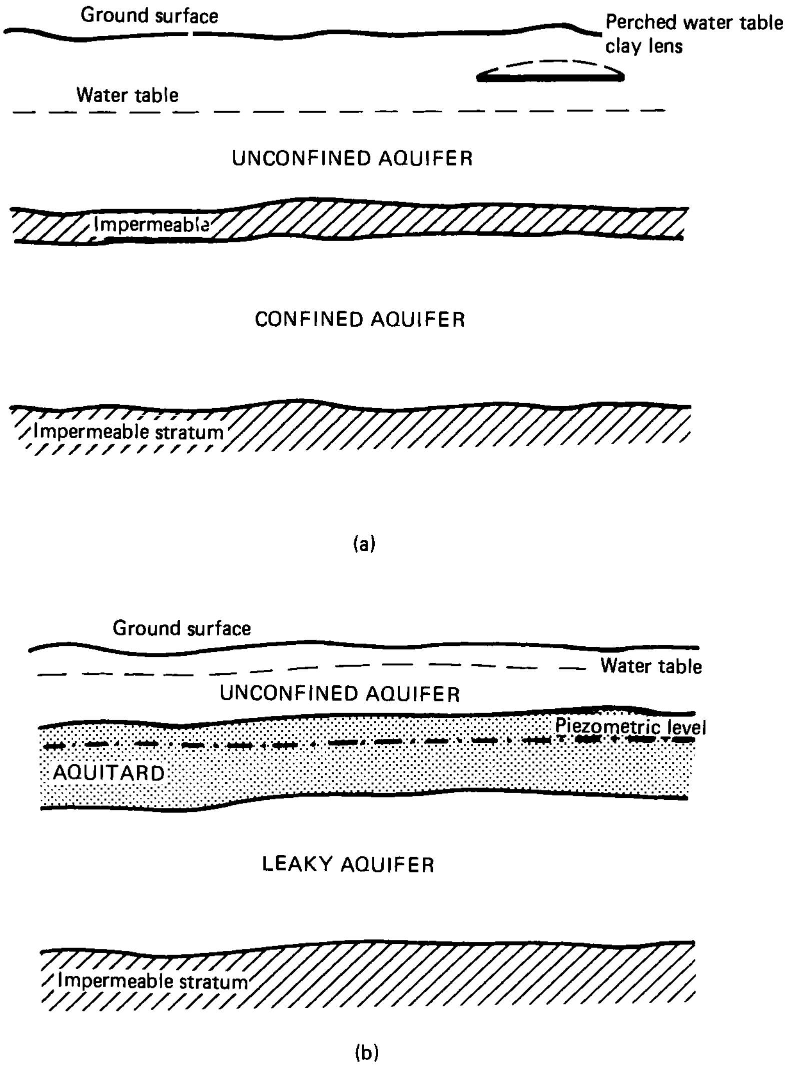


Figure 2.2 (a) Diagram illustrating unconfined and confined aquifers, with a perched water table in the vadose zone. (b) Diagram illustrating a leaky aquifer

atmosphere, that is, the aquifer is not overlain by material of lower permeability (*Figure 2.2*(*a*)). Conversely a confined aquifer is one which is overlain by impermeable rocks (*Figure 2.2*(*a*)). Confined aquifers may have relatively small recharge areas as compared with unconfined aquifers and therefore may yield less water.

An aquifer which is overlain and/or underlain by an aquitard(s) is described as a leaky aquifer (*Figure 2.2*(*b*)). Even though these semipervious bounding beds offer a relatively high resistance to the flow of water through them, large amounts of water may flow from aquitard to aquifer and vice versa as a result of the extensive contact area between them. The direction and quantity of leakage in either case depends on the difference in piezometric head which exists across the semipervious aquitard.

Very often the water in a confined aquifer is under piezometric pressure, that is, there is an excess of pressure sufficient to raise the water above the base of the overlying bed when the aquifer is penetrated by a well. Piezometric pressures are developed when the buried upper surface of a confined aquifer is lower than the water table in the aquifer at its recharge area. Where the piezometric surface is above ground level, then water overflows from a well. Such wells are described as artesian. A synclinal structure is the commonest cause of artesian conditions (*Figure 2.3*). The term subartesian is used to describe those conditions in which the water is not under sufficient piezometric pressure to rise to the ground surface.

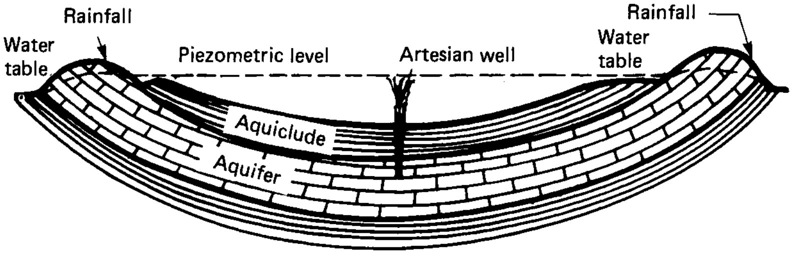


Figure 2.3 An artesian basin

2.2 Porosity and permeability

Porosity and permeability are the two most important factors governing the accumulation, migration and distribution of groundwater. However, both may change within a rock or soil mass in the course of its geological evolution. Furthermore, it is not uncommon to find variations in both porosity and permeability per metre of depth beneath the ground surface.

2.2.1 Porosity

The porosity, n, of a rock can be defined as the percentage pore space within a given volume and is expressed as follows

$$n = V_v/V \times 100 \tag{2.1}$$

where V_v is the volume of the voids and V is the total volume of the material concerned. A closely related property is the void ratio, e, that is, the ratio of the volume of the voids to the volume of the solids, V_s,

$$e = V_v/V_s \tag{2.2}$$

Where the ground is fully saturated the void ratio can be derived from

$$e = mG_s \tag{2.3}$$

m being the moisture content and G_s the relative density (specific gravity). Both the porosity and the void ratio indicate the relative proportion of void volume in the material and the relationship between the two is as follows

$$n = e/(1+e) \tag{2.4}$$

and

$$e = n/(1-n) \tag{2.5}$$

Total or absolute porosity is a measure of the total void volume and is the excess of bulk volume over grain volume per unit of bulk volume. It is usually determined as the excess of grain density (the same as specific gravity, now referred to as relative density) over dry density, per unit of grain density and can be obtained from the following expression

$$\text{Absolute porosity} = \left(1 - \frac{\text{Dry density}}{\text{Grain density}}\right) \times 100 \tag{2.6}$$

The effective, apparent or net porosity is a measure of the effective void volume of a porous medium and is determined as the excess of bulk volume over grain volume and occluded pore volume. It may be regarded as the pore space from which water can be removed.

The factors affecting the porosity of a rock include particle size distribution, sorting, grain shape, fabric, degree of compaction and cementation, solution effects and, lastly, mineralogical composition, particularly the presence of clay particles.

The highest porosity is commonly attained when all the grains are the same size. The addition of grains of different size to such an assemblage lowers its porosity and this is, within certain limits, directly proportional to the amount added. Irregularities in grain shape result in a larger possible range of porosity, as irregular forms may theoretically be packed either more tightly or more loosely than spheres. Similarly angular grains may either cause an increase or a decrease in porosity.

After a sediment has been buried and indurated, several additional factors help determine its porosity. The chief amongst these are closer spacing of grains, deformation and granulation of grains, recrystallization, secondary growth of minerals, cementation and, in some cases, solutioning. Hence diagenthic changes undergone by a rock may either increase or decrease its original porosity.

The porosity can be determined experimentally by using either the standard saturation method[3] or an air porosimeter[4]. Both tests give an effective value of porosity, although that obtained by the air porosimeter may be somewhat higher because air can penetrate pores more easily than can water.

The porosity of a deposit does not necessarily provide an indication of the amount of water that can be obtained therefrom. Nevertheless, the water content of a soil or rock is related to its porosity. The water content of a porous material is usually expressed as the percentage of the weight of the solid material, W_s, that is

$$m = (W_w/W_s) \times 100 \tag{2.7}$$

where W_w is the weight of the water. The degree of saturation, S_r, refers to the relative volume of water, V_w, in the voids, V_v, and is expressed as a percentage

$$S_r = (V_w/V_v) \times 100 \tag{2.8}$$

2.2.2 Specific retention and specific yield

The capacity of a material to yield water is of greater importance than its capacity to hold water as far as supply is concerned. Even though a rock or soil may be saturated, only a certain proportion of water can be removed by drainage under gravity or pumping, the remainder being held in place by capillary or molecular forces. The ratio of the volume of water retained, V_{wr}, to that of the total volume of rock or soil, V, expressed as a percentage, is referred to as the specific retention, S_{re}

$$S_{re} = V_{wr}/V \times 100 \tag{2.9}$$

The amount of water retained varies directly in accordance with the surface area of the pores and indirectly with regard to the pore space. The specific surface of a particle is governed by its size and shape. For example, particles of clay have far larger specific surfaces than do those of sand. As an illustration, a grain of sand, 1 mm in diameter, has a specific surface of about 0.002 m^2/g, compared with kaolinite, which varies from approximately 10 to 20 m^2/g. Hence clays have a much higher specific retention than sands (*Figure 2.4*).

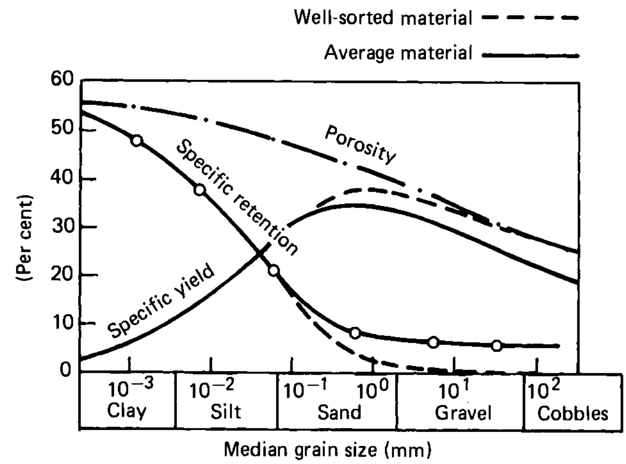


Figure 2.4 Relationship between grain size, porosity, specific retention and specific yield (from Bear, J., *Hydraulics of Groundwater*, McGraw Hill, New York (1979))

The specific yield, S_y, of a rock or soil refers to its water yielding capacity attributable to gravity drainage as occurs when the water table declines. It was defined by Meinzer[5] as the ratio of the volume of water, after saturation, that can be drained by gravity, V_{wd}, to the total volume of the aquifer, expressed as a percentage hence

$$S_y = V_{wd}/V \times 100 \tag{2.10}$$

The specific yield plus the specific retention is equal to the porosity of the material

$$n = S_y + S_{re} \tag{2.11}$$

when all the pores are interconnected. The relationship between the specific yield and particle size distribution is shown in *Figure 2.4*. In soils the specific yield tends to decrease as the coefficient of uniformity increases. Examples of the specific yield of some common types of soil and rock are given in *Table 2.1* (it must be appreciated that individual values of specific yield can vary considerably from those quoted). The specific yield can be determined in the laboratory, or can be estimated from data obtained from pumping tests, from neutron logging or from particle size and sorting[6].

2.2.3 Permeability

Permeability may be defined as the ability of a rock to allow the passage of fluids into or through it without impairing its structure. In ordinary hydraulic usage a substance is

TABLE 2.1. Some examples of specific yield

Material	*Specific yield* (%)
Gravel	15–30
Sand	10–30
Dune sand	25–35
Sand and gravel	15–25
Loess	15–20
Silt	5–10
Clay	1–5
Till (silty)	4–7
Till (sandy)	12–18
Sandstone	5–25
Limestone	0.5–10
Shale	0.5–5

termed permeable when it permits the passage of a measurable quantity of fluid in a finite period of time and impermeable when the rate at which it transmits that fluid is slow enough to be negligible under existing temperature–pressure conditions (*Table 2.2*). The permeability of a particular material is defined by its coefficient of permeability or hydraulic conductivity (the term *hydraulic conductivity* is now used in place of coefficient of permeability, it being the flow in m^3/s through a unit cross sectional area of a material). The transmissivity or flow in m^3/day through a section of aquifer 1 m wide under a hydraulic gradient of unity is sometimes used as a convenient quantity in the calculation of groundwater flow instead of the hydraulic conductivity. The transmissivity, T, and coefficient of permeability, k, are related to each other as follows

$$T = kH \tag{2.12}$$

where H is the saturated thickness of the aquifer.

The flow through a unit cross section of material is modified by temperature, hydraulic gradient and the hydraulic conductivity. The latter is affected by the uniformity and range of grain size, shape of the grains, stratification, the amount of consolidation and cementation undergone and the presence and nature of discontinuities. Temperature changes affect the flow rate of a fluid by changing its viscosity. The rate of flow is commonly assumed to be directly proportional to the hydraulic gradient but this is not always so in practice.

Permeability and porosity are not necessarily as closely related as would be expected, for instance, very fine textured sandstones frequently have a higher porosity than coarser ones, though the latter are more permeable. Nevertheless, Griffith[7] did find that a linear relationship existed between porosity and the logarithm of permeability in sands.

As can be inferred from above, the permeability of a clastic material is also affected by the interconnections between the pore spaces. If these are highly tortuous then the permeability is accordingly reduced. Consequently tortuosity figures importantly in permeability, influencing the extent and rate of free water saturation. It can be defined as the ratio of the total path covered by a current flowing in the pore channels between two given points to the straight line distance between them.

Stratification in a formation varies within limits both vertically and horizontally. It is frequently difficult to predict what effect stratification has on the permeability of the beds. Nevertheless, in the great majority of cases where a directional difference in

TABLE 2.2. Relative values of permeabilities

Rock types	Porosity: Primary (grain)	Porosity: Secondary (fracture)*	Permeability range (m/s): 10^{0} 10^{-2} 10^{-4} 10^{-6} 10^{-8} 10^{-10} (Very high, High, Medium, Low, Very low, Impermeable)	Well yields: High, Medium, Low	Type of water-bearing unit
	(%)				
Sediments, unconsolidated					
Gravel	30–40				Aquifer
Coarse sand	30–40				Aquifer
Medium to fine sand	25–35				Aquifer
Silt	40–50	Occasional			Aquiclude
Clay, till	45–55	Often fissured			Aquiclude
Sediments, consolidated					
Limestone, dolostone	1–50	Solutions joints, bedding planes			Aquifer or aquifuge
Coarse, medium sandstone	< 20	Joints and bedding planes			Aquifer or aquiclude
Fine sandstone	< 10	Joints and bedding planes			Aquifer or aquifuge
Shale, siltstone	—	Joints and bedding planes			Aquifuge or aquifer
Volcanic rocks, e.g. basalt	—	Joints and bedding planes			Aquifer or aquifuge
Plutonic and metamorphic rocks		Weathering and joints decreasing as depth increases			Aquifuge or aquifer

* Rarely exceeds 10 per cent

permeability exists, the greater permeability is parallel to the bedding. For example, the Permo-Triassic sandstones of the Mersey and Weaver Basins are notably anisotropic as far as permeability is concerned, the flow parallel to the bedding being higher than across it. Ratios of 5:1 are not uncommon and occasionally values of 100:1 have been recorded where fine marl partings occur.

The permeability of intact rock (primary permeability) is usually several orders less than the *in situ* permeability (secondary permeability). Although the secondary permeability is affected by the frequency, continuity and openness, and amount of infilling, of discontinuities, a rough estimate of the permeability can be obtained from their frequency (*Table 2.3*). Admittedly such estimates must be treated with caution and cannot be applied to rocks which are susceptible to solution.

TABLE 2.3. Estimation of secondary permeability from discontinuity frequency

		Permeability	
Rock mass description	*Interval* (m)	*Term*	k (m/s)
Very closely to extremely closely spaced discontinuities	Less than 0.2	Highly permeable	10^{-2}–1
Closely to moderately widely spaced discontinuities	0.2–0.6	Moderately permeable	10^{-5}–10^{-2}
Widely to very widely spaced discontinuities	0.6–2.0	Slightly permeable	10^{-9}–10^{-5}
No discontinuities	Over 2.0	Effectively impermeable	Less than 10^{-9}

Basaltic laval flows which are intersected by cooling joints provide an example of fissure flow through rock. As expected the frictional resistance to flow through such joint systems is frequently much lower than that offered by a porous medium, hence appreciable quantities of water may be transmitted. Another example is provided by the massive limestones of Lower Carboniferous age of the Pennine area, the permeability of an intact sample being much lower than that obtained by field tests (10^{-16} to 10^{-11} m/s, and 10^{-5} to 10^{-1} m/s respectively). The significantly higher permeability found in the field is attributable to the joint systems and bedding planes which have been opened by solutioning. The mass permeability of sandstones is also very much influenced by the discontinuities. For instance, the average laboratory permeability for the Fell Sandstone Group from Shirlawhope Well near Longframlington, Northumberland, is 17.4×10^{-7} m/s[8]. This compares with an estimated value of 2.4×10^{-3} m/s obtained from field tests. From the foregoing examples it can be concluded that as far as the assessment of flow through rock masses is concerned, field tests (see below) provide more reliable results than can be obtained from testing intact samples in the laboratory. However, the walls of discontinuities are invariably irregular and this has a retarding effect upon flow movement. Moreover, discontinuities tend to close with depth. Indeed joints, when exposed at the surface, have usually been opened up by weathering processes. Dissipation of residual stress on removal of overburden also aids joint development.

Dykes often act as barriers to groundwater flow so that the water table on one side may be higher than on the other. Fault planes occupied by clay gouge may have a similar effect. Conversely they may act as conduits where the fault plane is not sealed.

The movement of water across a permeable boundary which separates aquifers of different permeabioities leads to deflection of flow, the bigger the difference the larger the deflection. When groundwater meets an impermeable boundary it flows along it and, as noted previously, in some situations, such as the occurrence of a dyke, may be impounded. The nature of a rock mass also influences whether flow is steady or unsteady. Generally it is unsteady since it is usually due to discharge from storage.

2.2.4 Storage coefficient

The storage coefficient or storativity, S, of an aquifer has been defined as the volume of water released from or taken into storage per unit surface area of the aquifer, per unit change in head normal to that surface (*Figure 2.5*). It is a dimensionless quantity. Changes in storage in an unconfined aquifer represent the product of the volume of the aquifer between the water table before and after a given period of time and the specific yield. Indeed the storage coefficient of an unconfined aquifer virtually corresponds to the specific yield as more or less all the water is released from storage by gravity drainage and only an extremely small part results from compression of the aquifer and the expansion of water[9].

However, in confined aquifers water is not yielded simply by gravity drainage from the pore space because there is no falling water table and the material remains saturated. Hence other factors are involved regarding yield such as consolidation of the aquifer and expansion of water consequent upon lowering of the piezometric surface. Therefore, much less water is yielded by confined than unconfined aquifers. Indeed Jacob[10] maintained that the elasticity of a confined aquifer under artesian conditions is relatively important in terms of yield of groundwater. Consequently he took into

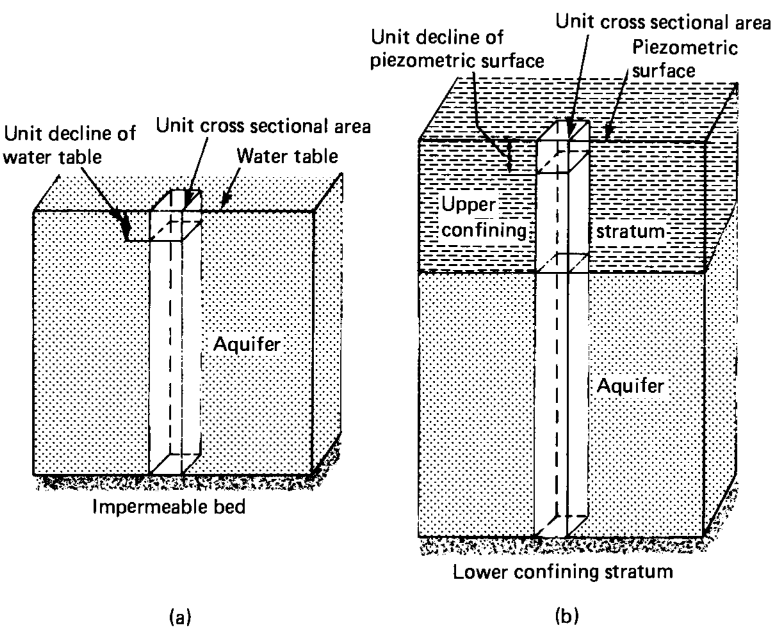


Figure 2.5 Diagram illustrating the storage coefficient of (a) an unconfined aquifer and (b) a confined aquifer

consideration the compressibility of both the water and the solid aquifer skeleton when determining the storage coefficient, S, of such an aquifer

$$S=\gamma_w n\beta H\left(1+\frac{\alpha}{n\beta}\right) \tag{2.13}$$

in which γ_w is the unit weight of water, n is the porosity, β is the compressibility of the water, H is the saturated thickness of the aquifer and α is the compressibility of the solid aquifer skeleton. According to Lohman[6] the values of the storage coefficient in most confined aquifers fall within the range 10^{-5} to 10^{-3}, which indicates that significant changes of pressure are required over extensive areas in order to produce substantial yields of water. Lohman also suggested a rule-of-thumb means for estimating the coefficient of confined aquifers, it is

$$S=3\times 10^{-6}H \tag{2.14}$$

Just as the specific yield can be determined by pumping tests or by using a neutron moisture probe, so can the storage coefficient. In the latter case the probe is used to determine the moisture content of the material saturated and when drained[9].

2.3 Flow through soils and rocks

Water possesses three forms of energy, namely, potential energy attributable to its height, pressure energy owing to its pressure, and kinetic energy due to its velocity. The latter can usually be discounted in any assessment of flow through soils. Energy in water is usually expressed in terms of head. The head possessed by water in soils or rocks is manifested by the height to which water will rise in a standpipe above a given datum. This height is usually referred to as the piezometric level and provides a measure of the total energy of the water. If at two different points within a continuous area of water there are different amounts of energy, then there will be a flow towards the point of lesser energy and the difference in head is expended in maintaining that flow. Other things being equal, the velocity of flow between two points is directly proportional to the difference in head between them. The hydraulic gradient, i, refers to the loss of head or energy of water flowing through the ground. This loss of energy by the water is due to the friction resistance of the ground material and this is greater in fine- than coarse-grained soils. Thus, there is no guarantee that the rate of flow will be uniform, indeed this is exceptional. However, if it is assumed that the resistance to flow is constant, then for a given difference in head the flow velocity is directly proportional to the flow path.

2.3.1 Darcy's law

Before any mathematical treatment of groundwater flow can be attempted certain simplifying assumptions have to be made, namely, that the material is isotropic and homogeneous, that there is no capillary action and that a steady state of flow exists. Since rocks and soils are anisotropic and heterogeneous, as they may be subject to capillary action and as flow through them is characteristically unsteady, any mathematical assessment of flow must be treated with caution.

The basic law concerned with flow is that enunciated by Darcy[11] which states that the rate of flow, v, per unit area is proportional to the gradient of the potential head, i, measured in the direction of flow

$$v = ki \tag{2.15}$$

and for a particular rock or soil or part of it, of area, A

$$Q = vA = Aki \tag{2.16}$$

where Q is the quantity in a given time. The ratio of the cross sectional area of the pore spaces in a soil to that of the whole soil is given by $e/(1+e)$, where e is the void ratio. Hence a truer velocity of flow, that is, the seepage velocity, v_s, is

$$v_s = [(1+e)/e]ki \tag{2.17}$$

Darcy's law is valid as long as a laminar flow exists. Departures from Darcy's law, therefore, occur when the flow is turbulent such as when the velocity of flow is high. Such conditions exist in very permeable media, normally when the Reynolds number* can attain values above four. Accordingly it is usually accepted that this law can be applied to those soils which have finer textures than gravels. Furthermore Darcy's law probably does not accurately represent the flow of water through a porous medium of extremely low permeability, because of the influence of surface and ionic phenomena and the presence of gases.

Apart from an increase in the mean velocity, the other factors which cause deviations from the linear laws of flow include, first, the non-uniformity of pore spaces, since differing porosity gives rise to differences in the seepage rates through pore channels. A second factor is an absence of a running-in section where the velocity profile can establish a steady state parabolic distribution. Lastly, such deviations may be developed by perturbations due to jet separation from wall irregularities.

Darcy omitted to recognize that permeability also depends upon the density, ρ, and dynamic viscosity of the fluid, μ, involved, and the average size, D_n, and shape of the pores in a porous medium. In fact, permeability is directly proportional to the unit weight of the fluid concerned and is inversely proportional to its viscosity. The latter is very much influenced by temperature. The following expression attempts to take these factors into account.

$$k = CD_n^2 \rho/\mu \tag{2.18}$$

where C is a dimensionless constant or shape factor which takes note of the effects of stratification, packing, particle size distribution and porosity. It is assumed in this expression that both the porous medium and the water are mechanically and physically stable, but this may never be true. For example, ionic exchange on clay and colloid surfaces may bring about changes in mineral volume which, in turn, affect the shape and size of the pores. Moderate to high groundwater velocities will tend to move colloids and clay particles. Solution and deposition may result from the pore fluids. Small changes in temperature and/or pressure may cause gas to come out of solution which may block pore spaces.

It has been argued that a more rational concept of permeability would be to express

* Reynolds number, N_R, is commonly used to distinguish between laminar and turbulent flow and is expressed as follows

$$N_R = \rho \frac{vR}{\mu}$$

where ρ is density, v is mean velocity, R is hydraulic radius and μ is dynamic viscosity. Flow is laminar for small values of Reynolds number.

it in terms that are independent of the fluid properties. Thus the intrinsic permeability (k_i) characteristic of the medium alone has been defined as

$$k_i = CD_n^2 \tag{2.19}$$

However, it has proved impossible to relate C to the properties of the medium. Even in uniform spheres it is difficult to account for the variations in packing arrangement. In this context a widely accepted relationship for laminar flow through a permeable medium is that given by Fair and Hatch[12]

$$k = \frac{1}{m\left[\dfrac{(1-n)^2}{n^3}\left(\dfrac{\theta}{100}\sum\dfrac{P}{D_m}\right)^2\right]} \tag{2.20}$$

where n is the porosity, m is the packing factor found by experiment to have a value of 5, θ is the particle shape factor varying from 6.0 for spherical to 7.7 for angular grains, P is the percentage of particles by weight held between each pair of adjacent sieves, and D_m is the geometric mean opening $(D_1D_2)^{1/2}$ of the pair.

The Kozeny–Carmen equation for deriving the coefficient of permeability also takes the porosity into account as well as the specific surface area of the porous medium, S_a, which is defined per unit volume of solid

$$k = C_0 \frac{n^3}{(1-n)^2 S_a^2} \tag{2.21}$$

where C_0 is a coefficient, the suggested value of which is 0.2.

2.3.2 General equation of flow

When considering the general case of flow in porous media, it is assumed that the media is isotropic and homogeneous as far as permeability is concerned. If an element of saturated material is taken, with the dimensions dx, dy and dz (*Figure 2.6*) and flow is

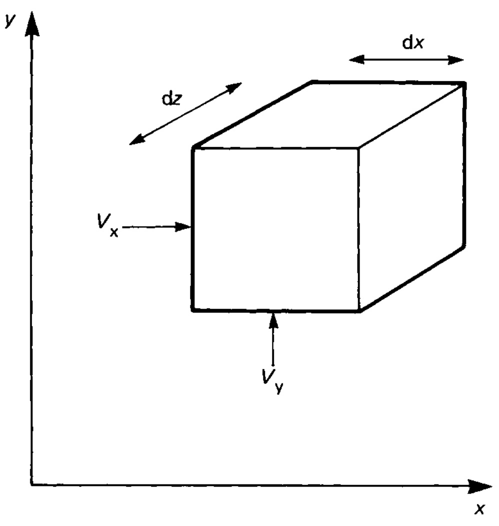


Figure 2.6 Seepage through a soil element

taking place in the x–y plane, then the generalized form of Darcy's Law is

$$v_x = k_x i_x \tag{2.22}$$

$$= k_x \frac{\delta h}{\delta x} \tag{2.23}$$

and

$$v_y = k_y i_y \tag{2.24}$$

$$_y = k_y \frac{\delta h}{\delta y} \tag{2.25}$$

where h is the total head under steady state conditions and k_x, i_x and k_y, i_y are, respectively, the coefficients of permeability and the hydraulic gradients in the x and y directions. Assuming that the fabric of the medium does not change and that the water is incompressible, then the volume of water entering the element is the same as that leaving in any given time, hence

$$v_x \,\mathrm{d}y\,\mathrm{d}z + v_y \,\mathrm{d}x\,\mathrm{d}z = \left(v_x + \frac{\delta v_x}{\partial x}\mathrm{d}x \right)\mathrm{d}y\,\mathrm{d}z + \left(v_y + \frac{\delta v_y}{\delta y}\mathrm{d}y \right)\mathrm{d}x\,\mathrm{d}z \tag{2.26}$$

In such a situation the difference in volume between the water entering and leaving the element is zero, therefore

$$\frac{\partial v_x}{\delta x} + \frac{\delta v_y}{\delta y} = 0 \tag{2.27}$$

Equation (2.27) is referred to as the flow continuity equation. If Equations (2.23) and (2.25) are substituted in the continuity equation, then

$$k_x \frac{\delta^2 h}{\delta x^2} + k_y \frac{\delta^2 h}{\delta y^2} = 0 \tag{2.28}$$

If there is a recharge or discharge to the aquifer ($-w$ and $+w$ respectively) then this term must be added to the right-hand side of the above equation. For all steady-state conditions the addition of the function w gives the Poisson equation.

If it is assumed that the hydraulic conductivity is isotropic throughout the media so that $k_x = k_y$, then Equation (2.28) becomes

$$\frac{\delta^2 h}{\delta x^2} + \frac{\delta^2 h}{\delta y^2} = 0 \tag{2.29}$$

This is the two-dimensional Laplace equation for steady-state flow in an isotropic porous medium (often written as $\nabla^2 h = 0$). The partial differential equation governing the two-dimensional unsteady flow of water in an anisotropic aquifer can be written as

$$T_x = \frac{\delta^2 h}{\delta x^2} + T_y \frac{\delta^2 h}{\delta y^2} = S \frac{\delta h}{\delta t} \tag{2.30}$$

where T and S are the coefficients of transmissivity and storage respectively. In polar coordinates the equivalent equation is:

$$\frac{\delta^2 h}{\delta r^2} + \frac{1}{r}\frac{\delta h}{\delta r} = \frac{S}{T}\frac{\delta h}{\delta t} \tag{2.31}$$

where r is the radial distance from the well. This form of the equation is often used in pumping test analysis (see Section 7.4.2).

2.3.3 Flow through stratified deposits

In a stratified sequence of deposits the individual beds will, no doubt, have different permeabilities, so that vertical permeability will differ from horizontal permeability. Consequently, in such situations, it may be necessary to determine the average values of the coefficient of permeability normal to (k_v) and parallel to (k_h) the bedding. If the total thickness of the sequence is H_T and the thickness of the individual layers are $H_1, H_2, H_3, \ldots, H_n$, with corresponding values of the coefficient of permeability k_1, k_2, $k_3, \ldots, k_n$, then k_v and k_h can be obtained as follows

$$k_v = \frac{H_T}{H_1/k_1 + H_2/k_2 + H_3/k_3 + \cdots + H_n/k_n} \tag{2.32}$$

and

$$k_h = \frac{H_1 k_1 + H_2 k_2 + H_3 k_3 + \cdots + H_n k_n}{H_T} \tag{2.33}$$

2.4 Fissure flow

Generally it is the interconnected systems of discontinuities which determine the permeability of a particular rock mass. Indeed, as mentioned above, the permeability of a jointed rock mass is usually several orders higher than that of intact rock. According to Serafin[13], the following expression can be used to derive the filtration through a rock mass intersected by a system of parallel-sided joints with a given opening, e, separated by a given distance, d.

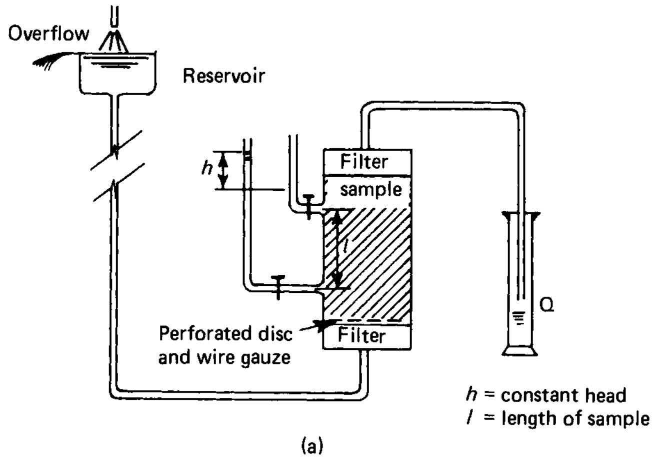
$$k = \frac{e^3 \gamma_w}{12 d \mu} \tag{2.34}$$

where γ_w is the unit weight of water and μ its viscosity. The velocity of flow, v, through a single joint of constant gap, e, is expressed by

$$v = \left(\frac{e^2 \gamma_w}{12\mu} \right) i \tag{2.35}$$

where i is the hydraulic gradient. Flow through a jointed mass was also considered by Castillo *et al.*[14].

Wittke[15] suggested that where the spacing between discontinuities is small in comparison with the dimensions of the rock mass, it is often admissible to replace the fissured rock, with regard to its permeability, by a continuous anisotropic medium, the permeability of which can be described by means of Darcy's law. He also provided a resumé of procedures by which three-dimensional problems of flow through rocks under complex boundary conditions could be solved.



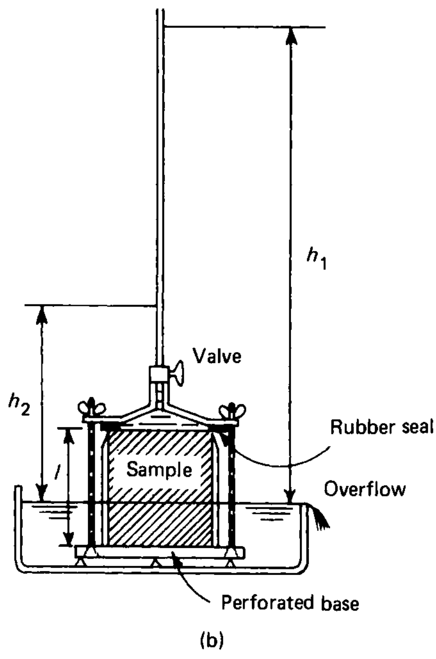


Figure 2.7 (a) The constant head permeameter. (b) The falling-head permeameter

2.5 Assessment of permeability

2.5.1 Assessment of permeability in the laboratory

Permeability is assessed in the laboratory by using either a constant head or falling head permeameter. A constant head permeameter (*Figure 2.7(a)*) is used to measure the permeability of granular materials such as gravels and sands[16]. A sample is placed in a cylinder of known cross sectional area, A, and water is allowed to move through it under a constant head. The amount of water discharged, Q, in a given period of time, t, together with the difference in head, h, over a given length of sample, l, measured by means of manometer tubes, is obtained. The results are substituted in the Darcy expression (see Section 2.3.1) and the coefficient of permeability, k, thereby derived.

$$Q/t=(Ak)h/l=Aki \tag{2.36}$$

where i is the hydraulic gradient.

Determination of the permeability of fine sands and silts, as well as many rock types, is made by using a falling head permeameter (*Figure 2.7(b)*). The sample is placed in the apparatus which is then filled with water to a certain height, h_1, in the standpipe. Then the stopcock is opened and the water infiltrates through the sample, the height of the water in the standpipe falling to h_2. The times at the beginning, t_1, and end, t_2, of the test are recorded. These, together with the cross sectional area, A, and length of sample, l, are then substituted in the following expression, which is derived from Darcy's law, to obtain the coefficient of permeability, k

$$k=\frac{2.303al}{A(t_2-t_1)}\log_{10}\left(\frac{h_1}{h_2}\right) \tag{2.37}$$

where a is the cross sectional area of the standpipe. The permeability of clay cannot be measured by using a permeameter, it must be determined indirectly, for example, from the consolidation test.

2.5.2 Assessment of permeability in the field

Test wells are normally of small diameter and therefore can be drilled at a fraction of the cost of full-sized wells but are still expensive. When a test well indicates a favourable location for water supply, it can usually be used as an observation well for a pumping test or converted to a production well by reaming or redrilling to increase its diameter.

An initial assessment of the magnitude and variability of the *in situ* hydraulic conductivity can be obtained from tests carried out in boreholes as the hole is advanced. By artificially raising the level of water in the borehole (falling head test) above that in the surrounding ground, the flow rate from the borehole can be measured. However, in very permeable soils it may not be possible to raise the level of water in the borehole. Conversely the water level in the borehole can be artificially depressed (rising head test) so allowing the rate of water flow into the borehole to be assessed. Wherever possible a rising and a falling head test should be carried out at each required level and the results averaged.

In a rising or falling head test in which the piezometric head varies with time, the permeability is determined from the expression

$$k=\frac{A}{F(t_2-t_1)}\ln\left(\frac{H_1}{H_2}\right) \tag{2.38}$$

where H_1 and H_2 are the piezometric heads at times t_1 and t_2, respectively, A is the inner cross sectional area of the casing in the borehole and F is an intake or shape factor[16]. Where a borehole of diameter, D, is open at the base and cased throughout its depth, $F=2.75D$. If the casing extends throughout the permeable bed to an impermeable contact, then $F=2D$. The test procedure involves observing the water level in the casing at given times and recording the results from which a graph of water level against time is constructed (*Figure 2.8*).

When tests are required in sands and gravels, they can normally be carried out with the casing flush with the bottom of the borehole. Careful 'shelling-out' will be necessary to avoid overboring the hole and the borehole must be kept topped up with water to prevent piping. Piping it likely to occur during a rising-head test in fine-grained

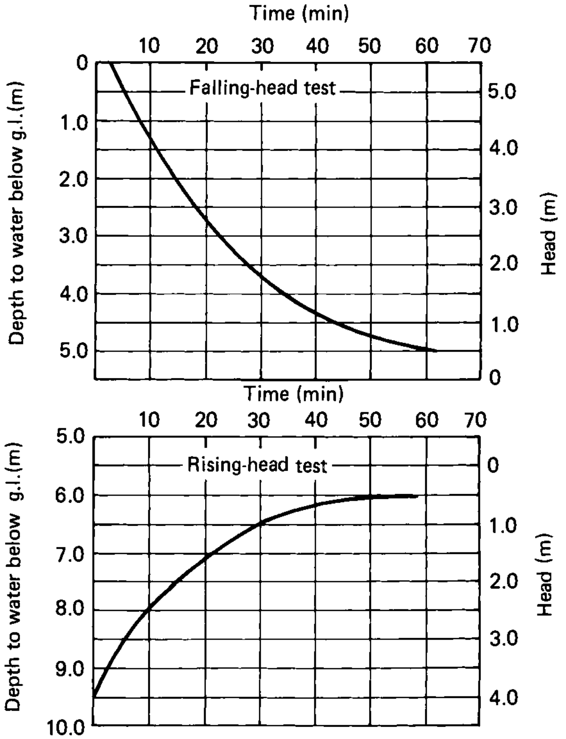


Figure 2.8 Rising- and falling-head permeability tests

materials and it is advisable to partially backfill the hole with about 1 m of gravel to counteract this. It will not be necessary to withdraw the casing unless the test is required over an unlined section of borehole. The hole should, if necessary, be flushed out so that the test is carried out in clean water conditions.

The constant head method of *in situ* permeability testing is used when the rise or fall in the level of the water is too rapid for accurate recording (i.e. occurs in less than 5 min). This test is normally conducted as an inflow test in which the flow of water into the ground is kept under a sensibly constant head (e.g. by adjusting the rate of flow into the borehole so as to maintain the water level at a mark on the inside of the casing near the top). The method is only applicable to permeable ground such as gravels, sands and broken rock, when there is a negligible or zero time for equalization. The rate of flow, Q, is measured once a steady flow into and out of the borehole has been attained over a period of some 10 min. The permeability, k, is derived from the following expression

$$k = Q/FH_c \tag{2.39}$$

where F is the intake factor and H_c is the applied constant head[17].

Sutcliffe and Mostyn[18] described the use of the above-mentioned tests in soils. The constant head tests were run through the drill string. Generally a section at the bottom of the hole, 3 m in length, was tested by withdrawing the drill string 3 m. Alternatively, the uncased section was tested by withdrawing the drill string into the casing. These authors discussed the problems, especially that of leakage, associated with these tests.

The permeability of an individual bed of rock can be determined by a water injection or packer test carried out in a drillhole. This is done by sealing off a length of uncased hole with packers and injecting water under pressure into the test section (*Figure 2.9*).

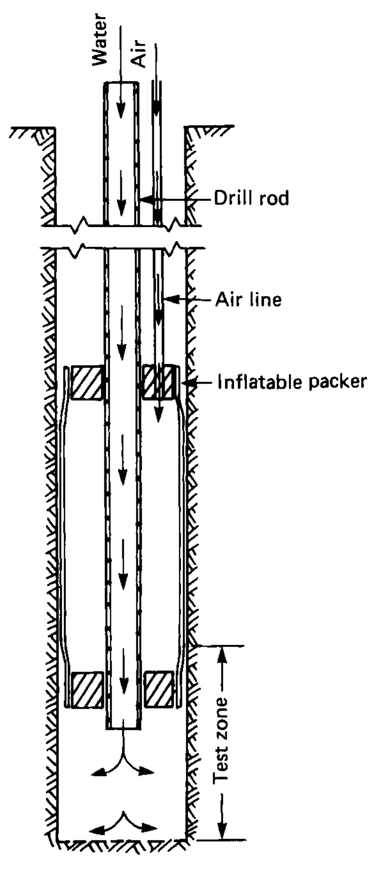


Figure 2.9 Double packer test equipment. The zone of rock to be tested in a drillhole is isolated by using two packers which seal off the drillhole, the water being pumped into the space between the packers. An alternative method which can be carried out only as drilling proceeds is to use a single packer for testing the zone between the bottom of the packer and the base of the drillhole. The average flow under equilibrium conditions is obtained from a metered water supply acting under a known pressure and gravity head

However, Brassington and Walthall[19] pumped water from between packers and, provided that low heads are used, showed that more or less the same results can be obtained as those from injection tests. Usually, because it is more convenient, these permeability tests are carried out after the entire length of a hole has been drilled. Two packers are used to seal off selected test lengths and the tests are performed from the base of the hole upwards. The hole must be flushed to remove sediment prior to a test being performed. Sutcliffe and Mostyn[18] found that using wireline pneumatic packers was much more effective and considerably quicker, particularly in poor ground conditions, than using mechanical or hydraulic packers. The identification of detailed flow patterns is of importance in relation to well design. With double packer testing the variation in hydraulic conductivity throughout the test hole is demonstrated.

The rate of flow of water over the test length is measured (usually through a flow meter) under a range of constant pressures and recorded. The permeability is calculated from the flow-pressure curve, a typical example of which is given in *Figure 2.10*.

Water is generally pumped into the test section at steady pressures for periods of 15 min, readings of water absorption being taken every 15 min. The test usually consists of five cycles at successive pressures of 6, 12, 18, 12 and 6 kPa for every metre depth of packer below the surface[20].

For each test section, the applied pressure, p, corrected for pipe friction losses, should be plotted against the flow rate, q. Some typical pressure curves for tests in which the

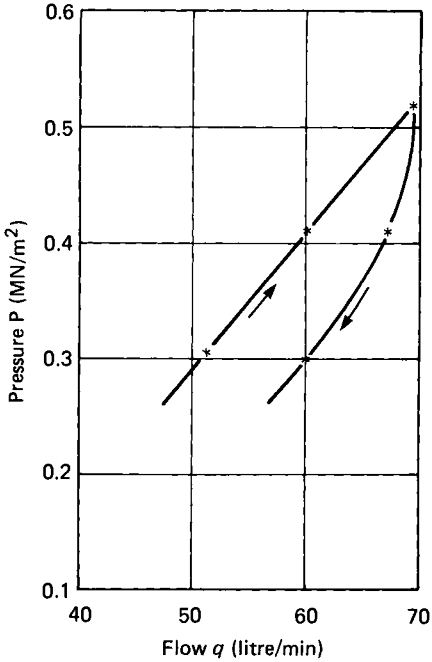


Figure 2.10 Flow pressure curve

flow rate is recorded both for increasing and decreasing pressure are presented in *Figure 2.11*. The results may not plot as a straight line. As the pressure of water increases within the hole so the fissures may open causing a non-linear increase in flow rate. The final analysis of the results depends upon the position of the groundwater level. If this is not already known a piezometer must be installed so that the groundwater level can be accurately determined.

The evaluation of the 'permeability' from packer tests is normally based upon methods using a relationship of the form[21]

$$k=\frac{q}{C_S RH} \tag{2.40}$$

where q is the steady flow rate under an effective applied head H (corrected for friction losses), R is the radius of the drillhole and C_S is a constant depending upon the length and diameter of the test section.

Bliss and Rushton[22] showed that equilibrium conditions in packer tests in fissured rocks are attained within a few minutes. On the other hand, where the hydraulic conductivity is very low this may take an hour or so.

Field pumping tests allow the determination of the coefficients of permeability and storage, as well as the transmissivity, for a larger mass of ground than the aforementioned tests. Pumping tests are described in Chapter 7.

The slug test involves injecting into or abstracting a known volume of water from a well and was used for assessing the transmissivity and storativity of confined aquifers by Cooper *et al.*[23]. Immediately after injection or abstraction of a slug of water, the water level in the well has an elevation, h_0, above or below that of the initial water table. As the level of the water rises or falls in the well, the difference, h, in the elevation between that at the original head and that at a given time, t, is measured. The ratio of the measured head to that of the subsequent head (h/h_0) is plotted against time on semi-

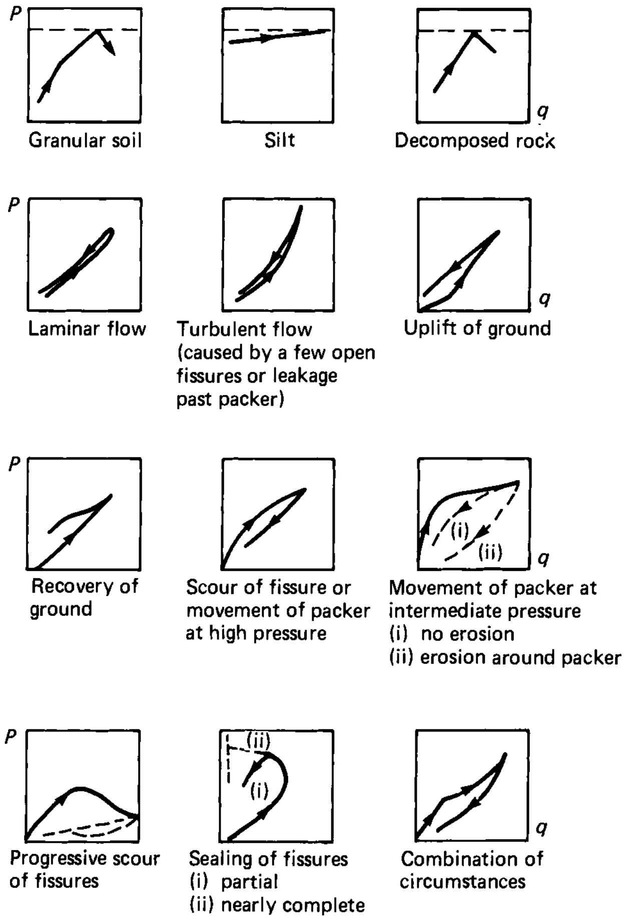


Figure 2.11 Typical pressure, P, against flow, q, curves for packer tests

logarithmic paper, the latter being plotted on the logarithmic scale. This curve is drawn at the same scale as that of a series of standard curves, above which it is overlain and fitted to a type curve (μ) which has the same curvature (*Figure 2.12*). The vertical time axis, t_1, which corresponds with the vertical axis for $Tt/r_c^2 = 1.0$ is selected. The transmissivity, T, is obtained from

$$T = \frac{1.0 r_c^2}{t_1} \tag{2.41}$$

where r_c is the radius of the well casing. The value of the storage coefficient or storativity, S, is found from

$$S = (r_c^2 \mu)/r_s^2 \tag{2.42}$$

where μ is the value of the μ curve for the field data and r_s is the radius of the well screen.

Bouwer and Rice[24] developed a slug test procedure for use in unconfined aquifers. As an alternative to using a slug of water, Black[25] suggested the use of a bailer of known volume which could be lowered down the well into the water. It is then quickly lifted out of the water and the change in level is recorded. Previously Skibitzke[26] had proposed a method for determining the transmissivity, T, from the recovery of the

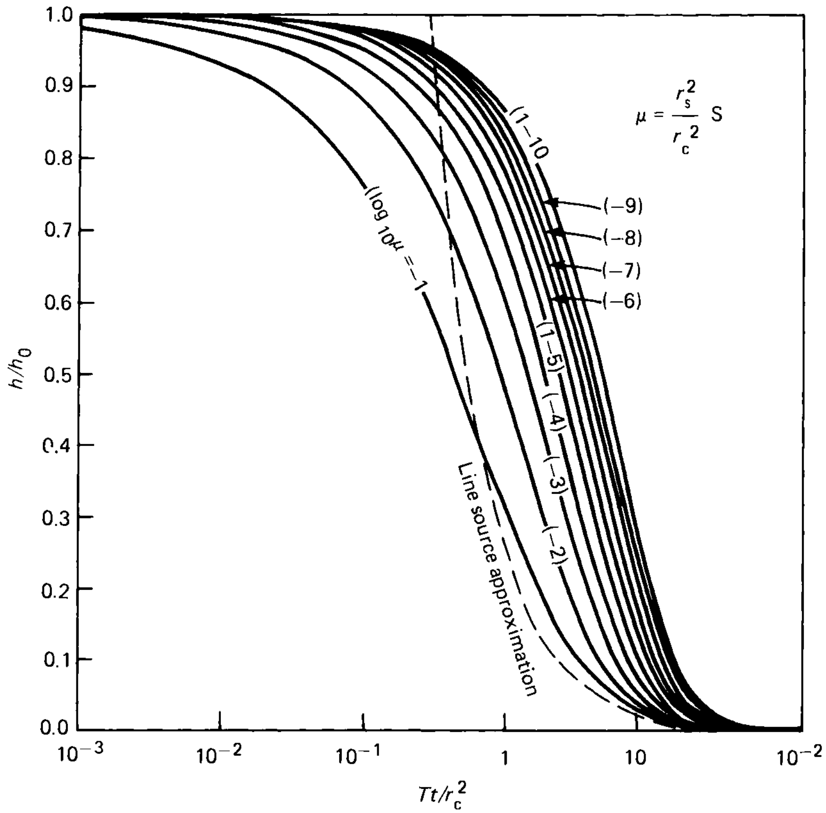


Figure 2.12 Type curves for slug test in a well of finite diameter (from Fetter, C. W., *Applied Hydrogeology*, Merrill Pub. Co., Columbus, Ohio (1980))

water level in a well that had been bailed. At any given point on the recovery curve, plotted from measurements taken during the test,

$$T=\frac{V_w}{4\pi s' t} \tag{2.43}$$

in which V_w is the volume of water removed in one bailing cycle, s' is the residual drawdown and t is the time elapsed since bailing ceased. If the residual drawdown is observed after several cycles of bailing, then the above equation becomes

$$T=\frac{1}{4\pi s'}\left[\frac{V_{w_1}}{t_1}+\frac{V_{w_2}}{t_2}+\frac{V_{w_3}}{t_3}+\cdots+\frac{V_{w_n}}{t_n}\right] \tag{2.44}$$

Other tests for assessing the *in situ* permeability and which are similar to the slug test, are the auger hole test and the piezometer method[27].

2.5.3 Measurement of water level and water pressure

Measurement of the level of the water in a borehole or drillhole is one of the most important down-the-hole measurements. A steel tape gives accurate results when a well is not being pumped. Electrical water level probes and air-lines are used to measure water levels in pumping wells.

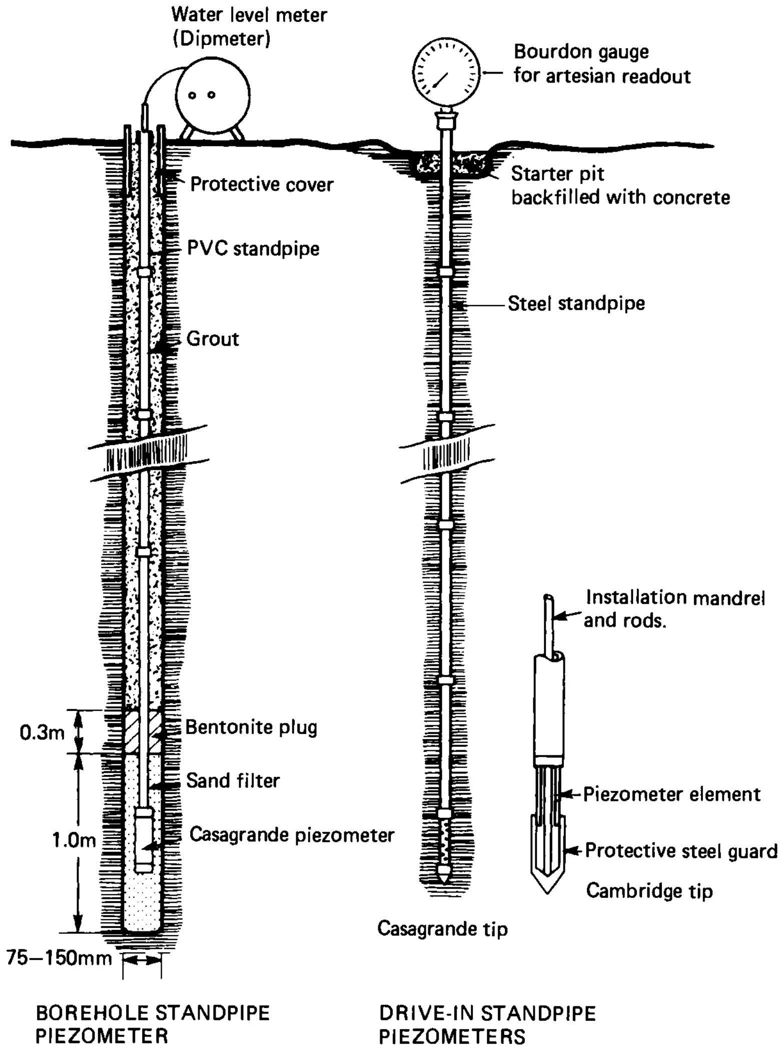


Figure 2.13 Standard piezometers (Courtesy of Soil Instruments Ltd)

The position of the water table is readily determined in homogeneous, relatively permeable ground, the hydrostatic head increasing at a linear rate below this level. A stand-pipe piezometer consisting of a perforated tube placed in a borehole or drillhole and surrounded by gravel to near the surface, with a seal at the top of 1 m to keep out surface drainage, records the upper free water surface (*Figure 2.13*). Other types of piezometers are used to measure water pressures. They are sealed into each zone where it is required to assess the water pressure. The installation of a piezometer must be carefully carried out otherwise its performance and measurements may be seriously affected. If more than one piezometer is installed in a borehole the likelihood of failure increases. After installation the ability of a piezometer to function should be checked by topping up and it should be observed daily until an apparent equilibrium is attained. Thereafter the intervals between observations can be lengthened to a week or a month.

2.6 Assessment of flow in the field

2.6.1 Flowmeters

A flowmeter log provides a record of the direction and velocity of groundwater movement in a drillhole. Flowmeter logging requires the use of a velocity-sensitive instrument, a system for lowering the instrument into the hole, a depth measuring device to determine the position of the flowmeter and a recorder located at the surface[28].

The direction of flow of water is determined by slowly lowering and raising the flowmeter through a section of hole, 6 to 9 m in length and recording velocity measurements during both traverses. If the velocity measured is greater during the downward than the upward traverse, then the direction of flow is upward and vice versa.

A flowmeter log made while a drillhole is being pumped at a moderate rate or by allowing water to flow if there is sufficient artesian head, permits identification of the zones contributing to the discharge. It also provides information on the thickness of these zones and the relative yield at that discharge rate. Because the yield varies approximately directly with the drawdown of water level in the well, flowmeter logs made by pumping, should be pumped at at least three different rates. The drawdown of water level should be recorded for each rate.

Patten and Bennett[29] discussed the use of flowmeters and referred to a thermal flowmeter which gave accurate, reliable measurements which could be taken rapidly. Measurement with the meter in a fixed position was preferable to continuous logging. In addition its small size made it easier to use than other flowmeters. They found that the flowmeter was unsatisfactory for measurement of slow flows and was unable to take a rapid series of flow measurements. Also it is generally imposible to log continuously with the instrument as each measurement had to be made in a fixed position.

A low velocity flowmeter can be used to measure the vertical flow in a drillhole. For example, Bruzzi *et al.*[30] used a flowmeter which was almost entirely insensitive to horizontal flow effects and had a minimum measurable velocity of about 0.005 m/s, for measuring the flows in boreholes in alluvial ground. The boreholes were cased, casings consisting of plastic with a series of large diametral slots lined with 0.5 mm tubular filter. Readings were taken at every half metre of depth which enabled the velocity curves to be checked with the stratigraphical log.

Use of the axial viewing head on television equipment enables the effects of horizontal or inclined flow to be observed directly in a drillhole. The flow movement is indicated by streamers suspended from beneath the television camera. Not only are they observed but they can be recorded on film or videotape. Also visible tracers allow assessment of flow direction to be made.

2.6.2 Tracers

A number of different types of tracer have been used to investigate the movement of groundwater and the interconnection between surface and groundwater resources. The ideal tracer should be easy to detect quantitatively in minute concentrations, it should not change the hydraulic characteristics of, or be adsorbed by the media through which it is flowing, it should be more or less absent from, and should not react with the groundwater concerned and it should have a low toxicity. The type of tracers in use include water soluble dyes which can be detected by colorimetry; sodium chloride or sulphate salts which can be detected chemically and strong electrolytes which can be detected by electrical conductivity. Radioactive tracers are also used and one of their advantages is that they can be detected in minute quantities in water[31]. Such tracers should have a useful half-life and should present the minimum of hazard. For example, tritium is not the best of tracers because of its relatively long half-life. In addition because it is introduced as tritiated water it is preferentially adsorbed by montmorillonite. Some of the more frequently used types of tracer are as follows

Chemical	*Colorimetric*	*Nuclear*	*Stable isotopes*
Copper sulphate	Sodium fluorescein	Bromide—82	Deuterium
Sodium iodide	Methylene blue	Chromium—51	Helium—4
Dextrose		Cobalt—60	Oxygen—18
		Iodine—131	
		Phosphorus—32	
		Rubidium—86	
		Tritium	

When a tracer is injected via a drillhole into groundwater it is subject to diffusion, dispersion, dilution and adsorption. Dispersion is a result of very small variations in the velocity of laminar flow through porous media. Molecular diffusion is probably negligible unless the velocity of flow is unusually low. Even if these processes are not significant, flow through an aquifer may be stratified or concentrated along discontinuities. Therefore, a tracer may remain undetected unless the observation drillholes intersect these discontinuities.

The vertical velocity of water movement in a drillhole can be assessed by using tracers. A tracer is injected at the required depth and the direction and rate of movement is monitored by a probe[32]. For instance, Ineson and Gray[33] determined the velocity of groundwater flow in a drillhole by recording the rate of movement of the peak of a saline slug, which had been injected, with an electrical conductivity probe. Changes in the form of the electrical conductivity profile indicate variations in the pattern of groundwater flow, due to inflow or outflow from surroudnding rocks. From a study of the differences between the original and superimposed profiles, quantitative assessment of loss from, or inflow of groundwater into, the well can sometimes be derived.

However, Patten and Bennett[29] maintained that brine tracing is a more troublesome means of monitoring flow than investigating it with a flowmeter. Furthermore, the technique is not exact enough to take measurements at precise depths within a drillhole, cannot produce satisfactory results when used where water is entering the drillhole and does not allow a number of measurements to be taken in rapid succession. Nonetheless, Patten and Bennett noted that a relatively high degree of accuracy can be achieved when a significant length of drillhole is used for measurement and that brine tracing can indicate the direction of flow at velocities which are too low to be recorded by a flowmeter.

Determination of the hydraulic conductivity in the field can be done by measuring the time it takes for a tracer to move between two test holes. Like pumping tests, this tracer technique is based on the assumption that the aquifer is homogeneous and that observations taken radially at the same distance from the well are comparable[34]. This method of assessing hydraulic conductivity requires that injection and observation wells are close together, to avoid excessive travel time and that the direction of flow is known so that observation holes are correctly sited. Several observation holes improve the chances of the tracer flowing into one of them but increase costs. Since the tracer flows through the aquifer with an average interstitial velocity, $\bar{v}$, then

$$\bar{v}=\frac{k}{n}\frac{h}{L} \tag{2.45}$$

where k is the hydraulic conductivity, n is the porosity, h is the difference in height between water levels in the test holes and L is the distance between them. However, $\bar{v}$ can also be obtained from

$$\bar{v}=\frac{L}{t} \tag{2.46}$$

where t is the time of travel over the distance involved, hence

$$k=\frac{nL^2}{ht} \tag{2.47}$$

In the point dilution method a tracer is introduced into the test hole and thoroughly mixed with the water therein. As water flows into and out of the well, measurements are taken of tracer concentration. Analysis of the resulting dilution curve provides an indication of the groundwater velocity. The velocity of groundwater also can be recorded by measuring the rate of dilution of a tracer in observation wells[35,36].

Hydrochemistry can be used to demonstrate the complexity of groundwater flow on a regional scale and to provide some appreciation of the magnitude of flow involved. For example, Sage and Lloyd[37] showed that low bicarbonate zones (i.e. less than 250 mg/l) in the Triassic Sandstone aquifer of north west Lancashire were related to the distribution of glacial sands in those cases where the sands merge with river courses at the eastern boundary of the sandstone. In other words, recharge of soft river water is taking place through the alluvium into the glacial sands. It is then conveyed over significant distances, remaining more or less unchanged, through the sand–sandstone aquifer. The harder bicarbonate water (i.e. greater than 350 mg/l) in the Triassic Sandstone is believed to have originated from aquifers in the Carboniferous system which occurs on the eastern boundary of the Triassic Sandstone. This indicates that hydraulic continuity exists between both these sets of aquifers. In the northern part of the area the hydrochemistry suggests that there is very limited groundwater flow across the coastal area, which is consistent with the presence of extensive thick marine clays which exist in Morecombe Bay.

Koerner *et al.*[38] reviewed various other methods for detecting groundwater seepage including temperature sensing, infrared sensing, microwave sensing, acoustic emission control and seismic and electrical methods.

2.7 Groundwater in igneous and metamorphic rocks

The porosities of igneous and metamorphic rocks are usually less than 1 per cent, also many of the pores may not be interconnected. Consequently for most practical purposes, their intact permeabilities can be regarded as zero. However, these rocks may be pervious due to the presence of joints, fissures and weathered zones. But in fresh rocks the fracture space is a small percentage of their volume and fractures tend to close with depth. Permeability therefore declines. Nevertheless, some openings must remain open at depths of hundreds or even thousands of metres, witness the ingress of water into some deep mines and tunnels. Well yields suggest that permeabilities consequent upon fractures in fresh rock, within a hundred or so metres of the surface, generally range between 1×10^{-8} to 1×10^{-4} m/s. This may increase to several centimetres per second in highly fractured zones.

Owing to the preferred orientation of water-bearing fractures, the permeability of metamorphic and plutonic igneous rocks is generally strongly anisotropic. The depth of weathering is usually less than 15 m, indeed in many terrains it is less than 1 m, but in regions of intense weathering such as the humid tropics it may extend over 100 m. The increase in bulk density consequent upon weathering increases porosity. In some cases the porosities of the weathered regoliths exceed 35 per cent. They decrease with depth as the material becomes less weathered. However, the highest permeabilities are found in partially decomposed rock where the development of clay minerals is not significant. Marble may contain solution cavities, developed along fractures, as a result of weathering.

Usually well yields are low in most plutonic igneous and metamorphic formations, over 90 per cent yielding less than about 165 000 l/day. Moreover, a large number of the wells sunk prove failures, in some areas the figure has been as high as 40 per cent. A common feature of wells drawing water from fracture systems is the high or moderate initial yield which decreases rapidly with time. Usually this is because of insufficient storage of groundwater near the well.

Volcanic rocks have a wide range of hydrogeological properties. For example, the porosity of volcanic rocks varies from less than 1 per cent in dense basalts to over 85 per cent in pumice, while vesicular rocks may have porosities ranging between 10 and 50 per cent. Porosity may be increased in some rocks by weathering. Volcanic sequences without interbedded sediments or pyroclasts have relatively low overall porosities if large volumes of rock are considered. Conversely interbedded pyroclasts and sediments greatly increase the overall porosity of thick sequences which are predominantly volcanic. Under favourable circumstances the pyroclasts and sediments provide storage space for water whereas the more permeable volcanic lavas transmit the water to wells. Permeability in lava flows is largely a function of their cooling joints, cavities, tunnels, pipes and vesicles, and the voids between successive flows. For instance, the porosity can be less than 5 per cent in volcanic rocks which are good to excellent aquifers.

The permeability of volcanic sequences is also anisotropic. Usually vertical permeability, which is largely attributable to cooling joints, is less than horizontal permeability. Flow in the latter case takes place mainly via the voids between individual lavas. Indeed vertical permeability in some regions is so low that separate confined aquifers may be present in the sequence. Average permeability may range from zero to over 1×10^{-2} m/s. Both porosity and permeability tend to decrease with increasing geological age, as the pore spaces in pyroclasts are reduced by consolidation and cementation or those in lavas are reduced by precipitation of secondary minerals.

Although transmissivities in recent basalts and andesites may be high, groundwater may be difficult to obtain since it may drain freely to points of discharge or its depth may be excessive[39,40,41]. Indeed the infiltration capacity of some recent volcanic terrains is so high that surface drainage is virtually absent. This is the case in some parts of the Hawaiian Islands. In such instances attention must be focused on less permeable zones, such as buried soil horizons or interbedded mud flows, which impede the loss of water and cause the water table to rise nearer the surface.

The porosities and permeabilities of pyroclasts are partly dependent on their size distribution and sorting but more importantly on their degree of induration. Welded tuffs and ignimbrites have low porosities and very low permeabilities whilst some recent coarse-grained ashes, which contain cinders, have high permeabilities. These, however, tend to weather rapidly to clayey soils, so reducing their permeability.

2.8 Groundwater in sedimentary rocks

Most fine-grained sedimentary rocks have relatively high porosities but very low permeabilities so that in practical terms they usually act as barriers to water movement. Nonetheless, their large porosity means that they can store vast quantities of water and this can drain slowly into aquifers. The porosity of argillaceous sediments tends to decrease with both depth and age, although the relationship is neither simple nor universal. Newly deposited muds, for instance, may have porosities ranging from 50 to 80 per cent and as these consolidate water is squeezed from them into more permeable beds.

As far as water supply is concerned the most important sedimentary rocks are sands, gravels, sandstones and limestones. The search for groundwater frequently starts with an investigation of unconsolidated sediments since the potential groundwater they contain is often close to the surface and so easily accessible. Such deposits are often near sources of recharge in the form of rivers and lakes. They tend to have high permeabilities and specific yields.

2.8.1 Sands and gravels

Sands and gravels are generally found in river[42], fluvio-glacial and aeolian deposits, as well as along coastal plains. The porosity of alluvial sands and gravel ranges from around 20 per cent in coarse, poorly sorted deposits to over 40 per cent in clean, uniformly sorted material. Most water bearing zones within alluvial deposits have permeabilities varying between 1×10^{-4} to 1×10^{-3} m/s, although values of over 5×10^{-3} m/s are not rare. Hence moderate yields of 40 to 230 l/min can be obtained from wells sunk in river deposits. Much larger yields can be expected where the permeable layers are thick. This is particularly the case where buried channels occur within river valleys.

Many fluvio-glacial deposits such as eskers, kames and outwash fans are composed of sands and gravels. However, some of these deposits form hillocks or occur on the slopes of hills and are therefore subject to rapid discharge, and are not recharged by streams. They therefore may contain no saturated zones. Also kames and eskers may be of limited extent. Although they may contain fewer fines, the hydrogeological characteristics of fluvio-glacial deposits and outwash fans in particular, are generally similar to those of alluvial deposits. Outwash deposits are consequently important aquifers in glaciated areas, for example, some wells in outwash deposits near Tacoma, United States, have yields of up to 9000 l/min.

Recent aeolian or wind-blown deposits are generally less common than alluvial or fluvio-glacial deposits. Usually wind-blown sands are uniformly sorted with porosities around 35 to 40 per cent and permeabilities between 5×10^{-5} and 5×10^{-4} m/s. The specific yield may be somewhere between 30 and 38 per cent. But wells are not usually sunk in dune sands as they tend to sediment up and drain rapidly. On the other hand, these sands can be quickly recharged.

Loess is characterized by vertical joints and rootholes, which mean that vertical permeability is much greater than horizontal permeability. Many loess deposits have an open structure which imparts porosities of 40 to 55 per cent. Specific yields may range from 15 per cent, for the fine-grained varieties, to 35 per cent for the coarse-grained types. The overall permeabilities vary from 1×10^{-9} m/s to 1×10^{-5} m/s. Accordingly loess is not generally used as an aquifer.

Sediments along coastal plains usually include both alluvial and marine deposits and in some areas deltas may be developed. The water-yielding properties of these sediments are, in general, quite similar to those of sediments in large valleys. Permeabilities in the water-bearing zones commonly vary between 1×10^{-5} and 1×10^{-3} m/s and the sand and gravel deposits may have specific yields between 15 and 35 per cent. Yields from some large wells may be as high as 12 000 l/min. Indeed in the United States the coastal plain deposits along the Gulf of Mexico contain the most important groundwater reserves.

2.8.2 Sandstones

The porosity of sandstones ranges up to a maximum of about 30 per cent, the most important factor in this respect being the amount of matrix and/or cement in the pores. Diagenetic processes can give rise to an irregular distribution of cement within sandstone. Generally speaking the older a sandstone, the more well cemented it is, for example, most quartz arenites of Cambrian age have extremely low porosities. Hence the permeabilities of sandstones can vary appreciably, from less than 1×10^{-9} to over 5×10^{-6} m/s. Wells sunk into sandstones usually have yields between 25 and 900 l/min. Firmly cemented sandstones may transmit water to wells via their discontinuities.

The Bunter Sandstone is the second most important aquifer in the UK. It generally has a high permeability so that the wells are capable of large supplies.

The saturated thickness of the Bunter Sandstone aquifer in north west Lancashire is generally greater than 120 m which is adequate to permit a prolific borehole yield where hydraulic conductivity is high[43]. However, the texture of the sandstone varies markedly from one location to another and pumping tests have indicated a wide range of formation characteristics with productive and uneconomic yields in all areas. Worthington[44] maintained that the calculated permeabilities from resistivity logging of the Bunter Sandstone in north west Lancashire agreed with the corresponding values deduced from pumping tests. This implies that much of the groundwater flow through the Bunter Sandstone in north west Lancashire is intergranular. However, this is not generally valid for Permo-Triassic sandstones as fissure flow has been shown to be a more significant feature of groundwater movement[45,46,47]. Moreover, Brereton and Skinner[48] maintain that the horizontal intergranular permeability of the Triassic Sandstone in the Fylde area is lower than the bulk aquifer permeability as determined by pumping tests by a factor of 17. Such a discrepancy can only be explained by a dominant fissure-flow component.

2.8.3 Limestone and chalk

The original porosity is relatively high in most young limestones. In older limestones the porosities may range up to 15 per cent. Secondary dolomitization of limestones involves increases in the porosity, a 13 per cent volume reduction being caused by the transformation of calcite to dolomite. These pores may be subsequently enlarged by dissolution. The permeabilities of limestones range from 1×10^{-6} to 5×10^{-4} m/s although that of dense crystalline limestone is generally less than 1×10^{-8} m/s. Dissolution and consequent enlarging of joints and bedding planes is much more significant than porosity as far as the permeability of limestone is concerned. Limestones with extensive solution channels or fractures developed in one direction have permeabilities that are strongly anisotropic. Yields from wells in limestone are commonly between 25 and 90 l/min although some may exceed 9000 l/min.

The chances of encountering large water-filled caverns in karstic terrains when sinking a well are quite small, although vast quantities of water are stored in such regions and the output of a well should be adequate. In terms of water supply, reef limestones and shelly limestones, especially those in which the original porosity has been increased as a result of solution, are often very productive.

According to Reeves *et al.*[49], a wide range of permeabilities is encountered in the Corallian Limestone aquifer of the Vale of Pickering, Yorkshire, indicating that fissure flow is the important controlling factor in groundwater movement. The importance of fissure flow is also indicated by the fact that the hydraulic conductivity observed in the field is substantially higher than values obtained from laboratory tests. What is more, the results of caliper, differential temperature and flowmeter tests indicate that the inflow of water into drillholes takes place at specific fissured horizons.

The bulk of groundwater discharge from the Corallian aquifer in the Vale of Pickering takes place from a series of large springs, the positions of which are governed by faulting. The springs generally appear where the Corallian aquifer passes beneath the impermeable clay cover of the central area of the Vale and in some instances they break through the clay along the line of a fault. The discharge at the primary springs represents a large proportion of the total discharge from the aquifer. Obviously major flow paths through the aquifer towards these springs carry most groundwater flow. Consequently fissures have been enlarged and the transmissivity of the limestone has been increased considerably in the areas around the major springs due to dissolution of the rock.

The Lincolnshire Limestone represents a carbonate aquifer some 20 to 30 m in thickness, the lithology of which varies both laterally and vertically[50]. The lower half is a cemented oolitic limestone with calcarenite facies and the upper half comprises a more massive limestone. Groundwater movement is almost entirely by fissure flow along well-developed bedding planes and joints. Transmissivities are high, usually exceeding 1000 m^2/day, although they are sometimes much higher. Total porosity values vary between 13 and 18 per cent with an exceedingly low intergranular permeability (less than 6.95×10^{-9} m/s). A significant quantity of run-off enters the aquifer via swallow holes.

The Chalk is the most important aquifer in Britain supplying over 40 per cent of the groundwater abstracted. It accounts for about 15 per cent of the potable water supply. Because of its high porosity (it may be as high as 50 per cent) the Chalk contains large volumes of water. For instance, each cubic kilometre of saturated Upper Chalk averages some 300 000 to 400 000 Ml of interstitial water and, assuming a maximum specific yield of 2 per cent up to 20 000 Ml of water is held in fissures.

However, since the pores are small the mass of the Chalk is characterized by low permeability, except where fissured[51]. These pores in the Chalk are so small (100 to 0.012 μm) that nearly all the water which they contain is held by capillary or molecular forces so that very little can drain under gravity[52]. Indeed drainage would only take place under suctions of the order of three atmospheres. Thus the Chalk has a very high specific retention. As a consequence, water held in the pores does not contribute directly to specific yield, so that not more than 3 per cent of the pores represent potentially useful storage. In the unsaturated zone evaporation and transpiration are the only processes which can substantially deplete the groundwater content.

As the incidence of fissures increases, the Chalk becomes a highly permeable aquifer. Steeply inclined major joints usually occur a metre or so apart in the Chalk, although evidence suggests that they may be tightly closed at shallow depth. Nonetheless, if each major joint has an effective opening of only 0.1 mm then the Chalk would develop a vertical permeability of 5×10^{-8} m/s which is 500 times greater than that attributable to intergranular permeability.

Smith *et al.*[53] concluded that 85 per cent of the total downward movement of water in the Chalk of the Berkshire Downs was due to intergranular seepage, at a net flow rate of 0.88 m/year. However, the existence of a dominantly slow-transit intergranular flow regime in the unsaturated zone is not easy to reconcile with the known hydraulic properties of the Chalk. Foster and Crease[54], for example, maintained that the Chalk of east Yorkshire is a fissure-flow, and largely fissure-storage, formation, with horizontal permeability in the active flow zones, in the range 2×10^{-5} to 2×10^{-6} m/s. They suggested that under natural hydraulic gradients the velocities of throughflow to discharge areas vary from 1×10^{-5} to 5×10^{-5} m/s. Toynton[55] maintained that the movement of water through the Chalk of Norfolk was primarily by fissure flow. Indeed, values of transmissivity were found to vary considerably with the orientation of the local discontinuity pattern.

Abstraction of water, from the Chalk generally takes place from wells sunk from 50 to 150 m in depth, depending on the overlying deposits. In areas of outcrop, if the water table is high enough, the best results are generally obtained from the first 60 m of saturated material irrespective of the division of the Chalk exposed. A common practice in the Upper Chalk of south east England is to sink a well and drive adits from it in an attempt to intercept the maximum number of water-bearing joints. In favourable conditions yields in excess of 9000 l/min have been obtained by using this method. Experience has shown, however, that a number of smaller diameter wells, strategically located, may be more successful than an extensive adit system. Acidification of wells is also used to increase the yield.

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Chapter 3

Groundwater exploration

A groundwater investigation requires a thorough appreciation of the hydrology and geology of the area concerned and a groundwater inventry needs to determine possible gains and losses affecting the subsurface reservoir. Of particular interest is the information concerning the lithology, stratigraphical sequence and geological structure, as well as the hydrogeological characteristics of the subsurface materials. Also of importance are the positions of the water table and piezometric level and their fluctuations.

In major groundwater investigations, records of precipitation, temperatures, wind movement, evaporation and humidity may provide essential or useful supplementary information. Similarly data relating to steamflow may be of value in helping to solve the groundwater equation since seepage into or from streams constitutes a major factor in the discharge or recharge of groundwater. The chemical and bacterial qualities of groundwater obviously require investigation. These factors are dealt with elsewhere.

Essentially an assessment of groundwater resources involves the location of potential aquifers within economic drilling depths. Whether or not an aquifer will be able to supply the required amount of water depends on its thickness and spatial distribution, its porosity and permeability, whether it is fully or partially saturated and whether or not the quality of the water is acceptable. Another factor which has to be considered is pumping lift and the effect of drawdown upon it.

3.1 The desk study and preliminary reconnaissance

The desk study involves a consideration of the hydrological, geological, hydrogeological and geophysical data available concerning the area in question. Particular attention should be given to assessing the lateral and vertical extent of any potential aquifers, to their continuity and structure, to any possible variations in formation characteristics and to possible areas of recharge and discharge. Additional information relating to groundwater chemistry; the outflow of springs and surface run-off, data from pumping tests, from mine workings, from waterworks, or meteorological data, should be considered. Information on vegetative cover, land utilization, topography and drainage pattern can prove of value at times.

Hence the desk study involves a survey of the relevant topographical and geological maps and memoirs and possibly aerial photographs, as well as any pertinent literature.

Sources of available information in the UK have been summarized by Dumbleton and West[1] and Chaplow[2]. The data obtained during the desk study should help in the planning of the project. Unfortunately, in some parts of the world, little or no literature, or maps are available. In the UK the Ordnance Survey supply topographical, soil and geological maps. Memoirs accompany many of the standard geological maps and these provide a detailed survey of the geology of the area in question, as well as recording drillhole logs.

A geological map can be used to indicate those rocks which should be investigated as potential sources of water. Some idea of the dimensions, continuity and geological structure of potential aquifers and the depth at which they occur can be obtained from maps and sections. Perhaps some indication of the quality of the water may be gleaned from the type of rock forming the aquifer. Soil maps and reports supply information on soil characteristics and surface gradients which influence run-off and infiltration.

The amounts of useful information which can be obtained from aerial photographs varies with the nature of the terrain and the type and quality of the photographs. Aerial photographs may prove of value at the feasibility stage of a project. They provide a relatively inexpensive method of obtaining information for an initial appraisal of a large area and they sometimes reveal features which cannot easily be detected from the ground. Aerial photographs can be taken rapidly for any locality and in the UK several concerns[3] keep extensive collections of airphotos.

The preliminary reconnaissance involves a walk over the site noting, where possible, the distribution of the soil and rock types present, the relief of the ground, the surface drainage and associated features, ground cover, etc. On such occasions simple tools such as the geological hammer, clinometer and auger should be taken along. The inspection should not be restricted to the site but should examine adjacent areas to see how they affect the site in question. The importance of the preliminary investigation is that it should assess the suitability of the site and if it is suitable, the data obtained will form the basis upon which the subsequent exploration is planned. The preliminary reconnaissance also allows a check to be made on any conclusions reached in the desk study.

3.2 Remote sensing imagery and aerial photographs

Remote sensing commonly represents one of the first stages of land assessment in underdeveloped areas and can be used for the purposes of regional mapping, but it does not provide the same amount of detail as aerial photographs. It involves the identification and analysis of phenomena on the Earth's surface by using devices borne by aircraft or spacecraft. In this way remote sensing imagery can yield information on recharge areas relating to the supply of groundwater reservoirs.

Most techniques used in remote sensing depend upon recording energy from part of the electromagnetic spectrum, ranging from gamma rays, through the visible spectrum to radar. The two principal systems of remote sensing are infrared linescan (IRLS) and side-looking airborne radar (SLAR). The scanning equipment used measures both emitted and reflected radiation and the employment of suitable detectors and filters permits the measurement of certain spectral bands. Signals from several bands of the spectrum can be recorded simultaneously by multi-spectral scanners. Lasers also are being developed for use in remote sensing.

Infrared linescanning is dependent upon the fact that all objects emit electromagnetic radiation generated by the thermal activity of their component atoms[4].

Photoelectrical detectors and optical mechanical scanners are used to record images in the thermal infrared spectral region. The data can be processed in colour as well as black and white, colours substituting for grey tones. Cool areas are depicted as purple and warm areas as red on the positive print, whereas warm areas are shown as black and cool as white on the black and white negative film base[5].

Identification of grey tones is the most important aspect as far as the interpretation of thermal imagery is concerned. Thermal inertia is important in this respect since rocks with high thermal inertia, such as dolostone or quartzite, are relatively cool during the day and warm at night. Rocks and soils with low thermal inertia, for example, shale, gravel or sand, are warm during the day and cool at night. In other words the variation in temperature of materials with high thermal inertia during the daily cycle is much less than those with low thermal inertia. Because clay soils possess relatively high thermal inertia they appear warm in pre-dawn imagery whereas sandy soils, because of their relatively low thermal inertia, appear cool. The moisture content of a soil influences the image produced, that is, soils which possess high moisture contents appear cool, irrespective of their type.

Texture can also help interpretation. For instance, outcrops of rock may have a rough texture due to the presence of bedding or jointing, whereas soils usually give rise to a relatively smooth texture. However, where soil cover is less than 0.5 m, the rock structure is usually observable on the imagery since deeper, more moist soil occupying discontinuities gives a darker signature. Free standing bodies of water are usually readily visible on thermal imagery, however, the high thermal inertia of highly saturated organic deposits may approach that of water masses, it may therefore sometimes prove difficult to distinguish between the two.

In side-looking airborne radar, short pulses of energy, in a selected part of the radar waveband, are transmitted to the ground from an aircraft[6] and are reflected back at given intervals. The reflected pulses are transformed into black and white photographs. Typical scales for radar imagery available commercially are 1:100 000 to 1:250 000, with a resolution of between 10 and 30 m. Smaller objects than this can appear on the image if they are strong reflectors and the original material can be enlarged. Mosaics are suitable for the identification of regional geological features and for preliminary identification of terrain units. Lateral overlap of radar cover can give a stereoscopic image, which offers a more reliable assessment of the terrain. Furthermore, imagery recorded by radar systems can provide appreciable detail of landforms, as they are revealed due to the low angle of incident illumination.

Beaumont and Beavan[7] noted that imagery of the Earth's surface obtained from space gives a broad view of an area, illustrating conditions as they exist at a particular time, and indicates the interrelationships between geology, landform, climate, vegetation and land-use. Small-scale space imagery provides a means of initial reconnaissance which allows areas to be selected for further, more detailed investigation, either by aerial and/or ground survey methods. Indeed in many parts of the world a Landsat image may provide the only form of base map available.

The capacity to detect surface features and landforms from imagery obtained by multispectral scanners on Landsat satellites is facilitated by energy reflected from the ground surface being recorded within specific wavelength bands[8]. If reflected energy from the shorter and longer ends of the visible spectrumare recorded separately, differentiation between rock types can be achieved. The ability to distinguish between different materials increases when imagery is recorded by different sensors outside the visible spectrum, that is, in the infrared wavelength bands.

In addition to the standard photographs at a scale 1:1 000 000, both transparencies (positive and negative) and enlargements, at scales of 1:250 000 and 1:500 000, are available, as are false colour composites. The latter often show up features not easily observable on black and white images.

Landsat images may be interpreted in a similar manner to aerial photographs, although the images do not come in stereopairs. Nevertheless, a pseudostereoscopic effect may be obtained by viewing two different spectral bands (band-lap stereo) of the same image or by examining images of the same view taken at different times (time-lap stereo). There is also a certain amount of side-lap, which improves with latitude. This provides a true stereoscopic image across a restricted strip of a print; however, significant effects are only produced by large relief features.

Interpretation of aerial photographs may aid recognition of broad rock and soil types and thereby help locate potential aquifers. The combination of topographical and geological data may help identify areas of likely groundwater recharge and discharge. In particular, the nature and extent of superficial deposits may provide some indication of the distribution of areas of recharge and discharge. Aerial photographs allow the occurrence of springs to be recorded.

Aerial photographs may be combined in order to cover large regions. The simplest type of combination is the uncontrolled print laydown which consists of photographs, laid along side each other, which have not been accurately fitted into a surveyed grid. Photomosaics represent a more elaborate type of print laydown, requiring more care in their production and controlled photomosaics are based on a number of geodetically surveyed points. They can be regarded as having the same accuracy as topographic maps.

There are four main types of film used in normal aerial photography, namely, black and white, infrared monochrome, true colour and false colour. Black and white film is used for topographic survey work and for normal interpretation purposes. The other types of film are used for special purposes. For example, infrared monochrome film makes use of the fact that near infrared radiation is strongly absorbed by water. Accordingly it is of particular value when mapping the presence of water on land as, for instance, at shallow depths underground. True colour photography displays variations of hue, value and chroma, rather than tone only and generally offers much more refined imagery. Consequently colour photographs have an advantage over black and white ones as far as photogeological interpretation is concerned, in that there are more subtle changes in colour in the former than in grey tones in the latter, hence they record more geological information. False colour is the term frequently used for infrared colour photography, since on reversal positive film green, red and infrared light are recorded respectively as blue, green and red. Water is portrayed as blue, green vegetation as magenta (a mixture of blue and red), and rocks as blue, green or red, mixed in the same proportions as they reflect green, red and infrared light respectively. False colour provides a more sensitive means of identifying exposures of bare grey rocks than any other type of film.

Variations in water content in soils and rocks which may not be readily apparent on black and white photographs are often clearly depicted by false colour. In fact, the specific heat of water is usually two to ten times greater than that of most rocks and this, therefore, facilitates its detection in the ground. Indeed, the specific heat of water can cause an aquifer to act as a heat sink which, in turn, influences near-surface temperatures.

Stereoscopic examination of consecutive pairs of aerial photographs allows observation of a three-dimensional image of the ground surface[9]. Allum[10] pointed out that when stereopairs of aerial photographs are observed, the image perceived represents a combination of variations in both relief and tone. However, relief and tone on aerial photographs are not absolute quantities for particular rocks. For instance, relief represents the relative resistance of rocks to erosion as well as the amount of erosion which has occurred. Tone is important since small variations may be indicative of different types of rock. Unfortunately tone is affected by light conditions, which vary with weather, time of day and season and processing. Nevertheless, basic intrusions commonly produce darker tones than acid intrusions. Quartzite, quartz schist, limestone, chalk and sandstone tend to give light tones, whilst slates, micaceous schists, mudstones and shales give medium tones and amphibolites give darker tones. Normally, only general rather than specific rock types are recognizable from aerial photographs; for example, superficial deposits, sedimentary rocks, metamorphic rocks, intrusive rocks and extrusive rocks. Regional geological structures are frequently easier to recognize on aerial photographs which provide a broad synoptic view, than they are in the field.

Because the occurrence of groundwater is much influenced by the nature of the ground surface, aerial photographs can yield useful information. Also the vegetative cover may be identifiable from aerial photographs and as such may provide some clue as to the occurrence of groundwater. In arid and semi-arid regions, in particular, the presence of phreatophytes, that is, plants which have a high transpiration capacity and derive water directly from the water table, indicates that the water table is near the surface[11]. By contrast, xerophytes can exist at low moisture contents in the soil and their presence would suggest that the water table was at an appreciable depth. Halophytes are plants which can exist where soluble salts occur in the soil moisture and indicate the occurrence of brackish or saline groundwater. Most types of pheatophytes utilize water which is potable, although some may be found tapping water which is saline.

Howe *et al.*[12] described the production of groundwater prediction maps made from aerial photographs. They recognized three types of area relating to potential water-bearing formations. For example, the most important class contained water-bearing formations such as granular materials in alluvial plains, terraces, outwash deposits, morainic deposits and sandy lake beds. Such regional groundwater maps demonstrate the general groundwater conditions, thus indicating the areas most (and least) favourable for prospecting for water. Hence, such maps help to locate the sites of test wells, thereby reducing the cost of prospecting.

3.3 Field exploration

The aim of an exploration programme is to try to determine the nature of the ground conditions, that is, field exploration involves delineating the spatial distribution depth, thickness and potential yield of aquifers. This involves both surface and subsurface investigations[13]. Exploration techniques used for the assessment of groundwater supply are largely governed by the local geological conditions. The latter must be understood in order to quantify the resource.

Geological mapping frequently forms the initial phase of exploration and should identify potential aquifers such as sandstones and limestones and distinguish them from aquicludes. Argillaceous rocks represent the most common aquicludes and constitute hydrogeological barriers in sedimentary sequences. The situation is further complicated where facies changes take place in a horizontal direction. Furthermore, geological mapping should accurately locate igneous intrusions and major faults.

Large intrusions can have a notable influence on the pattern of groundwater movement. Fault zones may be either highly permeable or may act as barriers to groundwater flow, depending upon the type of material occupying the fault zone. For example, in the Vale of Pickering, Yorkshire, faulting has either completely or partially reduced hydraulic conductivity between the confined part of the Corallian Limestone and its outcrop. Indeed Reeves *et al.*[14] found that faults formed a framework for the division of the aquifer into five hydrogeological zones. In addition it is important during the mapping programme to establish the geological structure of an area in which a hydrogeological survey is carried out[15].

Superficial deposits may perform a confining function in relation to the major aquifers (for example, the Chalk[16]) they overlie, or because of their lithology they may play an important role in controlling recharge to major aquifers. For instance, Sage and Lloyd[17] showed that the glacial sands of north west Lancashire are in hydraulic conductivity with the sandstone aquifer of Triassic age.

Lithology usually is the most important factor governing the distribution of groundwater in sedimentary rocks. Sherrell[18], for instance, pointed out that lateral changes in lithology played a dominant role in the yield of groundwater from the Bunter Sandstone of south east England. However, this is not always the case. For example, it is not so in limestone in which the development of the aquifer is a function of its exposure to groundwater flow. In such a situation the boundaries of the aquifer may not correspond with the stratigraphic units present. Nonetheless, detailed lithological descriptions facilitate correlation between borehole logs.

3.3.1 Subsurface exploration in soils

The main purpose of a subsurface exploration is to determine the character of the geology. In the UK the most commonly used rig for investigating soils is the light cable and tool-boring rig (*Figure 3.1*). The hole is sunk by repeatedly dropping one of the various tools into the ground. A power winch is used to lift the tool. By releasing the clutch of the winch the tool drops and cuts into the soil. Once a hole is established it is usually lined with casing, the drop tool operating within the casing. This type of rig is usually capable of penetrating about 60 m of soil, in doing so the size of the casing in the lower end of the borehole must be reduced. The basic tools are the shell and claycutter, which are essentially open-ended steel tubes, to which cutting shoes are attached. The shell, which is used in cohesionless soils, carries a flap valve at its lower end which prevents the material from falling out on withdrawal from the borehole. The clay is retained by its adhesion to the walls of the claycutter.

Because of the mode of operation of the shell, the borehole should be kept full of water so that the shell may operate efficiently. Since most cohesionless soils in the UK are water bearing, all that is necessary is for the water in the borehole to be topped up. If flow of water occurs, then it should be from the borehole to the surrounding soil. If water is allowed to flow into the borehole, 'piping' will probably occur. Provided the head of water in the borehole is greater than the natural head, piping can usually be prevented. To overcome artesian conditions the casing should be extended above ground and kept filled with water. According to Dixon and Clarke[19], the shell cannot generally be used in highly permeable, coarse gravels since it is usually impossible to maintain a head of water in the borehole. Fortunately, these conditions usually occur at, or near ground level and the problem can sometimes be overcome by using an excavator to open a pit either to the water table or to a depth of 3 to 4 m. Casing can then be inserted, the pit backfilled and boring can then proceed. Another method of

Figure 3.1 Light cable and tool rig (courtesy of Pilcon Engineering)

penetrating gravels and cobbles above the water table is to employ a special grab with a heavy tripod and winch and casing of 400 mm diameter or greater.

For boring in stiff clays the weight of the claycutter may be increased by adding a sinker bar. In very stiff clays a little water is often added to assist boring progress. Furthermore, in such clays the borehole can often be advanced without lining, except for a short length at the top to keep the hole stable. If cobbles or small boulders are encountered in clays, particularly tills, then these can be broken by using heavy chisels. When boring in soft clays, although the hole may not collapse it tends to squeeze inwards and to prevent the cutter operating, the hole must therefore be lined.

As far as soils are concerned samples may be divided into two types, disturbed and undisturbed. Disturbed samples can be obtained from the claycutter or shell of a boring rig. Undisturbed samples are obtained by a variety of samplers.

In the wash boring method the hole is advanced by a combination of chopping and jetting the soil or rock, the cuttings thereby produced being washed from the hole by the water used for jetting (*Figure 3.2*). The method cannot be used for sampling and therefore, its primary purpose is to sink the hole between sampling positions. When a sample is required, the bit is replaced by a sampler. Nevertheless, some indication of the type of ground penetrated may be obtained from the cuttings carried to the surface by the wash water, from the rate of progress made by the bit or from the colour of the wash water.

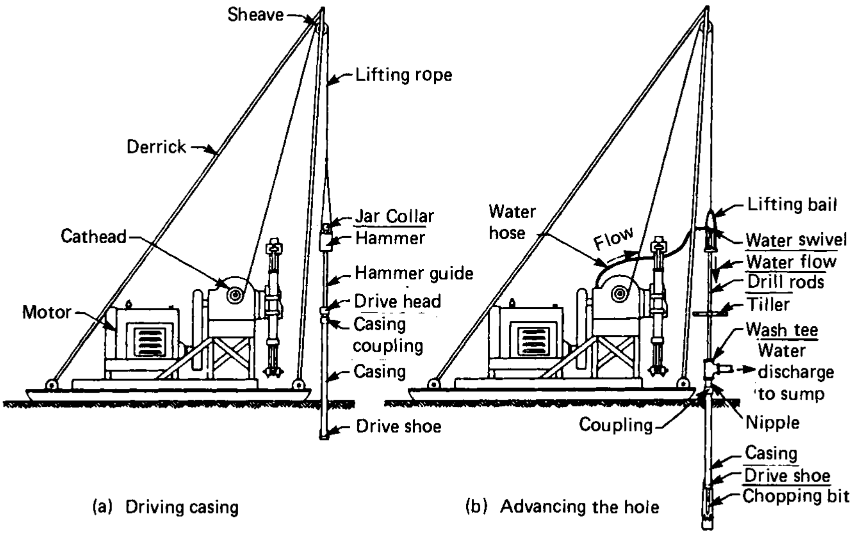

Figure 3.2 Wash boring rig

Several types of chopping bits are used. Straight and chisel bits are used in sands, silts, clays and very soft rocks whilst cross bits are used in gravels and soft rocks. Bits are available with either the jetting points facing upwards or downwards. The former type are better at cleaning the base of the hole than are the latter.

The wash boring method may be used in both cased and uncased holes. Casing obviously has to be used in cohesionless soils to avoid the sides of the hole collapsing. Although this method of boring is commonly used in the USA, it has rarely been employed in the UK. This is mainly because wash boring does not lend itself to many of the ground conditions encountered and also to the difficulty of identifying strata with certainty.

3.3.2 Subsurface exploration in rocks

Cable and tool or churn drilling is a percussion drilling method which is commonly used for drilling water wells[20] (see Section 6.2.1). This method of drilling can be used in more or less all types of ground conditions but the rate of progress tends to be slow. The rig can drill holes up to 0.6 m in diameter to depths of about 1000 m.

The hole is advanced by raising and dropping heavy drilling tools which break the soil or rock. Different bits are used for drilling in different formations and an individual bit can weigh anything up to 1500 kg, so that the total weight of the drill string may amount to several thousand kilograms. A slurry is formed from the broken material and the water in the hole. The amount of water introduced into the hole is kept to the minimum required to form the slurry. The slurry is periodically removed from the hole by means of bailers or sand pumps. In unconsolidated materials, casing should be kept near the bottom of the hole in order to avoid caving.

Changes in the type of strata penetrated can again be inferred from the cuttings brought to the surface, the rate of drilling progress or the colour of the slurry. Sampling, however, has to be done separately. Unfortunately, the ground which has to be sampled may be disturbed by the heavy blows of the drill tools.

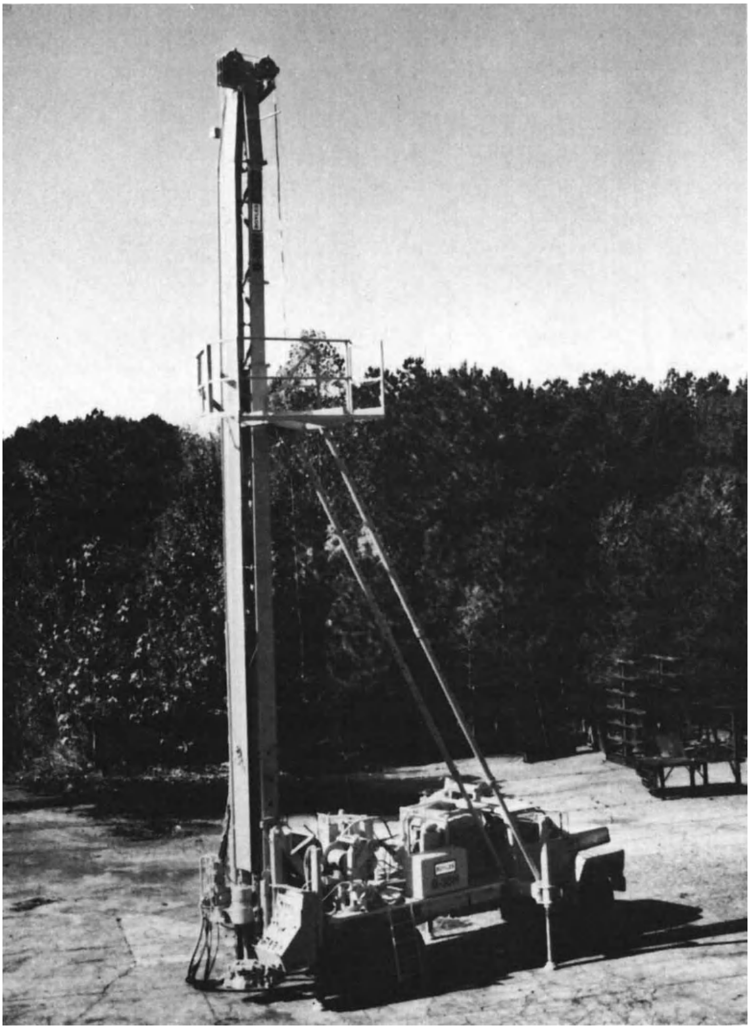

Figure 3.3 Lorry-mounted rotary drilling rig (courtesy of Landrill UK Ltd.)

Rotary drills are either skid-mounted, trailer mounted or, in the case of the larger types, mounted on lorries (*Figure 3.3*). They are used for drilling through rock, although they can, of course, penetrate soil.

Rotary-percussion drills are designed for rapid drilling in rock, the rock being subjected to high-speed impacts as the bit rotates. The hammering action is applied either at the top of the drill string, as in heavy drifters or wagon drills, or occurs immediately above the bit as in down-the-hole drills. However, in the former type of drill the impact energy transmitted from the piston to the bit during drilling diminishes as the depth of the hole increases. Since the rate of penetration is very much influenced by the speed at which the piston impacts, drilling performance may be improved by increasing the air pressure at which the drill operates. Unfortunately this leads to an increase in the rate of fatigue and failure of the drill rods. Hence usual maximum operating air pressures for such drills, for economic reasons, are around 0.5 MPa. This disadvantage is eliminated in down-the-hole drills since the impact mechanism is coupled to the drill bit. As a consequence operating air pressures are higher, generally around 1.7 MPa but in some instances they may range up to 2.5 MPa, which means that better rates of penetration are achieved. Nonetheless, these higher operating air pressures lead to increased wear and damage to bits.

Rotary-percussion drilling is most effective in brittle materials since it relies on breaking the rock by chopping brought about by the combined action of compression and shearing by the bit. Cross-chisel bits are used for drilling through overburden, soft rocks and highly fractured formations, whilst in medium to hard rocks studded bits are used.

Compressed air is used as the flushing medium. Drillhole cuttings should be sampled at regular intervals, when changes in the appearance of the cuttings occur and when significant changes in the rate of penetration take place. In this way rotary-percussion drilling can be used to provide some idea of the stratal succession. Rotary-percussion also can be used for drilling holes in which to perform *in situ tests*.

The selection of the bit in rotary drilling is very much influenced by the geological conditions, maximum rate of drilling being achieved when the bit is designed for the type of formation concerned. Hence estimates of the lengths of rock types, which probably will be met with during the exploration programme, are worthwhile. The rock properties which influence drillability include hardness, abrasiveness, grain size and the presence of discontinuities. The harder the rock, the more robust the bit which is required for drilling, since higher pressures need to be exerted. Where the rock conditions are likely to be continually changing, as within a formation of limestone containing thin beds of chert, the bit should be chosen in relation to the hardest rock to be encountered. Abrasiveness refers to the ability of a rock to wear away a bit. Bit wear is a more significant problem in rotary than percussive drilling. The most abrasive rocks are those containing appreciable amounts of quartz. In addition the size of the fragments produced during drilling operations influence abrasiveness. For example, large fragments may cause scratching but comparatively little wear, whereas the production of dust in tougher, but less abrasive rock, causes polishing. Generally coarse-grained rocks can be drilled more quickly than fine-grained varieties or those in which the grain size is variable.

The ease of drilling in rocks in which there are many discontinuities is influenced by their orientation in relation to the drillhole. Drilling over an open discontinuity means that part of the energy controlling drill penetration is lost. Where a drillhole crosses discontinuities at a low angle, then this may cause the bit to stick. It also may lead to excessive wear and to the hole going off line. If the ground is badly broken, then the drillhole may require casing. Where discontinuities are filled with clay this may penetrate the flush holes in the bit, causing it to bind or to deviate from alignment. As a consequence the rate of drilling is generally quicker if the hole runs at a high angle to the discontinuities.

Records should be kept of the bit type, drilling column weight, rotary speed, rate of drilling and type of flush, as well as rock type penetrated. Drag bits and cone rock bits are used in groundwater investigation. The former are primarily used in soft, weakly cemented formations. Cone bits can be used in any type of rock.

When using an air flush without casing it is not possible to support the walls of a drillhole in softer rock so that the method is only successful when used in harder rocks. On the other hand, when using a drilling fluid, holes can be sunk in almost any type of rock. The difference in head between the drilling fluid and the water table is kept high enough for the walls of the drillhole to remain stable without casing. The diameter of the drillhole should be large enough to permit the drilling fluid to rise fast enough to bring the cuttings to the surface. When mud is used as a flush the behaviour of the groundwater in the hole, for instance, its fluctuation, cannot be observed during drilling operations. What is more it is difficult to ascertain the depth from which the cuttings originate since separation occurs as the mud comes to the surface. The casing used is dependent on the geological conditions and likely requirements for testing aquifers. Exploration wells should be sunk with the idea of retrieving the casing.

If rock core material is required for examination then it is obtained with the aid of a bit and housed in a core barrel. Core recovery is dependent upon the properties of the rocks involved, the drilling pressure and the type of core barrel used. Core bits vary in size, ranging from 17.5 to 165 mm in diameter[21]. The bit is set with diamonds or tungsten carbide inserts. In set bits diamonds are set on the face of the matrix. The coarser surface set diamond and tungsten carbide tipped bits are used in softer formations. These bits are generally used with air rather than with water flush. Impregnated bits possess a matrix impregnated with diamond dust and their grinding action is suitable for hard and broken formations. In fact, most core drilling is carried out using diamond bits, the type of bit used being governed by the rock type to be drilled. In other words, the harder the rock, the smaller the size and the higher the quality of the diamonds that are required in the bit. Tungsten bits are not generally suitable for drilling in very hard rocks. Thick-walled bits are more robust but penetrate more slowly than thin-walled bits. The latter produce a larger core for a given hole size. This is important where several reductions in size have to be made.

A variety of core barrels are available for rock sampling. The single tube is the simplest type of core barrel but because it is suitable only for massive rocks it is rarely used. Double tubes may be of the rigid or swivel type. The disadvantage of the rigid barrel is that both the inner and outer tubes rotate together and in soft rock this can break the core as it enters the inner tube. It is therefore only suitable for hard rock formations. In the double tube swivel core barrel, the outer tube rotates while the inner tube remains stationary. It is suitable for use in medium and hard rocks and gives improved core recovery in soft friable rocks. The face-ejection barrel is a variety of the double tube swivel type in which the flushing fluid does not affect the end of the core. This type of barrel is a minimum requirement for coring badly shattered, weathered and soft rock formations. Triple tube barrels have been developed for obtaining cores from very soft rocks and from highly jointed and cleaved rocks.

As cores are cut under low drilling pressure, the rate of progress is slow. Also, after each core length has been drilled, the barrel must be withdrawn. Accordingly, cores are frequently only obtained from specified locations to obtain certain geological data. In this way, costs are kept down.

A side-wall coring device which shoots cylindrical bullets into the wall of a drillhole can be used to obtain samples from sections of the hole from which cores were not taken but about which more data subsequently are required. These bullets are fired by electrically ignited charges and remain attached to the gun by retrieving wires. Side-wall coring guns contain several bullets and so can obtain samples from a number of levels. The samples permit determination of porosity, permeability, fluid content, etc. to be made.

The formation fluid sampler can be used to obtain pore water from sands and operates in a similar manner to the side-wall coring gun. Bullets, which are connected by a tube to a reservoir, are fired into the formation. In so doing, the nose of the bullet opens and allows pore water to flow via the tube into the reservoir. When the reservoir is full it is raised up the drillhole, this movement seals the reservoir and removes the bullet from the formation.

Disturbance of the core is most likely to occur when it is removed from the core barrel. Most rock cores should be removed by hydraulic extruders while the inner tube is held horizontal. To reduce disturbance during extrusion the inner tube of double core barrels can be lined with a plastic sleeve before drilling commences. On completion of the core run the plastic sleeve containing the core is withdrawn from the barrel.

The results from a borehole or drillhole should be documented on a log (*Figure 3.4*). The fundamental requirement of a drillhole log is to show how the sequence of strata changes with depth[22]. Individual soil and rock types are generally presented in symbolic form on a log and the material must be adequately described. Particular mention of the water level in the drillhole and any water loss, when it is used as a flush during rotary drilling, should be noted, as these reflect the permeability of the ground. The depths from which samples are taken must be recorded as must the position of any *in situ* tests carried out. The spacing of discontinuities in rock cores should be recorded since it can convey a general impression of permeability conditions (see Chapter 2).

Direct observation of strata, discontinuities and cavities can be undertaken by cameras or closed-circuit underwater television equipment and drillholes can be viewed either radially or axially. Remote focusing for all heads and rotation of the radial head through 360° are controlled from the surface. The television heads have their own light source. Colour changes in rocks can be detected as a result of the varying amount of light reflected from the drillhole walls. Discontinuities appear as dark areas because of the non-reflection of light. However, if the drillhole is deflected from the vertical, variations in the distribution of light may result in some lack of picture definition. Callahan *et al.*[23] suggested that a small submersible pump could be fitted to the television to supply fresh water when the water in the drillholes is cloudy due to the presence of suspended material.

3.4 Geophysical exploration

Geophysical methods are used to determine the geological sequence and structure of subsurface rocks by the measurement of certain physical properties or forces. Hence they facilitate the planning of efficient and economical test drilling programmes. In other words, the data they yield regarding geological structure may be used to minimize the number of drillholes required and their depths may be estimated before drilling.

Apart from instrumental errors every geophysical measurement is subject to the effect of random, often local, variations in subsurface properties. The greater the 'noise' compared with the magnitude of the anomaly to be measured, the more closely spaced the stations must be if the anomaly is to be picked out. Indeed this noise may control detectability. Generally speaking, observations should be close enough for correlation between them to be obvious, so enabling interpolation to be carried out without ambiguity. Nevertheless, it must be admitted that an accurate and unambiguous interpretation of geophysical data is only possible where the subsurface structure is simple and even then there is no guarantee that this will be achieved[24]. Boreholes or drillholes, to facilitate correlation and interpretation of the geophysical measurements, are an essential part of any geophysical survey.

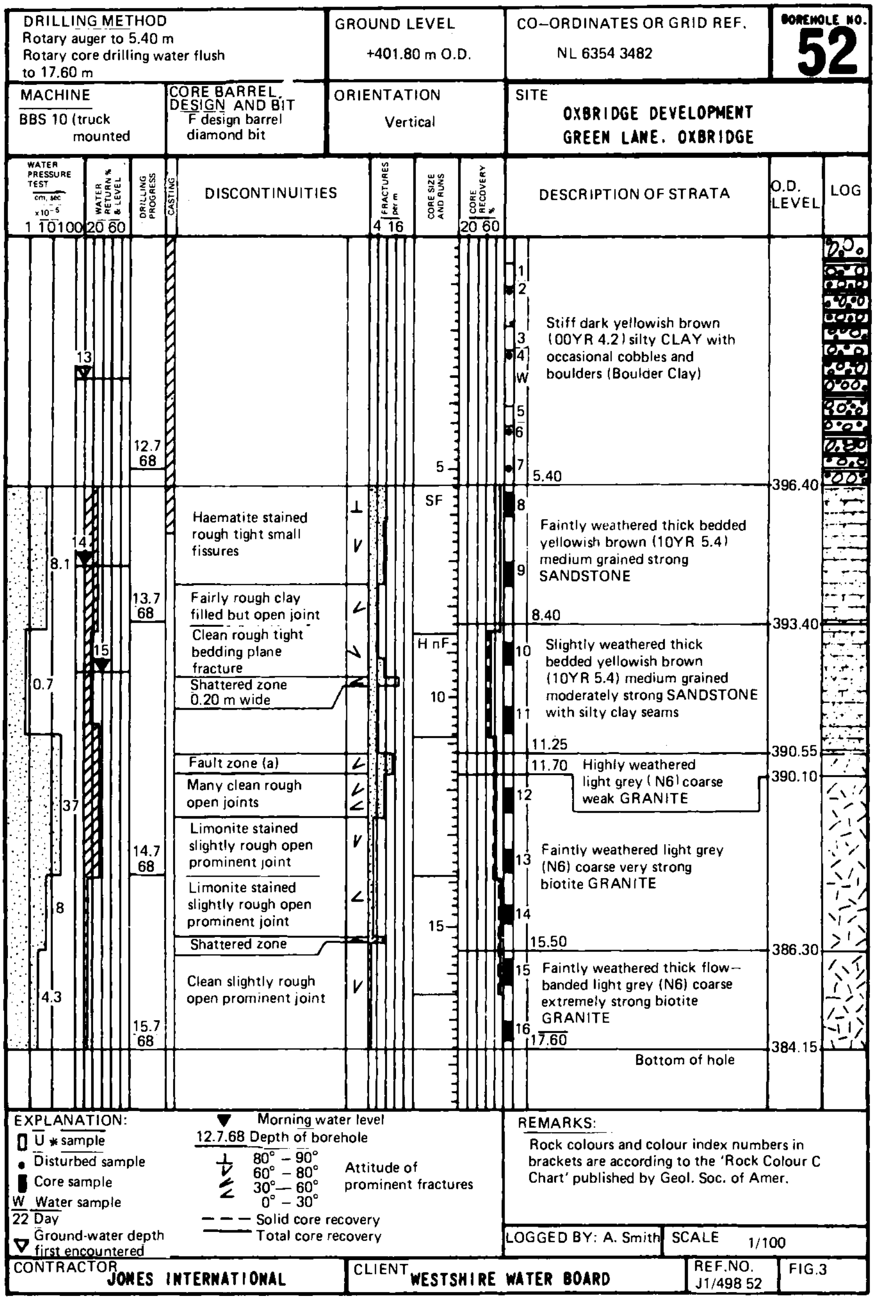

Figure 3.4 Drillhole log (courtesy of The Geological Society)

3.4.1 Seismic methods

The sudden release of energy from the detonation of an explosive charge in the ground generates seismic shock waves which radiate out in hemi-spherical wave fronts from the point of release. The waves generated are compressional, *P*, dilational shear, *S* and surface waves. The velocities of shock waves generally increase with depth below the surface since the elastic moduli increase more rapidly with depth than density. The compressional waves travel faster and are more easily generated and recorded than the shear waves. They are therefore used almost exclusively in seismic exploration.

The shock wave velocity depends on many variables, including rock fabric, mineralogy and pore water. In general, velocities in crystalline rocks are high to very high (*Table 3.1*). Velocities in sedimentary rocks increase concomitantly with consolidation and with increase in the degree of cementation and diagenesis. Unconsolidated sedimentary accumulations have maximum velocities varying as a function of the mineralogy, the volume of voids, either air-filled or water-filled and grain size. Most rocks and unconsolidated deposits are anisotropic. Notable anisotropism is present in stratified sedimentary accumulations and in metamorphic rocks possessing foliation and schistosity. Ordinary velocities in the directions parallel to planar structures in anisotropic rocks are greater than in directions perpendicular to these structures.

TABLE 3.1. Velocities of compressional waves of some common rocks

	(km/s) V_p		(km/s V_p
Igneous rocks		Sedimentary rocks	
Basalt	5.1–6.4	Gypsum	2.0–3.5
Dolerite	5.8–6.6	Limestone	2.8–7.0
Gabbro	6.5–6.7	Sandstone	1.4–4.4
Granite	5.5–6.1	Shale	2.1–4.4
Metamorphic rocks		Unconsolidated deposits	
Gneiss	3.5–7.0	Alluvium	0.3–0.6
Marble	3.7–6.9	Sands and gravels	0.3–1.8
Quartzite	5.6–6.1	Clay (wet)	1.5–2.0
Schist	3.5–5.7	Clay (sandy)	2.0–2.4
Slate	3.5–5.4		

As can be seen from above, the porosity tends to lower the velocity of shock waves through a material. Indeed Wyllie *et al.*[25] suggested that the compressional wave velocity (V_p) is related to the porosity (n) of a normally consolidated sediment as follows

$$\frac{1}{V_p}=\frac{n}{V_{pf}}+\frac{1-n}{V_{pl}} \tag{3.1}$$

where V_{pf} is the velocity in the pore fluid and V_{pl} is the compressional wave velocity for the intact material as determined in the laboratory. The compressional wave velocities may be raised appreciably by the presence of water. For example, Grainger *et al.*[26] noticed that the velocities in weathered chalk increased from 0.7 to 1.1 km/s when unsaturated to 1.95 km/s when saturated.

Worthington and Griffiths[27] suggested procedures for the use of seismic methods at various stages in the development of a sandstone aquifer. Because of the relationship between seismic velocity and porosity, they maintained that seismic velocity was also broadly related to intergranular permeability of sandstone formations. However, in most sandstones fissure flow makes a more important contribution to groundwater movement than does intergranular or primary flow.

When seismic waves pass from one layer to another, some energy is reflected back towards the surface while the remainder is refracted. Thus, two methods of seismic surveying can be distinguished, that is, seismic reflection and seismic refraction. Measurement of the time taken from the generation of the shock waves until they are recorded by the detector arrays at the surface forms the basis of the two methods.

The seismic reflection method is the most extensively used of all geophysical tests, its principal employment being in the oil industry. In this instance the depth of investigation is large compared with the distance from the shot to detector array. This is to exclude refraction waves. Indeed the method is able to record information from a large number of horizons down to depths of several thousands of metres. It also possesses appreciable accuracy, particularly when only the changes in depth of a reflector, rather than absolute depths, are required.

In the seismic refraction method one ray approaches the interface between two rock types at a critical angle which means that if the ray is passing from a low velocity (V_0) to a high velocity (V_1) layer it is refracted along the upper boundary of the latter layer. After refraction, the pulse travels along the interface with velocity V_1. The material at the boundary is subjected to oscillating stress from below. This generates new disturbances along the boundary which travel upwards through the low velocity rock and eventually reach the surface. At short distances from the point where the shock waves are generated the geophones record direct waves, whilst at a critical distance both the direct and refracted waves arrive at the same time. Beyond this, because the rays refracted along the high velocity layer travel faster than those through the low velocity layer above, they reach the geophones first.

In refraction work the object is to develop a time–distance graph which involves plotting arrival times against geophone spacing (*Figure 3.5*). Thus the distance between geophones, together with the total length and arrangement of the array has to be chosen carefully to suit each particular problem.

In the simple case of refraction by a single high velocity layer at depth, the travel time taken by the seismic wave which proceeds directly from the shot point to the detectors and the travel time taken by the critical refracted wave to arrive at the geophones, are plotted graphically against geophone spacing (*Figure 3.5*). The depth, Z, to the high velocity layer can then be obtained from the graph by using the expression

$$Z = \frac{x_1}{2}\sqrt{\frac{V_1 - V_0}{V_1 + V_0}} \tag{3.2}$$

where V_0 is the speed in the low velocity layer, V_1 is the speed in the high velocity layer and x_1 is the distance to the point where the direct and refracted waves arrive simultaneously, that is, the critical distance.

The method also works for multi-layered rock sequences if each layer is sufficiently thick and transmits seismic waves at higher speeds than the one above it, the time distance curve having more segments. The position of faults which displace beds can also be estimated from the time–distance graphs.

A highly fractured or weathered rock mass will exhibit a lower compressional velocity than one which is sound. The effect of discontinuities within a rock mass can be estimated by comparing the *in situ* compressional wave velocity, V_{pf}, with the

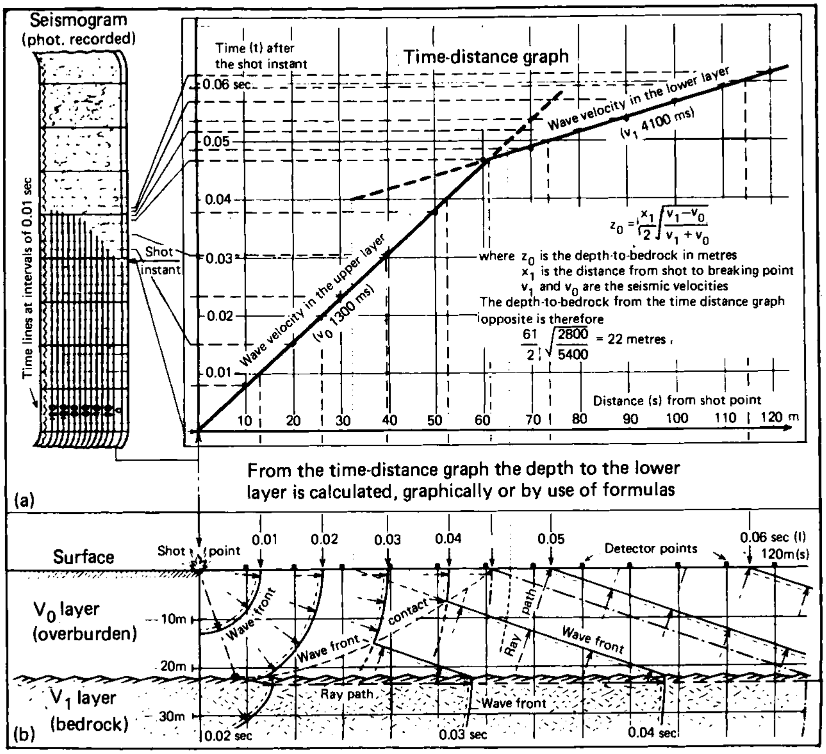

Figure 3.5 Time–distance graphs for a theoretical single-layer problem, with parallel interface. With non-parallel interfaces, both forward and reverse profiles must be surveyed

laboratory sonic velocity, V_{pl} of an intact core obtained from the same rock mass. The difference in these two velocities is caused by the discontinuities in the rock mass. The velocity ratio (V_{pf}/V_{pl}) has been proposed as a rock quality index. For high-quality, massive rock with only a few tight joints the velocity ratio approaches unity while lower and lower values below unity are recorded as rocks become more jointed and fractured. Furthermore, if the discontinuities within a rock mass possess a notable preferred orientation, then velocity anisotropy results. For instance, Nunn *et al.*[28] demonstrated that in the Chalk of north Lincolnshire there was a direct relationship between seismic velocity and the predominant discontinuity direction.

3.4.2 Resistivity methods

The advantages of the resistivity method are low cost, ease of operation, speed and accuracy[29]. However, the resistivity method does not provide satisfactory quantitative results if the potential aquifer(s) being surveyed are thin, that is, they are 6 m or less in thickness, especially if they are separated by thick argillaceous horizons. In such situations either cumulative effects are obtained or anomalous resistivities are measured, the interpretation of which is extremely difficult, if not impossible. In addition, the resistivity method is more successful when used to investigate a formation which is thicker than the one above it.

The resistivity of rocks and soils varies over a wide range. Since most of the principal

TABLE 3.2. Resistivity of some types of natural water

Type of water	*Resistivity* (ohm-m)
Meteoric water, derived from precipitation	30–1000
Surface waters, in districts of igneous rocks	30–500
Surface waters, in districts of sedimentary rocks	10–100
Ground water, in areas of igneous rocks	30–150
Ground water, in areas of sedimentary rocks	larger than 1
Sea water	about 0.2

rock-forming minerals are practically insulators, the resistivity of rocks and soils is determined by the amount of conducting mineral constituents and the water content in the pores. The latter condition is by far the dominant factor and in fact, most rocks and soils conduct an electric current only because they contain water[30].

The widely differing resistivity of the various types of impregnating water can cause variations in the resistivity of soil and rock formations ranging from a few tenths of an ohm-metre to hundreds of ohm-metres, as can be seen from *Table 3.2*. In addition, the resistivity of water changes markedly with temperature, even within the range of temperature at which fresh groundwater occurs. This is because, as the temperature of groundwater increases, it has greater ionic mobility associated with decreasing viscosity and so the resistivity decreases. Hence, for each bed under investigation the temperature of both rock and water must be determined or closely estimated and the calculated resistivity of the pore water at that temperature converted to its value at a standard temperature (i.e. 25°C, see *Figure 3.6*). In resistivity logging (see below), values of true resistivity have to be corrected for temperatures since temperature increases with depth.

Dry rocks, whether non-porous or porous, are therefore practically non-conductors, but resistivity decreases with increasing amount of pore water. In general, sedimentary

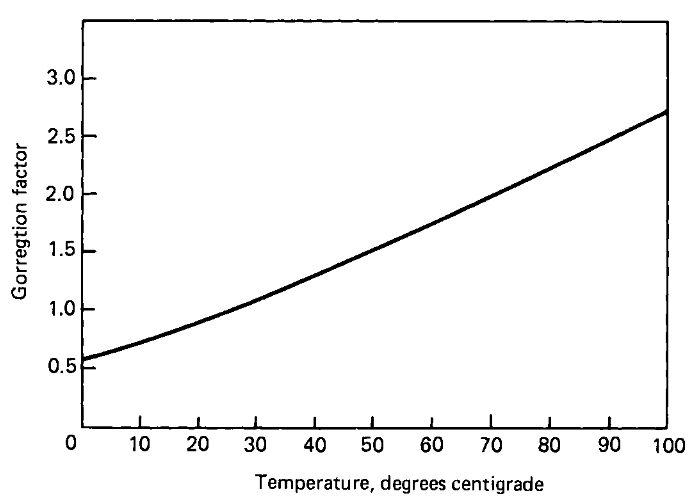

Figure 3.6 Correction factor to convert resistivity at other temperatures to resistivity at 25 °C (after Jones, P. H. and Burford, T. B., 'Electric logging applied to groundwater exploration[1], *Geophysics*, **16**, 115–39 (1951))

rocks are better conductors than igneous rocks. Clayey material has a higher conductivity than material of sandy type. This is because ions cluster on the surfaces of clay minerals and in so doing increase the current-carrying capacity. Compacted material also is a better conductor than unconsolidated material. It is possible accordingly to distinguish between the major groups of rocks. In stratified and schistose rocks the resistivity is higher in the direction normal to the bedding planes or schistosity than in the direction parallel to these features.

In the resistivity method an electric current is introduced into the ground by means of two current electrodes and the potential difference between two potential electrodes is measured. The difference in potential is measured in ohms and the value is converted to apparent resistivity by use of a factor that depends on the particular electrode configuration in use (see below).

The depth to which the current penetrates increases with increasing electrode distance. In addition, 50 per cent of the total current passes above a depth equal to about half the electrode separation and 70 per cent flows within a depth equal to the electrode separation. Analysis of the variation in the values of apparent resistivity with respect to electrode separation, which is taken to equal depth of penetration, enables inferences to be drawn about the subsurface formations.

The resistivity method is based on the fact that any subsurface variation in conductivity alters the pattern of current flow in the ground and therefore changes the distribution of electric potential at the surface. The first step in any resistivity survey should be to conduct a resistivity depth sounding at the site of a borehole or rock outcrop in order to establish a correlation between resistivity and lithological layers (*Table 3.3*). This will also aid the choice of electrode spacing for subsequent resistivity traversing. If a correlation cannot be established, then an alternative method is required.

The electrodes are normally arranged along a straight line, the potential electrodes being placed inside the current electrodes and symmetrically disposed with respect to the centre of the configuration. The configurations of the symmetric type that are mostly used are those introduced by Wenner and by Schlumberger. The expressions used to compute the apparent resistivity, ρ_a, for the Wenner and Schlumberger configurations are

$$\text{Wenner:} \quad \rho_a = 2\pi a R \tag{3.3}$$

$$\text{Schlumberger:} \quad \rho_a = \frac{\pi(L^2 - l^2)}{2l} \times R \tag{3.4}$$

TABLE 3.3 Resistivity values of some common rock types

Rock type	*Resistivity* (ohm-m)
Topsoil	5–50
Peat and clay	8–50
Clay, sand and gravel mixtures	40–250
Saturated sand and gravel	40–100
Moist to dry sand and gravel	100–3000
Mudstones, marls and shales	8–100
Sandstones and limestones	100–1000
Crystalline rocks	200–10000

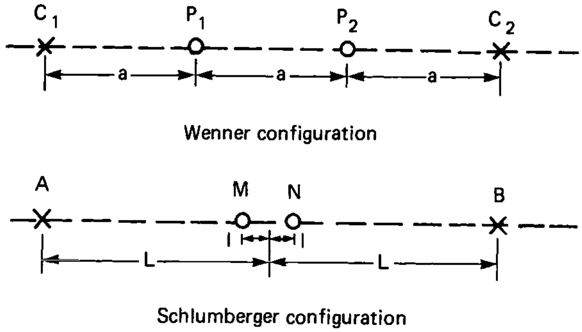

Figure 3.7 Wenner and Schlumberger configurations

where a, L and l are explained in *Figure 3.7* and R is the resistance reading. In the Wenner configuration the distances between all four electrodes are equal. The spacings can be progressively increased, keeping the centre of the array fixed (as in sounding) or the whole array, with fixed spacings, can be shifted along a given line (as in profiling). In the Schlumberger arrangement, the potential electrodes maintain a constant separation during sounding, while the current electrodes are moved outwards after each reading.

Horizontal profiling is used to determine the variations in the apparent resistivity in a horizontal direction at a pre-selected depth. For this purpose an electrode configuration, with suitable inter-electrode distances, is moved as an entity along straight traverses, resistivity determinations being made at stations set out at regular intervals. The Wenner spread is as satisfactory for constant separation traverses, in most instances, as any other, but if lateral resolution of anomalies is important, it may be better to use the Schlumberger method. When this type of spread is used, the magnitude of the measured anomalies is smaller, they are sharper and, if ground conditions are favourable, more information may be obtained. Much depends on the homogeneity of the various rock types involved, particularly of the overburden, since any type of spread using a small potential electrode separation is sensitive to local shallow resistivity variations and the spurious anomalies so produced may obscure the true ones.

In profiling, a constant spacing may be adopted to measure resistivities at a particular depth of interest, such as an aquifer. Spatial changes in resistivity may be interpreted in terms of aquifer limits and changes in the quality of groundwater.

Electrical sounding furnishes detailed information concerning the vertical succession of different conducting zones and their individual thicknesses and resistivities. For this reason the method is particularly valuable for investigations on horizontally or nearly horizontally stratified ground. In electrical sounding the midpoint of the electrode configuration is fixed at the observation station while the length of the configuration is gradually increased. As a result the current penetrates deeper and deeper, the apparent resistivity being measured each time the current electrodes are moved outwards. It therefore becomes more and more affected by the resistivity conditions at increasing depths. When using the Schlumberger configuration the interval between these electrodes is increased only a few times and in relatively small steps in order to obtain potential differences large enough to be measured with satisfactory precision. The Schlumberger configuration is preferable to the Wenner configuration for depth sounding because the field procedure is quicker and simpler and master curves are more readily available to analyse the results[31]. It should be noted, however, that the measured potential differences are less than for the Wenner configuration and a high-quality resistivity meter is required for precision, especially for low-resistivity ground.

Ground resistivities are never uniform so that a single depth determination cannot be relied upon. Accordingly readings may be repeated at the same spot but with the spread in a different azimuth and a few more sets of measurements are usually taken nearby. The data obtained are usually plotted as a graph of apparent resistivity against electrode separation, in the case of the Wenner array*, or half the current electrode separation in the Schlumberger array. The electrode separation at which inflection points occur on the graph provide an indication of the depth of interfaces (*Figure 3.8*).

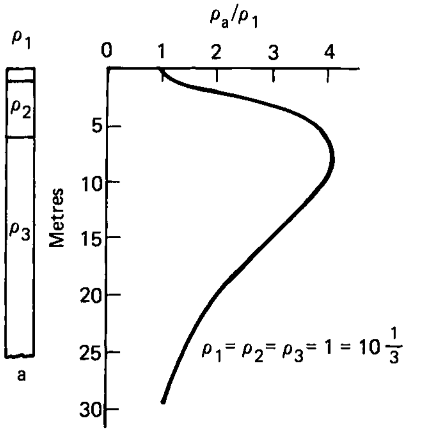

Figure 3.8 The variation of apparent resistivity with electrode separation for a three-layer earth

The apparent resistivity, however, is a measure of the effects of all the layers between the maximum depth of penetration and the surface. For this reason the greater the number of beds present, the more difficult the interpretation becomes. If a second layer is relatively thin and its resistivity much larger or smaller than that of the first layer, the interpretation of its lower contact will be inaccurate. Three or four distinct layers are about the maximum number for accurate interpretation unless other subsurface information is available. Furthermore, because changes of resistivity at great depths have only a slight effect on the apparent resistivity compared to those at shallow depths, the method is seldom effective for determining actual resistivities below several tens of metres.

When master curves are used for depth interpretation, the curve is plotted on logarithmic paper in order that comparisons can be made[30]. The method again allows detection of up to three or four layers.

For all depth determinations from resistivity soundings it is assumed that there is no change in resistivity laterally. This is not the case in practice. Indeed sometimes the lateral change is greater than that occurring with increasing depth and so corrections have to be applied for the lateral effects when depth determinations are made. In some

* The assumption that the electrode spacing in the Wenner configuration is directly related to depth of penetration can produce errors in calculated depths of several hundred per cent.

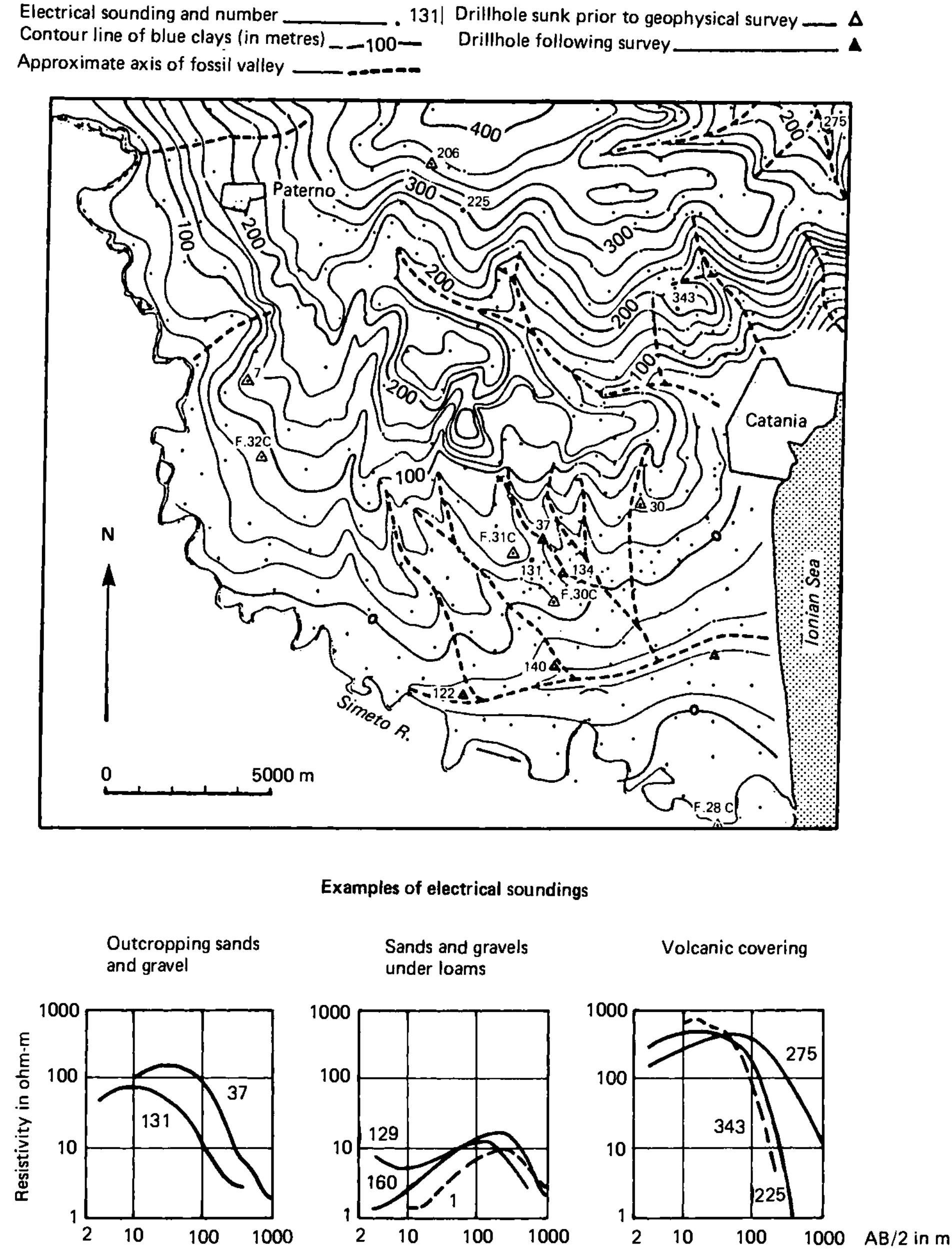

Figure 3.9 Catania Plain—isoresistivity map of Blue Clays (substratum) (after Breusse, J. J., 'Modern geophysical methods for subsurface water exploration', *Geophysics*, **28**, 633–57 (1963))

cases the lateral effect is the major feature of the curve and depth interpretation can be very inaccurate.

The data of a constant separation survey may be used to construct a contour map of lines of equal resistivity (*Figure 3.9*). From borehole and depth probe interpretations the relation between overburden thickness and apparent resistivity is derived and if it seems reasonably constant over the area the resistivity contours may be regarded as rough depth contours. In other words, resistivity maps and profiles can be used to indicate the extent and significance of intermediate layers, for example, a gravel bed sandwiched between two layers of clay. Once anomalies have been located they can be further investigated by resistivity depth probes, seismic refraction traverses or direct methods.

As the amount of water present is influenced by the porosity of a rock, the resistivity provides a measure of its porosity. For example, in granular materials in which there are no clay minerals, the relationship between the resistivity, ρ, on the one hand and the density of the pore water, ρ_w, the porosity, n, and the degree of saturation, S_r, on the other is as follows[32]

$$\rho = a\rho_w n^{-x} S_r^{-y} \tag{3.5}$$

where a, x and y are variables (x ranges from 1.0 for sand to 2.5 for sandstone and y is approximately 2.0 when the degree of saturation is greater than 30 per cent). If clay minerals do occur in sands or sandstones, then the resistivity of the pore water is significantly reduced by ion exchange between the latter and the clay minerals so that the above relationship becomes invalid. For those formations which occur below the water table and are therefore saturated, the above expression becomes

$$\rho = a\rho_w n^{-x} \tag{3.6}$$

since $S_r = 1$ (i.e. 100 per cent). In fact, if two rocks have the same water content and one has a porosity of 10 per cent and the other of 30 per cent, the former is ten times as resistive as the latter.

In a fully saturated sandstone a fundamental empirical relationship exists between the electrical and hydrogeological properties which involves the concept of the formation resistivity factor, F_a, defined as

$$F_a = \frac{\rho_0}{\rho_w} \tag{3.7}$$

where ρ_0 is the resistivity of the saturated sandstone and ρ_w is the resistivity of the saturating solution. In a clean sandstone, that is, one in which the electrical current passes through the interstitial electrolyte during testing with the rock mass acting as an insulator, the formation resistivity factor is closely related to the porosity. Worthington[33] showed that the formation resistivity factor was related to the true formation factor, F, by the expression

$$F = \frac{\rho_A F_a}{\rho_A - F_a \rho_w} \tag{3.8}$$

in which ρ_A is a measure of the effective resistivity of the rock matrix. In clean sandstone ρ_A is infinitely large, consequently $F = F_a$. Generally F is related to the porosity, n, by the equation

$$F = a/n^m \tag{3.9}$$

where a and m are constants for a given formation (for instance, in the Bunter Sandstone of the Fylde region, Lancashire, $a = 1.05$ and $m = 1.47$, according to Barker and Worthington[34]). It is also possible to obtain the formation factor from a seismic survey. For example, Barker and Worthington showed that the relationship between the compressional wave velocity, V_c, and the true formation factor, F, in the Bunter Sandstone took the form

$$V_c = 2.07 \log_{10} F + 0.35 \tag{3.10}$$

The true formation factor in certain formations has also been shown to be broadly related to intergranular permeability, k_g, by the expression

$$F = b/k_g^n \tag{3.11}$$

where b and n are constants for a given formation. Barker and Worthington found that the values of b and n for the same Bunter Sandstone were 3.3 and 0.17, respectively. In sandstones in which intergranular flow is important, the above expressions can be used to estimate hydraulic conductivity and thence, if the thickness of the aquifer is known, transmissivity.

According to Worthington and Griffiths[27] it appears that quantitative geophysical investigations of variations in transmissivity and hydraulic conductivity are economical only where intensive development of a sandstone aquifer is planned. The techniques are not useful in highly indurated sandstones, where the intergranular permeabilities are less than 1.0×10^{-7} m/s, and the flow is controlled by fissures. Similarly these methods are of little value in multi-layered aquifers except perhaps where the thicknesses of the different layers are fortuitously distributed so as to allow a complete and definite geophysical interpretation.

3.4.3 Magnetic and gravity methods

All rocks are magnetized to a lesser or greater extent by the Earth's magnetic field. As a consequence, in magnetic prospecting, accurate measurements are made of the anomalies produced in the local geomagnetic field by this magnetization. The intensity of magnetization and hence the amount by which the Earth's magnetic field is changed locally, depends on the magnetic susceptibility of the material concerned. In addition to the magnetism induced by the Earth's field, rocks possess a permanent magnetism that depends upon their history.

Aeromagnetic surveying has almost completely supplanted ground surveys for regional reconnaissance purposes. The results of a ground survey generally are used to produce isomagnetic contour maps or profiles across an anomaly. However, interpretation of magnetic anomalies is usually qualitative and depth determinations are the exception rather than the rule.

A magnetometer may be used for mapping geological structures. For example, in some thick sedimentary sequences it is sometimes possible to delineate the major structural features because the succession includes magnetic horizons. These may be ferruginous sandstones or shales, tuffs or possibly lava flows. In such circumstances anticlines produce positive and synclines negative anomalies. Faults and dykes are indicated on isomagnetic maps by linear belts of somewhat sharp gradient or by sudden swings in the trend of the contours. However, in many areas the igneous and metamorphic basement rocks, which underlie the sedimentary sequence, are the predominant influence controlling the pattern of anomalies since they are usually far more magnetic than the sediments above. Where the basement rocks are brought near the surface in structural highs, the magnetic anomalies are large and characterized by strong relief. Conversely, deep sedimentary basins usually produce contours with low values and gentle gradients on magnetic maps.

The Earth's gravity field varies according to the density of the subsurface rocks but at any particular locality its magnitude is also influenced by latitude, elevation, neighbouring topographical features and the tidal deformation of the Earth's crust. The effects of these latter factors have to be eliminated in any gravity survey where the aim is to measure the variations in acceleration due to gravity precisely. These can then be used to construct a gravity contour map.

Gravity methods are mainly used in regional reconnaissance surveys to reveal anomalies which may be subsequently investigated by other methods. Since the gravitational effects of geological bodies are proportional to the contrast in density between them and their surroundings, gravity methods are particularly suitable for the location of structures in stratified formations. For example, Spangler and Libby[35] used the gravity method to derive a regional picture of the subsurface geology and to detect geological structures of interest in watershed groundwater hydrology.

Eaton *et al.*[36] reviewed the application of gravity measurements to reconnaissance mapping of sub-alluvial bedrock topography, the estimation of depth to bedrock and the *in situ* measurement of aquifer porosity. They concluded that gravimetric mapping of an alluviated terrain underlain by relatively dense bedrock provided a rapid means of obtaining a qualitative picture of buried bedrock topography and that the information would be of use to the geohydrologist. Precise gravity surveys in areas where unconsolidated sediments overlie uniformly dense, crystalline bedrock can yield figures on depth to bedrock with an average error of only ± 10 per cent, provided that data exist for establishing depth control in some part of the area. Where depth to bedrock is known independently, detailed gravity measurements, coupled with laboratory determinations of average grain density, allow calculation of an average, *in situ* porosity for an aquifer overlying bedrock.

3.5 Logging of boreholes or drillholes

3.5.1 Resistivity logging

The form of resistivity curve produced by logging a drillhole depends upon the configuration of the electrodes, the diameter of the drillhole, the resistivity of the drilling fluid and its depth of infiltration into the walls of the hole, the thickness of the individual beds involved and their porosity and electrolyte content of the water contained in the pores of these beds. The electrical resistivity method of drillhole logging makes use of various electrode configurations down-the-hole[37]. As the instrument is raised from the bottom to the top of the hole it provides a continuous record of the variations in resistivity of the wall rock. In the normal or standard resistivity configuration there are two potential and one current electrode in the sonde (*Figure 3.10*). The depth of penetration of the electric current from the drillhole is influenced by the electrode spacing. In a short normal resistivity survey spacing is about 400 mm, whereas in a long normal survey spacing is generally between 2.5 and 1.75 m (*Table 3.4*). Unfortunately in such a survey, because of the influence of thicker, adjacent beds, thin resistive beds yield resistivity values which are much too low, whilst thin conductive beds produce values which are too high. The microlog technique may be used in such situations. In this technique the electrodes are very closely spaced (25 to 50 mm) and are in contact with the wall of the drillhole. This allows the detection of small lithological changes so that much finer detail is obtained than with the normal electric log (*Figure 3.11*). A microlog is particularly useful in recording the position of permeable beds.

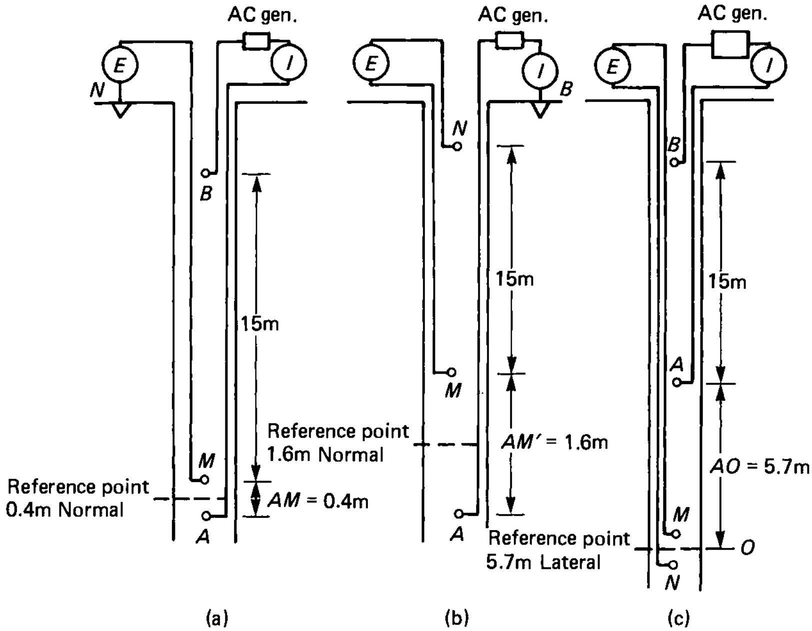

Figure 3.10 Typical electrode arrangements and standardized distances for resistivity logs (a) short normal (b) long normal (c) lateral

If, for some reason the current tends to flow between the electrodes on the sonde instead of into the rocks, then the laterolog or guard electrode is used. The laterolog 7 has seven electrodes in an array which focuses the current into the strata of the drillhole wall. The microlaterolog, a focused microdevice, is used in such a situation instead of the microlog (*Table 3.4*).

Croft[38] showed how the resistivity values obtained from drillhole logging could be used to determine permeability and transmissivity (see also Kelly[39]). First, the formation resistivity was determined from the long-normal curve and then corrected to a standard temperature of 25°C. Then the resistivity of the water was determined from readings taken of water in the hole. The formation factor (see above) was derived from these values and then converted to permeability by using *Figure 3.12*. The latter was then converted to transmissivity. The determination of permeability and transmissivity from resistivity curves of electric logs of drillholes has been used for the construction of transmissivity maps in areas where such logs are abundant.

Alger[40] suggested that in sandstone aquifers the resistivity value obtained from a 400 mm normal curve is very close to the true formation resistivity in many shallow water wells, provided that the well was drilled with a fresh mud flush that had a resistivity close to that of the formation water, mud invasion was shallow and the sandstone beds were relatively thick. The long-normal curve (1.6 m) can be used where the aforementioned conditions are not met with.

3.5.2 Induction logging

Induction logging measures the conductivity of strata by means of induced alternating currents[41]. Accordingly, insulated coils, rather than electrodes, are used to energize the

TABLE 3.4. Drillhole geophysical logging methods (after Johnson)*

Method	*Uses*	*Recommended conditions*
Electric logging:		
Single-electrode resistance.	Determining depth and thickness of thin beds. Identification of rocks, provided general lithologic information is available, and correlation of formations. Determining casing depths.	Fluid-filled hole. Fresh mud required. Hole diameter less than 200 to 250 mm. Log only in uncased holes.
Short normal (electrode spacing of 400 mm).	Picking tops of resistive beds. Determining resistivity of the invaded zone. Estimating porosity of formations (deeply invaded and thick interval). Correlation and identification, provided general lithologic information is available.	Fluid-filled hole. Fresh mud. Ratio of mud resistivity to formation-water resistivity should be 0.2 to 4. Log only in uncased part of hole.
Long normal (electrode spacing of 1.6 m).	Determining true resistivity in thick beds where mud invasion is not too deep. Obtaining data for calculation of formation-water resistivity.	Fluid-filled hole. Ratio of mud resistivity to formation-water resistivity should be 0.2 to 4. Log only in uncased part of hole.
Deep lateral (electrode spacing approximately 5.8 m).	Determining true resistivity where mud invasion is relatively deep. Locating thin beds.	Fluid-filled uncased hole. Fresh mud. Formations should be of thickness different from electrode spacing and should be free of thin limestone beds.
Limestone sonde (electrode spacing of 0.8 m).	Detecting permeable zones and determining porosity in hard rock. Determining formation factor *in situ*.	Fluid-filled uncased hole. May be salty mud. Uniform hole size. Beds thicker than 1.5 m.
Laterolog	Investigating true resistivity of thin beds. Used in hard formations drilled with very salty muds. Correlation of formations, especially in hard-rock regions.	Fluid-filled uncased hole. Salty mud satisfactory. Mud invasion not too deep.
Microlog	Determining permeable beds in hard or well-consolidated formations. Detailing beds in moderately consolidated formations. Correlation in hard-rock country. Determining formation factor *in situ* in soft or moderately consolidated formations. Detailing very thin beds.	Fluid required in hole. Log only in uncased part of hole. Bit-size hole (caved sections may be logged, provided hole enlargements are not too great).
Microlaterolog	Determining detailed resistivity of flushed formation at wall of hole when mudcake thickness is less than 1 cm in all formations. Determining formation factor and porosity. Correlation of very thin beds.	Fluid-filled uncased hole. Thin mud cake. Salty mud permitted.

TABLE 3.4—*contd.*

TABLE 3.4—*contd.*

Method	*Uses*	*Recommended conditions*
Induction logging	Determining true resistivity, particularly for thin beds (down to about 0.6 m thick) in wells drilled with comparatively fresh mud. Determining resistivity of formations in dry holes. Logging in oil-base muds. Defining lithology and bed boundaries in hard formations. Detection of water-bearing beds.	Fluid-filled or dry uncased hole. Fluid should not be too salty.
Spontaneous potential.	Helps delineate boundaries of many formations and the nature of these formations. Indicating approximate chemical quality of water. Indicate zones of water entry in drillhole. Locating cased interval. Detecting and correlating permeable beds.	Fluid-filled uncased hole. Fresh mud.
Sonic logging	Logging acoustic velocity for seismic interpretation. Correlation and identification of lithology. Reliable indication of porosity in moderate to hard formations; in soft formations of high porosity it is more responsive to the nature rather than quantity of fluids contained in pores.	Not affected materially by type of fluid, hole size, or mud invasion.
Radioactive logging:		
Gamma ray	Differentiating shale, clay, and marl from other formations. Correlations of formations. Measurement of inherent radioactivity in formations. Checking formation depths and thicknesses with reference to casing collars before perforating casing. For shale differentiation when holes contain very salty mud. Radioactive tracer studies. Logging dry or cased holes. Locating cemented and cased intervals. Logging in oil-base muds. Locating radioactive ores. In combination with electric logs for locating coal or lignite beds.	Fluid-filled or dry cased or uncased holes. Should have appreciable contrast in radioactivity between adjacent formations.
Neutron	Delineating formations and correlation in dry or cased holes. Qualitative determination of shales, tight formations and porous sections in cased wells. Determining porosity and water content of formations, especially those of low porosity. Distinguishing between water- or oil-filled and gas-filled reservoirs. Combining with gamma-ray log for better identification of lithology and correlation of formations. Indicating cased intervals. Logging in oil-base muds.	Fluid-filled or dry cased or uncased hole. Formations relatively free from shaly material. Diameter less than 150 mm for dry holes. Hole diameter similar throughout.

TABLE 3.4—*contd.*

Method	*Uses*	*Recommended conditions*
Temperature logging	Locating approximate position of cement behind casing. Determining thermal gradient. Locating depth of lost circulation. Locating active gas flow. Used in checking depths and thickness of aquifers. Locating fissures and solution openings in open holes and leaks or perforated sections in cased holes. Reciprocal-gradient temperature log may be more useful in correlation work.	Cased or uncased hole. Can be used in empty hole if logged at very slow speed, but fluid preferred in hole. Fluid should be undisturbed (no circulation) for 6 to 12 h minimum before logging; possibly several days may be required to reach thermal equilibrium.
Fluid-conductivity logging.	Locating point of entry of different quality water through leaks or perforations in casing or opening in rock hole. (Usually fluid resistivity is determined and must be converted to conductivity.) Determining quality of fluid in hole for improved interpretation of electric logs. Determining fresh-water–salt-water interface.	Fluid required in cased or uncased hole. Temperature log required for quantitative information.
Fluid-velocity logging	Locating zones of water entry into hole. Determining relative quantities of water flow into or out of these zones. Determine direction of flow up or down in sections of hole. Locating leaks in casing. Determine approximate permeability of lithologic sections penetrated by hole, or perforated section of casing.	Fluid-filled cased or uncased hole. Injection, pumping, flowing, or static (at surface) conditions. Flange or packer units required in large diameter holes. Caliper (section gauge) logs required for quantitative interpretation
Casing-collar locator	Locating position of casing collars and shoes for depth control during perforating. Determining accurate depth references for use with other types of logs.	Cased hole.
Caliper survey	Determining hole or casing diameter. Indicates lithologic character of formations and coherency of rocks penetrated. Locating fractures, solution openings, and other cavities. Correlation of formations. Selection of zone to set a packer. Useful in quantitative interpretation of electric, temperature and radiation logs. Used with fluid-velocity logs to determine quantities of flow. Determining diameter of underreamed section before placement of gravel pack. Determining diameter of hole for use in computing volume of	Fluid-filled or dry cased or uncased hole. (In cased holes does not give information on beds behind casing.)

TABLE 3.4—*contd.*

TABLE 3.4. Drillhole geophysical logging methods (after Johnson)*

Method	*Uses*	*Recommended conditions*
	cement to seal annular space. Evaluating the efficiency of explosive development of rock wells. Determining construction information on abandoned wells.	
Dipmeter survey	Determining dip angle and dip direction (from magnetic north) in relation to well axis in the study of geologic structure. Correlation of formations.	Fluid-filled uncased hole. Carefully picked zones needing survey, because of expense and time required. Directional survey required for determination of true dip and strike (generally obtained simultaneously with dipmeter curves).
Directional (inclinometer) survey	Locating points in a hole to determine deviation from the vertical. Determining true depth. Determining possible mechanical difficulty for casing installation or pump operation. Determining true dip and strike from dipmeter survey.	Fluid-filled or dry uncased hole.
Magnetic logging	Determining magnetic field intensity in borehole and magnetic susceptibility of rocks surrounding hole. Studying lithology and correlation, especially in igneous rocks.	Fluid-filled or dry uncased hole.

* Johnson, A. I. An outline of geophysical logging methods and their uses in hydrologic studies. *US Geol. Surv.*, Water Supply Paper 1892, 158–164 (1968)

rocks concerned. The drillhole can contain any fluid, or be empty. However, it must be uncased (*Table 3.4*).

When operating, an alternating current of constant magnitude and frequency is fed to the transmitter coil through an oscillator. The alternating magnetic field from this current induces 'current loops' in the rocks immediately opposite the sonde. These currents, in turn, have a magnetic field of their own, which induces a signal in the receiver coil. The signals are amplified, changed to direct current and transmitted to surface recording equipment. Readings may be taken continuously or the difference in conductivity cells may be recorded.

Induction logging provides an accurate and detailed record of strata over a wide range of conductivity values. Its focusing ability has excellent resolving power and shows almost no distortion opposite thin beds. This is the principal advantage of induction logging over conventional electric logging, that is, its ability to locate thin beds. Good definition can be obtained for beds of only 50 mm thickness. Also the depths to contacts between beds can be determined accurately.

3.5.3 Spontaneous potential logging

A spontaneous potential (SP) log is obtained by measuring the very small differences in electrical potential which exist at the boundaries of permeable rock units and especially

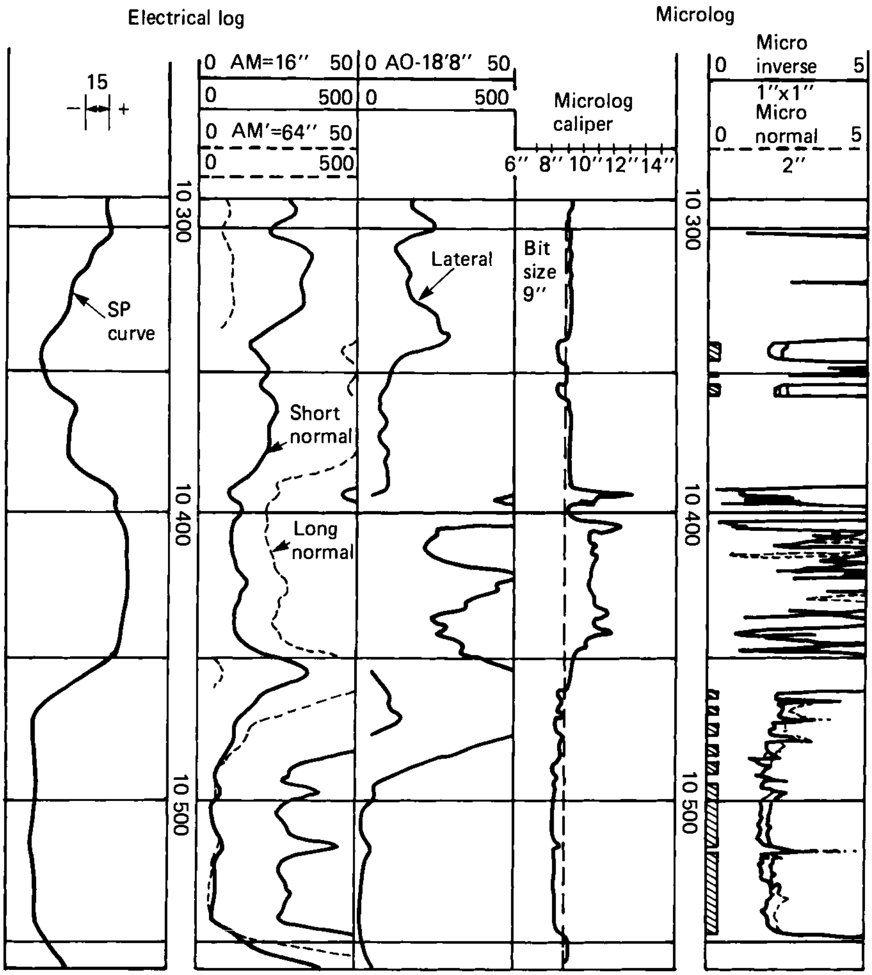

Figure 3.11 Microlog curves. Microresistivity curves are shown at the right. Permeable portions of the section penetrated are indicated (cross-hatched bars) by extensions of the 2" (50 mm) micro-normal curve beyond the microinverse. Note that the diameter of the bore, as recorded by the microlog caliper, is smaller than bit size where a mud filter cake is formed at the position of permeable beds. A standard electrical log of the same stratigraphic interval is shown at the left for comparison (courtesy of Schlumberger Inland Services Inc)

between such data and less permeable beds (*Table 3.4*). According to Brown[42], the SP curve is affected by bed thickness, clay content, the presence of hydrocarbons, natural potentials, instrumental potentials and the ratio of the activity (electrochemical potential) of the drilling mud to the activity of the pore water.

Measurements, usually in millivolts, are obtained from a recording potentiometer connected to two electrodes, one usually being placed in the uncased, fluid-filled drillhole while the other remains in a mud pit at the surface. Since the surface electrode is stationary, its potential is constant. Consequently the SP log represents a record of the variations in potential down the hole. Potential values range from zero to several hundred millivolts. Positive values occur with flow from the formation into the

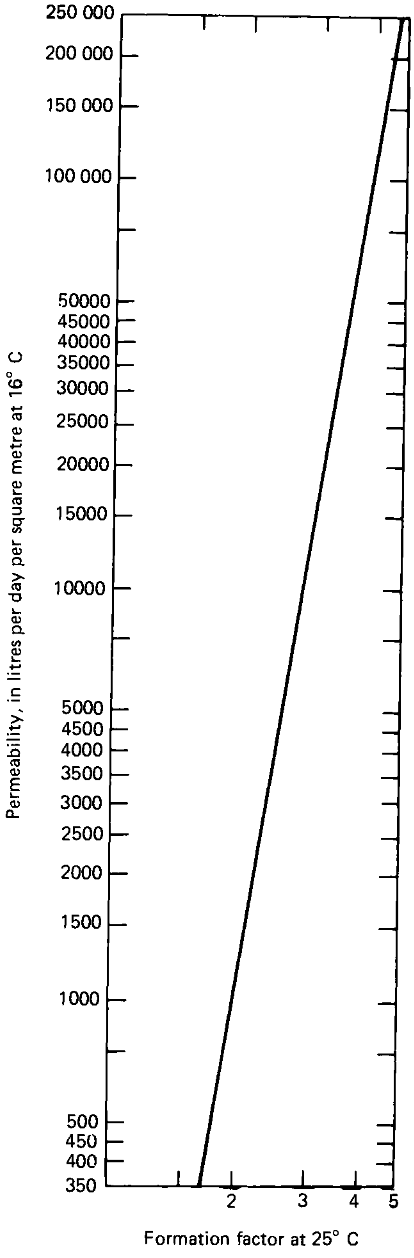

Figure 3.12 Formation factor versus permeability (after Croft[38])

drillhole, negative values for the reverse flow. Consequently, potential logs are read in terms of positive and negative deflections from an arbitrary baseline. For example, the potential recorded opposite a thick bed of shale or clay is conventionally taken as zero. Hence, a line, which is usually termed the shale line, can be drawn on a log, connecting all points of zero potential. Then deflections of the spontaneous potential curve to the left of the shale line are negative, while those to the right are positive.

Permeable sandstones and limestones show large spontaneous potentials. If sandstones and shales are interbedded then the SP curve has numerous troughs separated by sharp or rounded peaks, the widths of which vary in proportion to the thicknesses of the sandstones. Where there are no sharp contrasts in permeability between adjacent beds of rock, SP curves lack relief and are of little value.

Spontaneous potential logs are frequently recorded at the same time as resistivity logs. Interpretation of both sets of curves yields precise data on the depth, thickness and position in the sequence of the beds penetrated by the drillhole. They also enable a semi-quantitative assessment of lithological and hydrogeological characteristics to be made.

3.5.4 Sonic logging

The sonic logging device consists of a transmitter–receiver system, transmitter(s) and receiver(s) being located at given positions on the sonde[42]. The transmitters emit short, high-frequency pulses several times a second and differences in travel times between receivers are recorded in order to obtain the velocities of the refracted waves. The travel times are recorded continuously as the sonde is drawn up the drillhole. The velocity of sonic waves propagated in sedimentary rocks is largely a function of the character of the matrix and their porosity. Normally beds with high porosities have low velocities and dense rocks are typified by high velocities.

In the case of sandstones Pickett[43] showed that sonic velocities are dependent not only on porosity but also on the amount of argillaceous material contained and the pressure differential between overburden and fluid pressures. Hence, usable porosity predictions can be made from sonic drillhole logs if measured velocities are corrected for effects of pressure differential and 'shaliness'. Pickett assumed that a constant proportionality exists between pressure differential and depth and so provides a means of accounting for pressure differential effects on velocity. 'Shaliness' can be accounted for by using the self-potential log.

Previously, Wyllie *et al.*[25] had proposed that the porosity, n, of a formation could be determined from

$$n = \frac{(V_{\log} - V_{m})}{(V_{f} - V_{m})} \tag{3.12}$$

where V_f and V_m are the velocities of the fluid and the matrix respectively. Accurate determinations of porosity can be obtained when a sonic log is used in conjunction with a neutron log. As velocity values vary independently of resistivity or radioactivity, the sonic log permits differentiation amongst strata which may be less evident on the other types of log.

3.5.5 Radioactive logging

Radioactive logs include gamma-ray or natural gamma, gamma-gamma or formation density and neutron logs[44]. They have the advantage of being obtainable in either a cased or uncased drillhole which is empty or filled with fluid, while the various electric and sonic logs can only be used in uncased holes. The natural gamma log provides a record of the natural radioactivity or gamma radiation from elements such as potassium 40, and uranium and thorium isotopes, in the rocks (*Table 3.4*). This radioactivity varies widely amongst sedimentary rocks, being generally high for clays

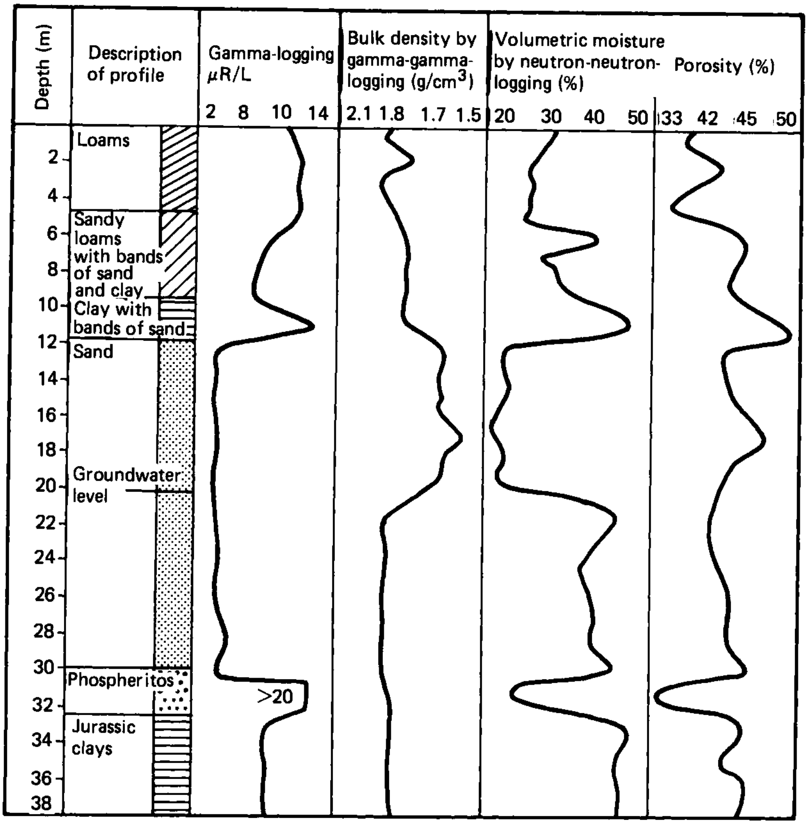

Figure 3.13 Radioactivity/porosity logs (after Feronsky, V. I., 'Nuclear techniques in engineering geology', *Proc. 1st Cong. Engng. Geol. Paris*, 719–731, 1970)

and shales, especially those with high organic contents, and lower for sandstones and limestones. Evaporites give very low readings. In fact, the gamma log essentially distinguishes shales from other formations. When the natural gamma log is considered in association with a resistivity log, it helps to determine the thickness of the beds in a drillhole and aids estimation of porosity and permeability (*Figure 3.13*).

Gamma and resistance logging of drillholes in the Chalk of south Humberside and Lincolnshire enabled Barker *et al.*[45] to recognize marker horizons and thereby to correlate the stratigraphy and locate the major geological structures of possible hydrological significance.

The gamma-gamma log uses a source of gamma rays which are sent into the wall of the drillhole. There they collide with electrons in the rocks and thereby lose energy. The returning gamma ray intensity is recorded, a high value indicating low electron density and hence low formation density, that is, high porosity. In fact, the porosity, R, can be obtained from

$$n=\rho_g-\rho_b/\rho_g-\rho_w$$

where ρ_g is the drain density, ρ_b is the bulk density and ρ_w is the density of the pore water.

The neutron curve is a recording of the effects caused by bombardment of the strata with neutrons during neutron logging of a hole. As the neutrons are absorbed by atoms of hydrogen, which then emit gamma rays, the log provides an indication of the quantity of hydrogen in the strata around the sonde. The amount of hydrogen is related to the water (or hydrocarbon) content and therefore provides another method of estimating porosity. In other words, as most hydrogen is in water, the activity registered on the log is inversely proportional to the water content of the formation about the drillhole. If the formation is saturated, the activity is inversely proportional to the porosity. Since carbon is a good moderator of neutrons, carbonaceous rocks are liable to yield spurious indications as far as porosity is concerned. Generally, therefore, a log of the natural gamma radiation is made to facilitate porosity determination. It also allows clays with a high carbon content to be distinguished.

3.5.6 Temperature logging

The temperature of groundwater encountered in a drillhole is influenced by the geothermal gradient which increases with depth, by the temperature of the groundwater in storage, by any water percolating from the surface and mixing with the groundwater and by the specific heats of the rock masses concerned. Temperature measurements can be taken down either a cased or uncased drillhole and are recorded by a probe suspended from a cable. A continuous reading of temperature is recorded. The differential temperature between two probes, set about 1.5 m apart, also can be recorded. The resulting log has been referred to as a 'delta-log' by Jones and Skibitzke[46] and they pointed out that the principal advantage of its use is that it responds passively to normal thermal gradients and actively to abnormal gradients. Differential temperatures are measured in milli-degrees Celsius whilst the accuracy of measurements made with a single probe is around one-hundredth degree Celsius. According to Tate *et al.*[47], temperature logging should be carried out in a drillhole before any other physical measurements are made.

A temperature log of a well can be used to distinguish the positions of the aquifer or aquifers which it penetrates (*Table 3.4*). The temperature of water from deeper aquifers is usually higher than that from shallower aquifers. Also a change in temperature tends to occur from the base of a thick aquifer upwards. Rapid changes in the almost constant rise of temperature with depth indicate inflows of water into or outflows from a well. In addition thermometry may be used in making detailed studies of the effects of artificial recharge, of the effects of injecting wastes into the ground and of groundwater movement[48].

Jones and Skibitzke[46] stated that for an accurate record of the relative thermal conductivity of the beds penetrated by a drillhole to be made, the temperature log must be taken several days after drilling has ceased in order that the thermal equilibrium between the beds and the drillhole fluid is established. Interpretation of temperature logs in terms of the lithology of the beds present is only possible when equilibrium conditions have been attained. The ability to recognize zones which have a constant thermal gradient depends upon the thickness of the bed and the contrast it offers in thermal conductivity to the surrounding beds. In other words, an alternating sequence of thin beds provides no significant anomaly on a depth–temperature curve.

3.5.7 Fluid conductivity logging

A fluid conductivity log provides a record of the electrical conductivity of the fluid in a drillhole (*Table 3.4*). It is carried out with the aid of a pair of closely spaced electrodes which are lowered down the drillhole on a cable. The fluid conductivity log may be used to locate points of entry or exit from a drillhole of waters of different quality, to locate the interface between fresh and salt water, to correct head measurements for fluid density differences, to locate waste waters and to follow the movement of saline tracers. Barker *et al.*[45] found that they could use the data obtained from fluid conductivity and temperature logs to determine the levels at which marl bands occurred in the Chalk of Lincolnshire. The bands probably acted as confining layers. The conductivity data of the fluid column are also important in interpreting SP, resistivity and neutron logs which may be affected by saline changes.

3.5.8 Caliper logging

Calipers are sensitive devices which measure the diameter of a drillhole[46]. The degree of drillhole diameter variation with depth reflects the relative hardness, competence, water content and, at times, the permeability of the rock mass penetrated, the action of the drilling tools and the extent to which the rocks are eroded by, dissolved in or become hydrated by a fluid-flushing medium. Cavities encountered along the drillhole may mean that it has penetrated a local fracture zone or a solution opening in the rock mass which, in turn, may indicate the presence of a water-bearing zone.

The caliper possesses independently operating measuring arms which ride the wall of a drillhole and detect variations as small as 6 mm in diameter. Because of the independent action of each arm, the diameter recorded is that of the circle described by the tips of the arms. Different types of calipers have different numbers of caliper arms, usually three to six. The reading is an average of all the arms and is recorded at the surface as a single curve.

A caliper log also provides information which can be used for determining the settings of packers in packer testing (*Table 3.4*). In addition, flowmeter investigation in uncased wells yields more accurate results if the velocities are adjusted for drillhole diameter, and the determination of true resistivity from resistivity logs in sequences where thin beds occur requires data on the diameter of the drillhole.

3.6 Maps

Some of the more important maps used in relation to groundwater investigations include geological maps with cross sections, facies maps, structural contour maps and various types of isopachyte maps including depth to aquifer maps. Although it is essential to have a map of the solid geology of the area concerned, a map showing the distribution of superficial deposits (i.e. a drift map) is also of value. The type of superficial deposits present and their thickness influence groundwater recharge[17]. For instance, the probability that surface and groundwater resources are interconnected is appreciably higher in areas covered by thin, permeable deposits than where there are thick deposits of dense till. Moreover, the thickness of superficial deposits can influence the location of a well and their type can influence the method of drilling employed.

Fine-grained sedimentary rocks represent aquicludes and as such form barriers to groundwater movement. Their regional distribution and position in the stratigraphical sequence is, therefore, important. In this context the likelihood of a change in facies is important, as is the occurrence of intrusive igneous rocks and major faults. Indeed a facies map may be of value in helping to locate productive wells.

Lloyd[15] pointed out that the construction of structural contour maps depends on the interpretation of solid geological maps and associated cross sections. The choice of a suitable surface for representation by structural contours depends on the complexity of

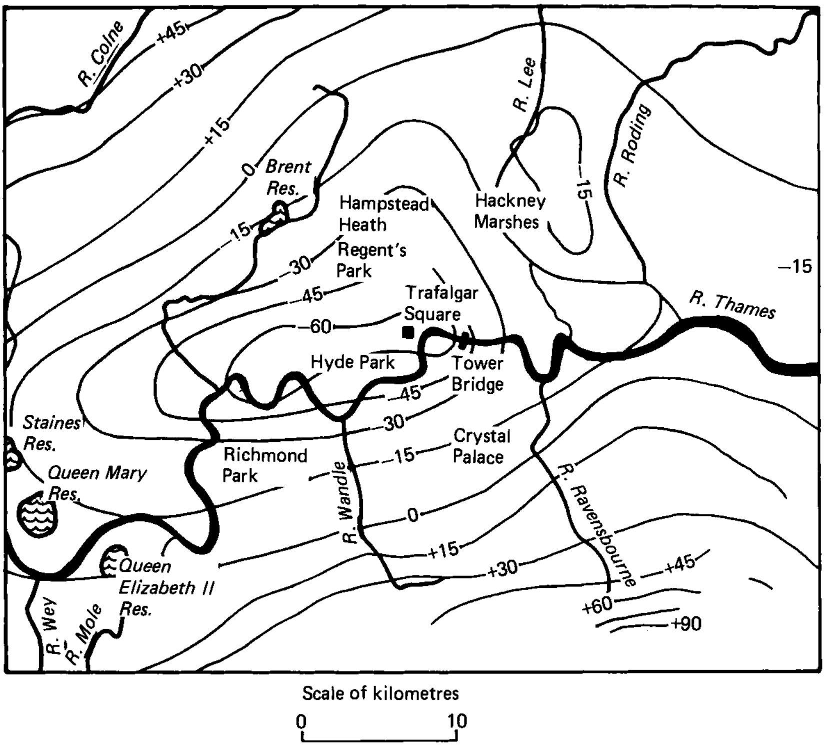

Figure 3.14 1980 groundwater levels in the London Basin (all contours in metres relative to Ordnance Datum) (after March, T. J. and Davies, P. A., 'The decline and partial recovery of the groundwater levels below London', *Proc. Inst. Civ. Engrs.*, Part 1, **74**, 263–276 (1983)

the geological conditions. The upper and lower surfaces of an aquifer generally are the surfaces selected. Structural contour maps help locate the positions of drillholes.

Isopachyte maps can be drawn to show the thickness of a particular aquifer or the depth below the surface of a particular bed. They can be used together with structural contour maps to estimate the depths of drillholes. They also provide an indication of the distribution of potential aquifers.

Maps showing groundwater contours (*Figure 3.14*) are compiled when there is a sufficient number of observation wells to determine the configuration of the water table. Data on surface water levels in reservoirs and streams that have free connection with the water table also should be used in the production of such maps. These maps are usually compiled for the periods of the maximum, minimum and mean annual positions of the water table. A water table contour map is most useful for studies of unconfined groundwater.

As groundwater moves from areas of higher potential towards areas of lower potential and as the contours on groundwater contour maps represent lines of equal potential, the direction of groundwater flow moves from highs to lows at right-angles to the contours. Analysis of conditions revealed by groundwater contours is made in accordance with Darcy's law. Accordingly spacing of contours is dependent on the flow rate and on aquifer thickness and permeability. If continuity of flow rate is assumed, then the spacing depends upon aquifer thickness and permeability. Hence areal changes in contour spacing may be indicative of changes in aquifer conditions. However, because of the heterogeneity of most aquifers, changes in gradient must be carefully interpreted in relation to all factors. The shape of the contours portraying the position of the water table helps to indicate where areas of recharge and discharge of groundwater occur. Groundwater mounds can result from the downward seepage of surface water. In an ideal situation the gradient from the centre of such a recharge area will decrease radially and at a declining rate. An impermeable boundary or change in transmissivity will affect this pattern.

Piezometric surface maps are similar to water table contour maps except that they are based on the piezometric potential developed in an aquifer, as measured by piezometers.

Depth to water table maps show the depth to water from the ground surface. They are prepared by overlaying a water table contour map on a topographical map of the same area (and scale) and recording the values at the points where contours intersect. Depth to water contours are then interpolated in relation to these points. A map indicating the depth to the water table can also provide an indication of areas of recharge and discharge. Both are most likely to occur where the water table approaches the surface.

Water level change maps are constructed by plotting the change in the position of the water table recorded at wells during a given interval of time (see *Figure 9.6*). The effect of local recharge or discharge often shows as distinct anomalies on water level change maps. For example, a water level change map may indicate that the groundwater levels beneath a river have remained constant while falling everywhere else. This would suggest an influent relationship between the river and aquifer. Hence, such maps can help identify the locations where there are interconnections between surface water and groundwater. These maps also permit an estimation to be made of the change in

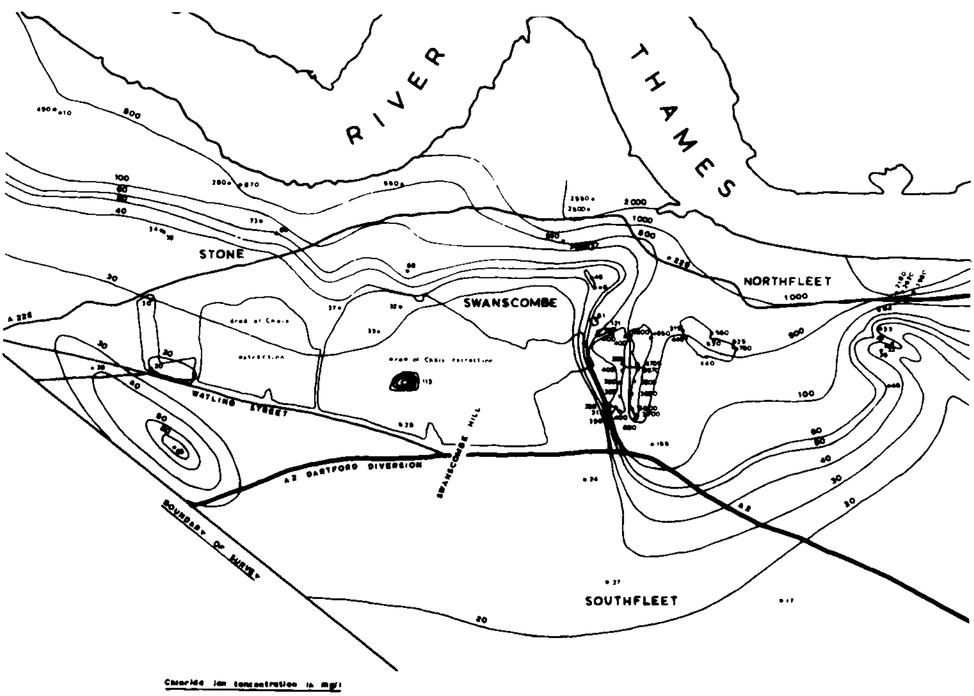

Figure 3.15 Chloride ion concentration in north Kent, 1972 (courtesy of The Southern Water Authority)

groundwater storage which has occurred during the lapse in time involved. In other words, multiplication of the change in volume by the porosity of the aquifer provides some indication of the change in gross storage.

According to Brown *et al.*[49], groundwater flow may be portrayed on a regional basis. The associated maps show groundwater flow either as isopleths of mean long-term infiltration (in $l/s/km^2$) or as coefficients of groundwater flow (that is, percentage of precipitation). These maps indicate the distribution of mean long term, or the absolute, values of discharging groundwaters over a drainage basin and the amount of the total precipitation which recharges the groundwater supply. The likely aquifer yield within the drainage basin, as well as the quality of the water, can be shown on such maps.

Maps indicating water quality may either show the total dissolved solids content of groundwater or the content of a certain chemical component (e.g. chlorine or sulphate ions). These are depicted by isopleths (*Figure 3.15*).

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Chapter 4

Assessment of aquifer recharge and potential well yield

4.1 Introduction

The assessment of the yield of a groundwater resource is more difficult and more subjective than the comparable calculation for a surface water resource. This is largely due to the fact that the underground reservoir or catchment is invisible from the ground surface and so the determination of its lateral and vertical dimensions must be accomplished by indirect means such as geophysical investigation or borehole exploration, interpolation and guesswork. Once aquifers have been located and their physical properties assessed, the data may be presented in the form of maps. Typically maps would be prepared showing the variation of the coefficients of storage and transmissivity in the study area. These factors partly determine the ease with which water can flow to a well, hence such maps are useful in helping to locate potential well sites.

Because of the problems inherent in determining the characteristics of an aquifer, it is inevitable that estimates of groundwater yield will be subjective and approximations to the truth. Consequently it is always advisable to use more than one method of calculation and, depending upon the margin of safety required, adopt either the smallest or the average value of yield obtained. However, even if the available methods of analysis are not particularly accurate, it should be appreciated that any estimate of potential yield, even if it contains an error of ± 30 per cent, is better than no estimate at all when trying to decide whether or not it is feasible to develop an aquifer for water supply.

To understand fully the methods that may be employed to determine the yield of an aquifer or well, it is first necessary to have some knowledge of the hydrological cycle. It is this cycle that provides the source of the vast majority of groundwater and governs the rate of groundwater recharge and thus determines the maximum rate at which it may be abstracted. A knowledge of the complex processes of groundwater recharge and discharge is a prerequisite of aquifer yield estimation and this, in turn, requires a quantitative knowledge of the components that form the hydrological cycle.

4.2 The hydrological cycle

The hydrological cycle involves the movement of water in all its forms over, on and through the Earth (*Figure 4.1*). The cycle can be crudely visualized as starting with the

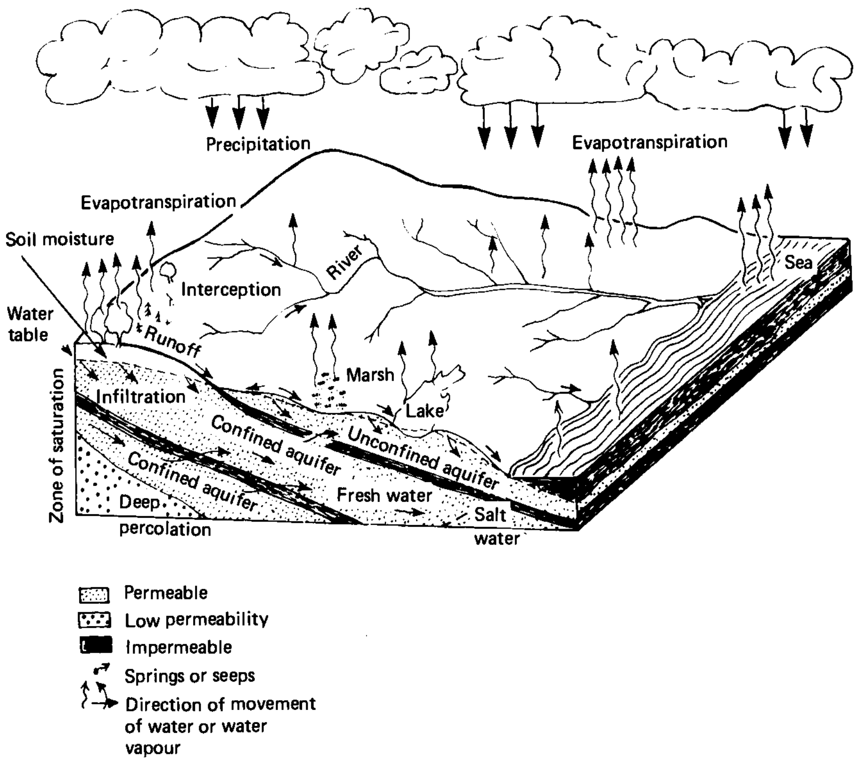

Figure 4.1 The hydrological cycle

evaporation of water from the oceans and the subsequent transport of the resultant water vapour by winds and moving air masses. Some water vapour condenses over land and falls back to the surface of the Earth as precipitation. To complete the cycle this precipitation must then make its way back to the oceans via streams, rivers or underground flow, although some precipitation may be evapotranspired and describe several subcycles before completing its journey[1].

The circulation of water within the hydrological cycle is an extremely slow process and it may take many thousands of years for a particle of groundwater to be transmitted from the recharge area through the aquifer and eventually to the sea. The groundwater in parts of the Chalk of the London Basin, for instance, has been estimated to have an 'age' in excess of 25 000 years[2]. Consequently the cycle can be regarded as a series of storage components, with water moving slowly from one to another[3] until one circuit has been completed. *Table 4.1* shows the estimated amount of water available within the various storage components and the total quantity that would be available if it could all be released from storage. Only 0.5 per cent of the total water resources of the world is in the form of groundwater. According to Huisman[5] not all this is available for exploitation since about half is below 800 m and therefore is too deep for economic utilization. However, the capacity of the underground resource should not be underestimated; about 98 per cent of the usable fresh water of the Earth is stored underground. By any standard 7000×10^{12} m^3 is a lot of water.

The hydrological balance in a catchment (or drainage basin) over a period of less

TABLE 4.1. The water inventory of the Earth (from Nace[4])

Storage component	*Volume of water* (10^{12} m^3)	*Total water* (%)	
Oceans	1 350 400	97.6	
Saline lakes and inland seas	105	0.008	
Ice caps and glaciers	26 000	1.9	
Soil moisture	150	0.01	
Groundwater	7 000	0.5	Usable fresh water = 0.51%
Freshwater lakes	125	0.009	
Rivers	2	0.0001	
Atmosphere	13	0.0009	
TOTAL	1 384 000	100	

All figures are approximate estimates and rounded

than one year in duration can be represented by the equation

$$\text{Recharge} = \text{Discharge} + \text{Change in storage} \tag{4.1}$$

This can be broken down into the following components[1,6,7]

$$P = AET + R_s + R_g + U + dS_m + dS_g + dS_s \tag{4.2}$$

where P is precipitation, AET is actual evapotranspiration, R_s is direct surface run-off, R_g is groundwater discharge, including interflow, U is underflow (groundwater flow at depth through one or more aquifers), dS_m is change in soil moisture storage, dS_g is change in groundwater storage and dS_s is change in surface water storage.

The solution of this equation may require the introduction of other parameters, such as artificial discharge and recharge. It is doubtful if there are many catchments with sufficient instrumentation to allow this equation to be accurately evaluated, so several simplifying assumptions are usually introduced. If the hydrological balance is calculated over an interval of time which has been carefully selected so that the values of soil moisture storage and surface water storage are almost the same at the beginning and end of the balance period, then the terms dS_m and dS_s may be ignored. An additional simplification can be introduced by assuming that the surface and groundwater watersheds (divides) of the catchment coincide. This assumption may be reasonable for very large areas, but it is unlikely to be valid with small catchments. Nevertheless, it allows the underflow term, U, to be omitted from Equation (4.2). Finally, if it is assumed that the only groundwater discharge is to the river system so that it is included in river flow records and

$$R_0 = R_s + R_g \tag{4.3}$$

where R_0 is the totel run-off, including the groundwater component of river discharge, then Equation (4.2) becomes

$$P = AET + R_0 + dS_g \tag{4.4}$$

It is, however, possible to make one further simplification. If the groundwater levels are the same at the beginning and end of the balance period, the term dS_g has a value of zero. The same result can be obtained by calculating the hydrological balance over a period of many years, in which case, short-term or seasonal fluctuations in ground-

TABLE 4.2. Long-term annual water budget of Great Britain and the continents (after Barry[8] and Anon.[9])

Country	*Precipitation* $\bar{P}$ (mm/yr)	*Total run-off* $\bar{R}_0$ (mm/yr)	*Actual evapo-transpiration* $\overline{AET}$ (mm/yr)	$\overline{AET}/\bar{P}$ (%)
Britain	1050	650	400	38.1
Africa	670	160	510	76.1
Asia	610	220	390	63.9
Europe	600	240	360	60.0
N. America	670	270	400	59.7
S. America	1350	490	860	63.7
Australia and New Zealand	470	60	410	87.2
Mean value derived after weighting according to area	725	243	482	66.5

water level, around a relatively static mean value, can be ignored. In the *long-term* the recharge to an aquifer balances the discharges so that a state of equilibrium exists and mean water levels are approximately constant. Thus

$$\bar{P}=\overline{AET}+\bar{R}_0 \tag{4.5}$$

where the overbars represent the long-term average values. Even in this modified form it is doubtful if there are many catchments with sufficient instrumentation and which comply adequately with all the simplifying assumptions, to allow the equation to be accurately evaluated. Nevertheless, Equation (4.4) can be used to estimate annual groundwater recharge (Section 4.5.2) while Equation (4.5) has been used to investigate the water balance of the continents[8] (see *Table 4.2*). Although local conditions and climatic variations are likely to be the most important factors in determining the hydrological balance of any particular area, *Table 4.2* does illustrate the relative magnitude of precipitation, actual evapotranspiration and run-off in various regions of the world. Perhaps the most striking fact to emerge from the data is the significance of evapotranspiration and the very high proportion of precipitation that is returned to the atmosphere as a result of this process.

4.2.1 Precipitation

There are four major types of precipitaton, namely, drizzle, rain, snow and hail. With the exception of high latitude or high altitude regions, rain tends to be the most important form of precipitation. During winter in some temperate latitudes, however, snow can be as important or more important than rain, and when it melts suddenly it releases large volumes of water. However, it is impractical to record the depth of snowfall so that generally the water equivalent is used as a measure of the precipitation fallen, that is, roughly 10 to 12 mm depth of snow equals 1 mm depth of water.

The rate at which groundwater is replenished (and hence the rate at which it can be used safely) is basically dependent upon the quantity of precipitation falling on the recharge area of an aquifer, although rainfall intensity is also very important. Frequent rainfall of moderate intensity is more effective in recharging groundwater resources than short concentrated periods of high intensity. This is because the rate at which the ground can absorb water is limited, any surplus water tending to become run-off or, as it is sometimes called, overland flow.

4.2.2 Evapotranspiration

Not all the precipitation that falls on a groundwater recharge area actually becomes groundwater. Some will run off the surface into streams and rivers and some will be lost through evaporation and transpiration.

In the context of the hydrological cycle, evaporation involves the conversion of the solid or liquid precipitation that reaches the Earth's surface into water vapour and thereby its return to the atmosphere. Transpiration is the water loss from plants and occurs when the vapour pressure in the air is less than that in the leaf cells. During the process, water is transferred through the soil by capillary action, then from the soil through the roots of plants by osmosis and thence to their leaves. Due to the fact that the processes of evaporation and transpiration are difficult to separate, the term evapotranspiration (ET) is generally used to describe the combined process. It has been estimated that approximately 66 per cent of the precipitation that falls on the continents is returned to the atmosphere through ET[8], although in Britain this figure is below 40 per cent (*Table 4.2*). In water resource studies ET is often called the 'water loss', since it represents that part of precipitation which is not available for water supply.

The rate at which water can be lost from a surface through ET is obviously dependent to some extent upon the amount of water that is present in the soil. Thus, potential evapotranspiration (PET) is defined as the maximum amount of water that would be removed from the land surface by ET if sufficient water was available in the soil to meet demand. If there is insufficient water available, then the actual evapotranspiration (AET) is the amount of water evapotranspired under the existing conditions. AET rates may approach the potential value in wet marshy or swampy conditions, or where the groundwater level is near the land surface as in areas of groundwater discharge. Elsewhere AET is about 50 to 90 per cent of the potential value. Just how, when and why the ET rate switches from the potential to the actual rate is one of the most debated issues in soil physics.

4.2.3 Surface run-off

Run-off has been defined as the natural gravity movement of water on the Earth's surface, and, together with precipitation and evapotranspiration, forms one of the three major components of the hydrological cycle. Run-off originates as precipitation and terminates in the sea, but while precipitation is spasmodic, in Britain run-off is generally continuous. One reason for this is that run-off is made up of two basic components, surface water run-off and groundwater discharge. The former is usually the more important and is responsible for the major variations in river flow such as floods. It generally increases in magnitude as the time from the beginning of precipitation increases. This can be represented on a hydrograph as shown in *Figure 4.2*. However, in a hydrograph the surface flood water is superimposed on the flow that existed in the river before precipitation started. This original flow is called the baseflow. The baseflow of a stream is the discharge when the surface run-off and the rate of depletion of channel storage have become negligible. It differs from the groundwater discharge only by the amount lost due to evaporation and transpiration[6,10].

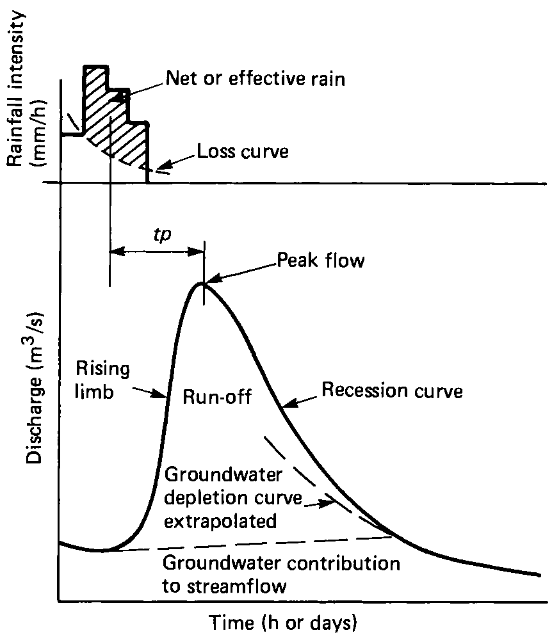

Figure 4.2 Component parts of a hydrograph. When rainfall commences there is an initial period of interception and infiltration before any measurable run-off reaches the stream channels. During the period of rainfall these losses continue in a reduced form so that the rainfall graph has to be adjusted to show effective rain. When the initial losses are met, surface run-off begins and continues to a peak value which occurs at time, *tp*, measured from the centre of gravity of the effective rain on the graph. Thereafter, surface run-off declines along the recession limb until it disappears. Baseflow represents the groundwater contribution along the banks of the river

4.3 Natural groundwater recharge

The proportion of rain water that manages to gravitate to the water table may be referred to as natural groundwater recharge. This term is adopted to emphasize that the process under consideration is a natural part of the hydrological cycle. This is totally different from artificial groundwater recharge, in which water is pumped *down* wells or spread on the ground surface so as to induce infiltration (see Section 9.3, p. 296).

4.3.1 Infiltration and percolation

Infiltration is the name given to the process whereby water penetrates the ground surface and starts moving down through the zone of aeration. The subsequent gravitational movement of the water down to the zone of saturation is termed percolation, although there is no clearly defined point where infiltration becomes percolation. Most groundwater is derived from precipitation, so infiltration and percolation are the means by which water reaches the water table[11]. However, not all the precipitation that falls on to a land surface becomes infiltration. Some is intercepted by the vegetation cover and never reaches the ground, while some proportion is lost as evapotranspiration. After this, however, any further rain is then available to saturate the soil surface and must either infiltrate the surface layer or run off the surface towards a stream channel. Whether infiltration or run-off is the dominant process at a particular time depends upon several factors, such as the intensity of the rainfall and the porosity and permeability of the surface. For example, if the rainfall intensity is much greater than the infiltration capacity of the soil, then run-off is high. On the other hand, if the rainfall is fairly gentle, then infiltration may be increased at the expense of run-off.

Figure 4.3 illustrates the situation where rain of intensity, *i*, mm/h is falling on to a

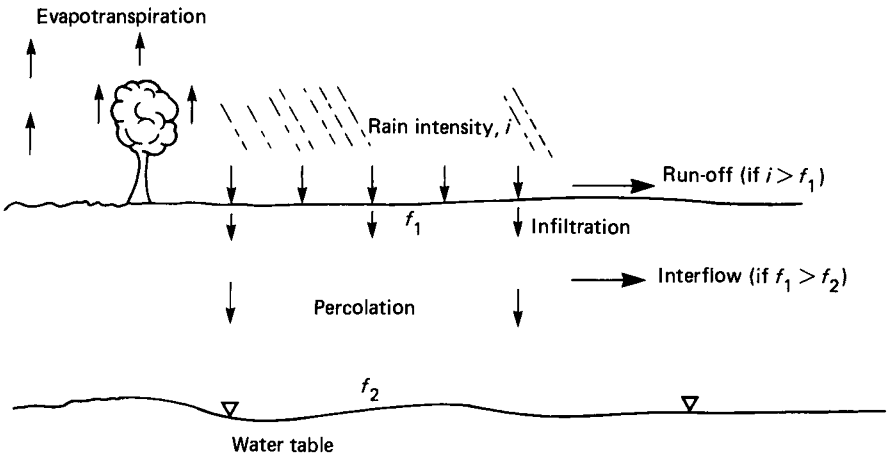

Figure 4.3 Diagrammatic representation of infiltration. If the rainfall intensity is much greater than the infiltration capacity, then run-off may be predominant. However, if the rainfall intensity is less than the infiltration rate, infiltration may be favoured at the expense of run-off. If the infiltration capacity of the formation decreases with depth, some of the percolating water may become interflow and move laterally above the water table

surface that has an infiltration capacity, f_1, mm/h. If i exceeds f_1 then surface run-off occurs, but if i is less than f_1 most of the rain becomes infiltration. However, if the lower strata are less permeable than the surface layer, the infiltration capacity, f_2, will be reduced so that some of the water that has penetrated the surface moves parallel to the water table and is called interflow. The water that becomes interflow will probably be discharged to a river channel at some point and forms part of the baseflow of the river. The remaining water may continue down through the zone of aeration until it reaches the water table and becomes groundwater recharge. This can be a slow process (typically about 1 m/year), since the percolating water may become temporarily suspended in the zone of aeration as a result of the various dynamic forces that operate in this region.

Other important factors that influence infiltration are the initial soil moisture content and time. If the soil is dry and there is a substantial soil moisture deficit, the potential storage capacity of the soil is relatively high and infiltration is encouraged. However, the infiltration rate will decrease with time as the soil moisture is progressively diminished and the soil becomes wetter from the surface downwards. As the soil becomes saturated, surface run-off increases and becomes more important.

Although infiltration may be high in a dry soil, the fact that the soil is dry means that water is more likely to be held in the surface layers of the soil and either evaporated or transpired by plants and therefore less likely to reach the water table. Consequently in Britain around 80 per cent of groundwater recharge takes place during winter and early spring when the ground is comparatively wet and evapotranspiration is relatively insignificant. During the summer evapotranspiration rates are high, so that much of the rain that falls probably does not have the opportunity to percolate down to the water table. When evapotranspiration exceeds precipitation and vegetation has to draw on reserves of water in the soil to satisfy transpiration requirements, soil moisture deficits occur. The soil moisture deficit (SMD) at any time is the difference between the

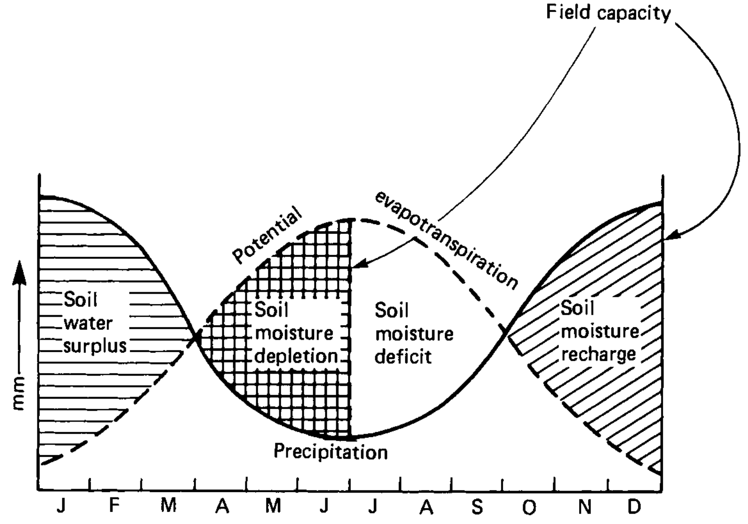

Figure 4.4 Schematic diagram of the relationship between potential evapotranspiration (PET) and precipitation in Britain. In spring and summer PET usually exceeds precipitation so that excess soil water is depleted, and when the soil moisture content falls below field capacity a soil moisture deficit (SMD) is created. Excess soil water will not exist again until precipitation exceeds PET and the accumulated SMD has been recharged

moisture remaining and the field capacity of the soil, which is the amount of water retained in the soil by capillary forces after excess water has been drained from it.

A simple inverse cyclic relationship between precipitation and potential evapotranspiration (PET), which is typical of Britain, is shown diagrammatically in *Figure 4.4*. In the UK evapotranspiration often exceeds precipitation during the summer months, while by early autumn the situation is reversed. The fact that precipitation now exceeds evapotranspiration does not, however, result in an immediate rise in groundwater levels. Before this can happen, any soil moisture deficit must be made up so that the soil is returned to its field capacity. Once this has been achieved, then any additional or excess water may become groundwater recharge.

The importance of winter precipitation with respect to infiltration and groundwater recharge can be verified by considering the results from infiltrometers or percolation gauges. The results from gauges at Harrogate and Papplewick (*Table 4.3*) show that during the winter half year, on average, 76 per cent of the available precipitation was recorded as percolation. During the summer half year the equivalent figure was 24 per cent. The table also indicates that on average 77 per cent of the total annual percolation was observed in the winter half year, and that about 50 per cent of the available annual precipitation was recorded as percolation.

4.3.2 The water balance technique

Percolation gauge results of the sort shown in *Table 4.3* are very useful for indicating which months of the year are most significant in relation to groundwater recharge at a given location. However, in many localities these data are not available. As an alternative means of investigation, the water balance at a particular site may be calculated. Thornthwaite[13] and Penman[14] separately developed an accounting

TABLE 4.3. Observed average percolation

(a) *Harrogate, West Yorkshire, 1916–50* (based on Smith[12])

	Oct	*Nov*	*Dec*	*Jan*	*Feb*	*Mar*	*Apr*	*May*	*Jun*	*Jul*	*Aug*	*Sep*	*Year*
Precipitation, P (mm)	75	79	71	80	62	49	53	62	51	72	74	63	791
Percolation, f (mm)	30	53	54	57	47	29	22	15	11	13	14	19	364

Winter precipitation = 416 mm
Winter percolation = 270 mm
Proportion of winter precipitation becoming percolation = 65 per cent
Proportion of annual percolation occurring during winter = 74 per cent

Summer precipitation = 375 mm
Summer percolation = 94 mm
Proportion of summer precipitation becoming percolation = 25 per cent
Proportion of annual percolation occurring during summer = 26 per cent

Proportion of annual precipitation becoming percolation = 46 per cent

(b) *Papplewick, Nottinghamshire, 1925–66* (based on Skeat[7])

	Oct	*Nov*	*Dec*	*Jan*	*Feb*	*Mar*	*Apr*	*May*	*Jun*	*Jul*	*Aug*	*Sep*	*Year*
Precipitation, P (mm)	64	74	58	66	48	46	48	51	51	64	66	56	692
Percolation, f (mm)	33	64	56	66	48	41	25	10	5	10	13	13	384

Winter precipitation = 356 mm
Winter percolation = 308 mm
Proportion of winter precipitation becoming percolation = 87 per cent
Proportion of annual percolation occurring during winter = 80 per cent

Summer precipitation = 336 mm
Summer percolation = 76 mm
Proportion of summer precipitation becoming percolation = 23 per cent
Proportion of annual percolation occurring during summer = 20 per cent

Proportion of annual precipitation becoming percolation = 55 per cent technique that kept a running balance of the three variables, precipitation, PET and soil moisture deficit (SMD). Thus the water balance of a particular location is obtained by calculating the arithmetic difference, either positive or negative, between precipitation and PET for each month throughout the year.

An example of the application of the Penman technique was given by Smith[12], see *Table 4.4*. It is apparent from the table that the SMD can be thought of as the *accumulated* difference or deficit between precipitation and PET, and consequently the SMD continues to increase in magnitude for as long as PET continues to exceed precipitation. When precipitation again exceeds PET, the SMD is reduced by the arithmetic difference between these two variables, but the SMD is not actually ended until the accumulated excess of precipitation over PET equals or exceeds the SMD value. When the SMD has ended, excess soil water exists and further precipitation becomes run-off. In this instance it has been assumed that most plants could draw upon 76 mm of water before the moisture reserves of the soil became exhausted and a water deficiency resulted. However, values other than 76 mm may be adopted depending upon the conditions at the site under investigation. The results from the table are plotted graphically in *Figure 4.5*, and it is apparent from the diagram that there is a distinct water surplus during the winter months, with a period of soil moisture utilization during the summer. In Britain, soils generally reach field capacity during winter and spring and the soil moisture deficit is eliminated somewhere around October although exceptionally it may be carried through to the following summer. Such an occurrence is illustrated by *Table 4.5* which shows the deviation from field capacity at Oxford during several drought periods[15]. It can be seen, for example, that in 1964 there was still a SMD of 47 mm at the end of the year. Consequently there would be no groundwater recharge during the autumn of 1964 at this location.

The water balance technique can provide a useful insight into both groundwater recharge and aquifer characteristics. For instance, the winter water surplus and summer soil moisture deficit means that fluctuations in groundwater level usually follow an annual pattern, with the maximum elevation of the water table being recorded between January and May and the minimum value between September and November (*Figure 4.6*). The total seasonal variation in groundwater level may range from less than 1 m to over 30 m, depending on the aquifer material. Usually the largest fluctuations occur in materials of low porosity where a small volume of recharge produces a large increase in saturated thickness. Additionally, wells located in high, hilly areas tend to produce greater seasonal variations in water level than those in river valleys. According to Smith[12], this is because there tends to be a more rapid lateral movement of groundwater in river valleys which reduces the seasonal rise and fall in level. Additionally, there is often a hydraulic connection between a river and the underlying aquifer so that fluctuations of the water table become damped (*Figure 4.6*). For instance, groundwater recharge which could cause an appreciable rise in water level may discharge or spill into the river system instead, so that the subsequent rise in the water table is small. During periods of low groundwater levels the river may recharge the aquifer, thus preventing any further decline.

4.4 Natural groundwater discharge

Under natural conditions most aquifers discharge either directly or indirectly to rivers and seas by way of seepage and springs. This is distinct from artificial discharge which

TABLE 4.4. Water balance for Harrogate—Penman method (after Smith[12])

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Year
1. *Precipitation*	80	62	49	53	62	51	72	74	63	75	79	72	792
2. *PET*	4	8	29	46	72	86	84	70	41	18	4	3	465
3. *Storage change*	0	0	0	0	−10	−35	−12	+4	+22	+57	0	0	
4. *SMD*	0	0	0	0	10	45	57	53	31	0	0	0	
5. *Storage balance*	76	76	76	76	66	31	19	23	45	76	76	76	
6. *Water deficiency*	0	0	0	0	0	0	0	0	0	0	0	0	
7. *Water surplus*	76	54	20	7	0	0	0	0	0	26	75	69	327
		157									170		

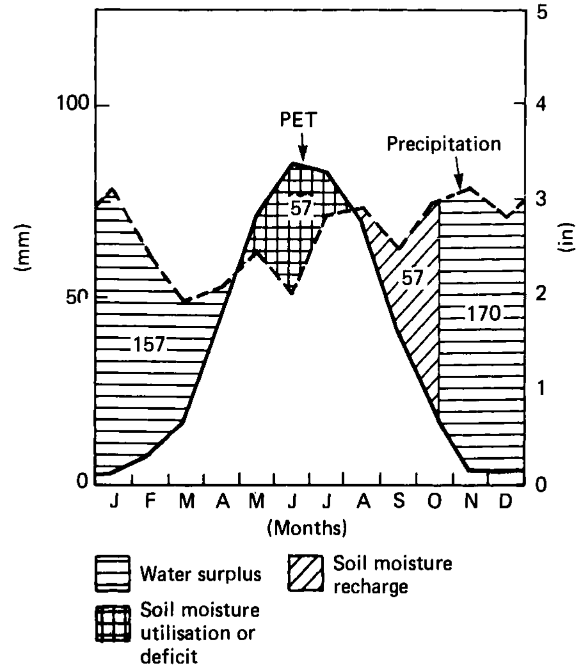

Figure 4.5 Mean monthly water balance (mm) for Harrogate, West Yorkshire, England, computed according to the Penman method (from Smith[12])

TABLE 4.5. Deviations from field capacity at Oxford during several drought periods (after Rodda[15])

Year	*Jan*	*Feb*	*Mar*	*Apr*	*May*	*June*	*July*	*Aug*	*Sep*	*Oct*	*Nov*	*Dee*
1921	+59	−1	−12	−47	−98	−110	−114	−105	−105	−102	−57	−30
1922	+25	+43	+17	+21	−60	−102	−89	−17	−28	−41	−19	+41
1933	+38	+70	+9	−25	−57	−103	−107	−109	−103	−86	−64	−63
1934	−22	−33	−21	−14	−83	−105	−110	−106	−97	−78	−42	+97
1943	+90	+9	−17	−44	−78	−104	−109	−104	−103	−58	−30	−10
1944	+22	+11	−33	−72	−106	−105	−102	−105	−91	−51	+35	+33
1962	+87	−3	−6	−4	−44	−106	−105	−81	−17	−9	+43	+42
1963	+22	−3	+49	+6	−39	−85	−104	−67	−58	−29	+80	+13
1964	+9	+11	+74	+13	−37	−64	−105	−109	−107	−103	−84	−47

1. Positive values indicate the amount of run-off or infiltration (mm) for months when the soil was at field capacity
2. Negative values indicate the amount of soil moisture deficit (mm)

takes place as a result of man's intervention in the natural cycle of groundwater movement.

4.4.1 Groundwater discharge to rivers

The most common form of groundwater discharge is that to a river. The groundwater discharge which becomes the baseflow of a river, is the outflow from unconfined or artesian aquifers bordering the river, which go on discharging more and more slowly with time as the differential head falls. The hydrograph of baseflow is near to an

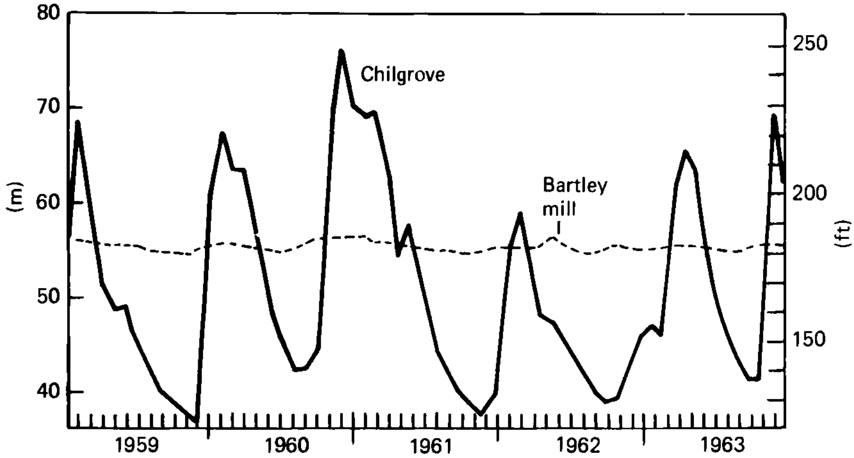

Figure 4.6 Hydrographs for two wells in Sussex, England, over the period 1959–63. The Chilgrove well is located half-way up the dip slope of the South Downs. The Bartley Mill well is on the flood plain of the river Teise and is in hydraulic contact with the river. Consequently, its fluctuations in level are heavily damped (after Lovelock *et al.*[16])

exponential curve[17,18], and the quantity at any time, t, may be approximated by

$$Q_t = Q_0 e^{-\alpha t} \tag{4.6}$$

where Q_t is the discharge at the end of a period, Q_0 is the discharge at the start of the period, α is the coefficient of the aquifer and e is the base of natural logarithms.

Several methods are available for separating the surfaces and groundwater components of natural hydrographs[17]. However, these techniques assume a rather simple relationship between the two components as in *Figure 4.2*, whereas in reality the water enclosed by the hydrograph is derived from many sources, such as direct surface run-off, interflow, bank storage and groundwater discharge, as shown in *Figure 4.7*. Bank storage is the water temporarily held in store in the ground adjacent to the river between the low- and high-water levels. This water is released as the river level falls.

The lower part of the recession limb of a hydrograph, the groundwater depletion curve (*Figure 4.2*), represents the surplus groundwater resource of the catchment or drainage area[19,20,21,22]. The recession curve can, therefore, be used to investigate the contribution of an aquifer to riverflow, or in other words, to assess the magnitude of groundwater discharge from an aquifer (see Section 4.5.3).

According to Barry[8], about 30 per cent of the total annual riverflow worldwide is the result of groundwater discharge, although this amount varies considerably within different geographical zones. Some idea of the influence of surface geology on the groundwater component of river discharge may be derived from *Table 4.6*. Indeed the schematic subdivision of a hydrographic record into surface and groundwater components (*Figure 4.8*) can provide a hydrogeologist with some idea of the geology of a catchment. For example, in a chalk region where there are few surface streams the groundwater contribution is quite high, whereas in a catchment covered by impermeable material the surface water component may form the major part of the hydrograph. Smith[12] claimed that the Chalk catchment of the River Itchen in

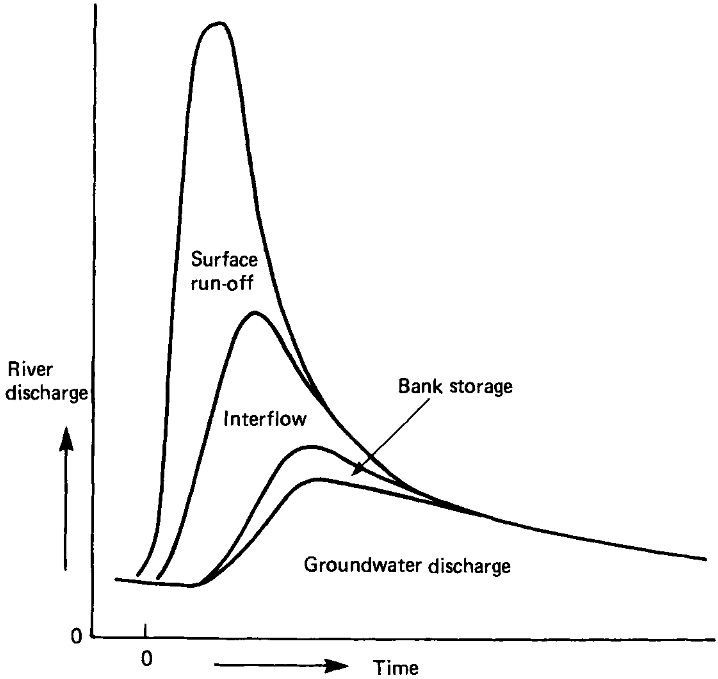

Figure 4.7 Schematic diagram of the components of river discharge

Hampshire resulted in a groundwater component of between 77 and 90 per cent of the total discharge in any year.

The groundwater contribution to streamflow is most important in summer, when surface run-off is reduced as a result of soil moisture deficits. During the summer months a very high proportion of streamflow may be derived from groundwater sources. In fact it may be the groundwater contribution to flow during this period that prevents the stream from drying up. A watercourse that maintains a continuous flow throughout the year is referred to as a perennial stream, while those that dry up periodically are known as ephemeral or intermittent streams. In the Chalklands of England such intermittent streams are quite numerous and are often called bournes. The seasonal nature of these streams is often due to the annual fluctuation of groundwater levels, which can be quite large in the Chalk, as shown in *Figure 4.6*. Thus, when the water table is above the level of the watercourse during winter and spring, the stream is fed either directly or indirectly by a series of springs. When the water table is below the level of the watercourse, generally during summer and autumn, the stream disappears.

4.4.2 Springs

Springs develop at the points where underground conduits discharge water at the surface. Water rarely moves uniformly throughout an entire rock mass and most springs therefore issue as concentrated flows. Springs which percolate from many small openings have been termed seepage springs and they may discharge so little water that they are barely noticeable. By contrast, large springs have often been used as sources of water supply.

TABLE 4.6. Influence of surface geology on the groundwater component of river discharge (after Skeat[7])

Catchment	*Area of surface catchment* (km^2)	*Lithology of major deposits*		*Period*	*Groundwater component expressed as percentage of total river discharge*
		Surface deposits	*Solid strata*		
Stour	855	Clay	Chalk, sands and gravels	1935–49	38
Nidd (Hunsingore)	484	Clay	Sandstone and limestones	1961–62	32
Swale	381	Clay	Sandstones, shales, and limestones	1961–63	32
Itchen	360*	Chalk	Chalk	1958–62	83
Gipping	329	Clay	Chalk	1961–65	39
Tone	202	Sandstones, marls, and shales	Sandstones, marls, and shales	1961–65	45
Nar	163*	Chalk and clay	Chalk	1956–60	72
Tas	148	Clay	Chalk, sands, and gravels	1957–61	55
Kislingbury	106	Sands, gravels, and clays	Clay	1948–51	56
Halse Water	88	Sandstones and marls	Sandstones and marl	1961–65	68
Ter	75	Clay	Sands and gravels	1938–46	39
Harpers Brook	70	Clay	Limestones and clays	1948–51	55
Mill River	23	Sands and gravels	Clay	1957–61	90
Nidd (Howstean)	16	Clay	Sandstones and shales	1961–62	18
Burbage Brook	16	Clay, peat	Sandstones, shales, and limestones	1940–45	39

*Area of groundwater catchment exceeds that of surface topographic catchment

The location of a spring is dependent upon a number of factors—climate and geology being two of the most important. Intermittent springs are very much influenced by climate. After a heavy rainfall the water table may rise to intersect the ground surface and so produce a spring, the bournes of the Chalk uplands providing examples. In dry weather the water table sinks and the stream disappears.

The commonest geological setting for a spring is at the contact between two beds of differing permeability. This setting often gives rise to a spring line and these springs are referred to as stratum springs. There are two types of stratum springs (*Figure 4.9(a)*). If the downward percolation of water in a permeable horizon is impeded by an underlying impermeable layer, then the spring which issues is termed a contact spring. Conversely an overflow spring may be formed where a permeable bed dips beneath an impermeable one. Unconformities may give rise to springs along their outcrop. When a

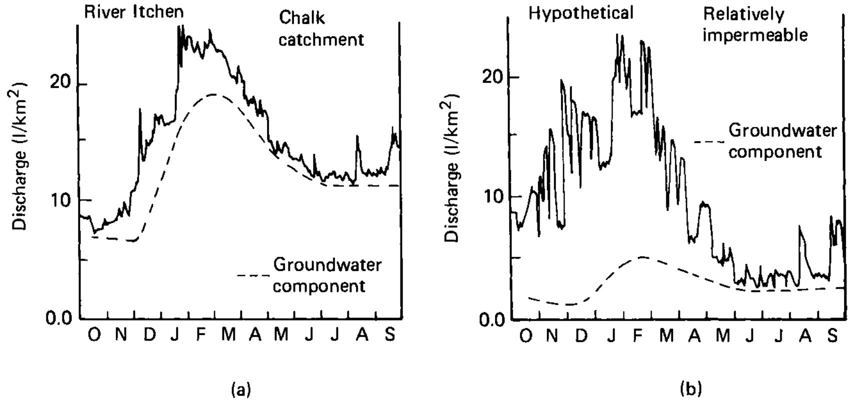

Figure 4.8 Effect of groundwater discharge on hydrograph configuration. (a) Hydrograph of the river Itchen in Hampshire 1959–60. Note the large contribution to total flow made by groundwater discharge, particularly during the summer. This is typical of a permeable catchment such as the Chalk. Surface run-off is of secondary importance (after Smith[12]). (b) Hypothetical hydrograph resulting from the same rainfall as (a), but falling on a relatively impermeable catchment. The groundwater component of river discharge is significantly reduced, particularly during the summer so surface run-off is responsible for a much greater proportion of the total flow and peak flows

permeable formation is thrown against an impermeable formation by faulting, water may issue at the surface along the fault plane (*Figure 4.9*(*b*)). Water table or valley springs emerge where a valley is carved beneath the water table in thick permeable formations (*Figure 4.9*(*c*)). The discharge from such springs is usually small. The intermittent springs mentioned above are a type of water table spring.

Tolman[23] distinguished solution springs. These are found in areas of massive limestone where underground streams emerge at the surface from solution channels or galleries. Springs may also issue from tunnels in lava flows.

Artesian springs, which draw on water from a confined aquifer under piezometric pressure, are not common. They occur where the overlying rock is broken in some way. The 'blow wells' which occur in Lincolnshire probably represent examples of artesian springs. Water, under pressure in the Chalk, escapes through the overlying till where it forms a thin cover and is fissured.

The most common type of thermal spring is the hot spring; however, the temperatures of thermal springs may range from lukewarm to near boiling point. Hot springs are found in all volcanic districts, even some of those where the volcanoes are extinct. They presumably originate from steam given off by a magmatic source. On its passage to the surface the steam commonly encounters ground water, which is consequently heated and contributes to the hot springs. A geyser is a hot spring which periodically discharges a column of hot water. In several places in the world hot springs are utilized for the generation of electric power, for example, at Larderello in Italy, at Wairakei in New Zealand and in Somona County, California. In Iceland hot springs are used for domestic heating. However, hot springs generally contain dissolved gases and minerals such as carbon dioxide, hydrogen sulphide, calcium carbonate and silica. These may cause serious problems of corrosion and precipitation in plant which harnesses such springs. On the other hand, the solution of mineral matter by warm springs did lead to the development of spa towns, the water supposedly having some therapeutic value.

Springs, in their various forms, can have a significant influence upon the character of

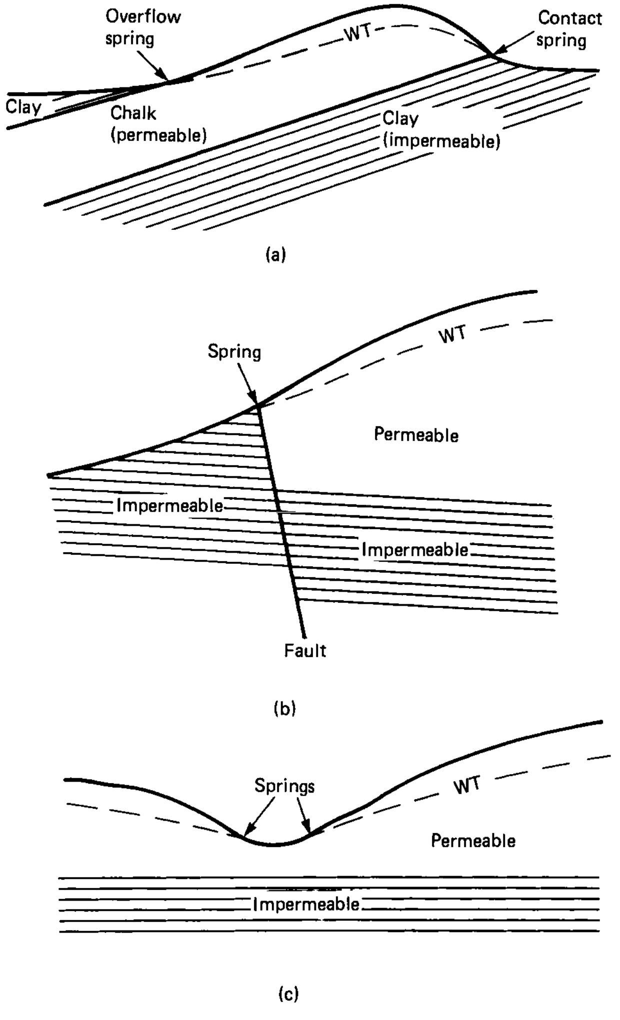

Figure 4.9 Types of spring. (a) Stratum springs. (b) Fault spring. (c) Valley spring. (WT = water table)

a particular region. In some parts of England spring line settlement is evident, where villages and towns are located at intervals along a hillside, having grown around the water supply provided by the springs.

From the hydrogeological point of view, chalk and limestone areas can be most interesting, though complex. In these regions it is not uncommon for entire rivers to vanish underground through 'swallow holes' or 'sink holes'. Since the surface and subterranean drainage systems usually have different orientations, especially in limestone, a stream that enters a sink hole will probably reappear in a different valley. For instance, the water that flows from Malham Tarn in Yorkshire enters a sink hole (*Figure 4.10*) and emerges at Kirkby Malham some 11.2 km away, and not at Malham Cove as would be expected from the course of the dry river valley.

The quantification of groundwater discharge via springs and river baseflow is an important step in some methods of estimating groundwater recharge and aquifer yield (see Section 4.5.3). With complex aquifers this is no easy task.

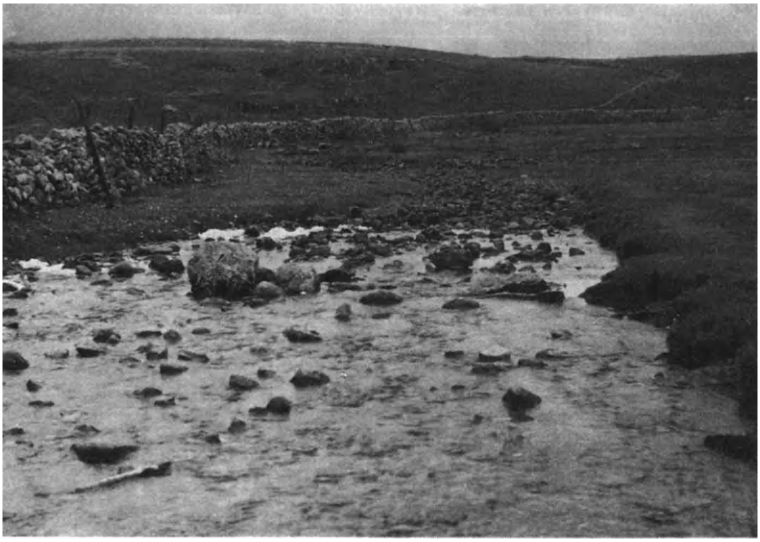

Figure 4.10 Sinkholes near Malham, Yorkshire. The stream flowing south from Malham Tarn disappears underground via the sinkholes

4.5 Assessment of aquifer recharge and potential well yield

4.5.1 What is meant by 'yield'?

The 'yield' of a groundwater resource is difficult to determine with any degree of accuracy and as a result, the term may be applied simply to represent the pumpage from a well without having any clear or specified meaning. However, the meaning of 'yield', as applied to a surface water resource, is well defined, and there are many different definitions that may be applied in various parts of the world. The two definitions given below are generally used in connection with surface sources, but are equally applicable to groundwater reservoirs.

In the United Kingdom the term 'yield' is often associated with a return interval, so that it can be defined as the steady supply that could be maintained through a drought of specified severity. For example, the yield that could be maintained through a 1 in 50 year (2 per cent) drought would be greater than that which could be maintained through a 1 in 100 year (1 per cent) drought. Thus, yield is not an absolute quantity, but a variable that depends upon the specified frequency of occurrence of the limiting drought conditions. Since droughts occur irregularly, their frequency of occurrence and their severity cannot always be predicted with any confidence, so it follows that some uncertainty must also be attached to estimates of well yield.

Yield may also be defined as the steady supply that could be maintained through the worst drought on record. This may mean that the yield is the steady supply that could be maintained through the period of lowest groundwater levels on record. In this case the severity of the limiting drought conditions depends upon the rainfall (and thus groundwater level) recorded in the years preceding the drought period, the length of the record available for analysis, and chance—a short record may contain a particularly severe drought, while a much longer record elsewhere may not. For this reason the first definition is to be preferred. Several other definitions are given by Twort *et al.*[24]

The 'safe yield' of an aquifer was defined by Meinzer[25] as 'the practicable rate of withdrawing water from it perennially for human use'. As a result, the safe yield of an aquifer is often thought of as the yield that can be maintained over a long period of time (including droughts) without causing an unacceptable reduction in groundwater level, an unacceptable pumping lift, or initiating a decline in water quality. Unfortunately, the concept of a 'safe' yield is not practical, since any yield is safe until it is proved otherwise. Thus, it is not unusual for the safe yield of an aquifer to be over-estimated initially (possibly as a result of erroneous conclusions based upon short period observations) and subsequently reduced as operational experience is accumulated.

Kazmann[26] likened the concept of safe yield to the former speed limit in the State of Tennessee, where no limit was specified in law only a request to 'please drive carefully'. However, the speed limit was considered to have been broken if an accident occurred, since someone had obviously not been driving slowly enough. Exceeding the safe yield must, almost by definition, produce some sort of 'dangerous' or undesirable condition in the aquifer and care should be taken to ensure that this does not happen. So as not to give a false sense of security, the term 'safe yield' is best avoided, 'perennial yield' being more preferable if such a term has to be used. Regardless of what it may be called, the basic problem remains that of determining the proportion of the natural flow through an aquifer that can be abstracted over a considerable period of time without incurring any undesirable consequence. Before this problem can be solved, some knowledge of the aquifer recharge–discharge process is required.

An aquifer can be thought of as a pipe or conduit that transfers water from the recharge area to areas of discharge, and which also has a storage component. A simple equation representing flow through the aquifer is

$$\text{Recharge to the aquifer} - \text{Discharge from the aquifer} = \text{Change in groundwater storage} \tag{4.7}$$

Thus, if outflow exceeds inflow there is a reduction in the amount of water stored in the aquifer and the water level falls. Obviously this can only continue until such time as the storage becomes exhausted, when the maximum yield available at any particular time will be equal to the inflow.

The seasonal variation in the amount of water available for aquifer recharge was discussed in Section 4.3 and it was shown that water levels follow an annual cycle. Such fluctuations are due to a temporary inequality between discharge and recharge and therefore represent fairly minor changes in the quantity of water stored in the aquifer. Over a prolonged period of time ($>$ 10 years) the rates of recharge and discharge will be very nearly equal, so that under natural conditions the aquifer is in a state of approximate dynamic equilibrium. Discharge by wells represents a new condition superimposed upon a previously stable system and must be balanced by an increase in the recharge of the aquifer, a decrease in the old natural discharge, a loss of storage, or by a combination of these[27]. Since a reduction in storage yields only a limited amount of water and cannot continue indefinitely, it follows that if abstraction is to continue for a period of many years, the new balance must be created as a result of either increased recharge or decrease discharge. The maximum yield that can be obtained over a prolonged period is equal to the average annual recharge which in turn equals the long-term flow through the aquifer, provided that natural discharge is stopped and no water passes the well (*Figure 4.11*). The last proviso is very important, for it is difficult or impossible to prevent some natural discharge from an aquifer in the form of river baseflow (*Figures 4.7* and *4.8*), springs or seepage. To prevent the natural flow of water

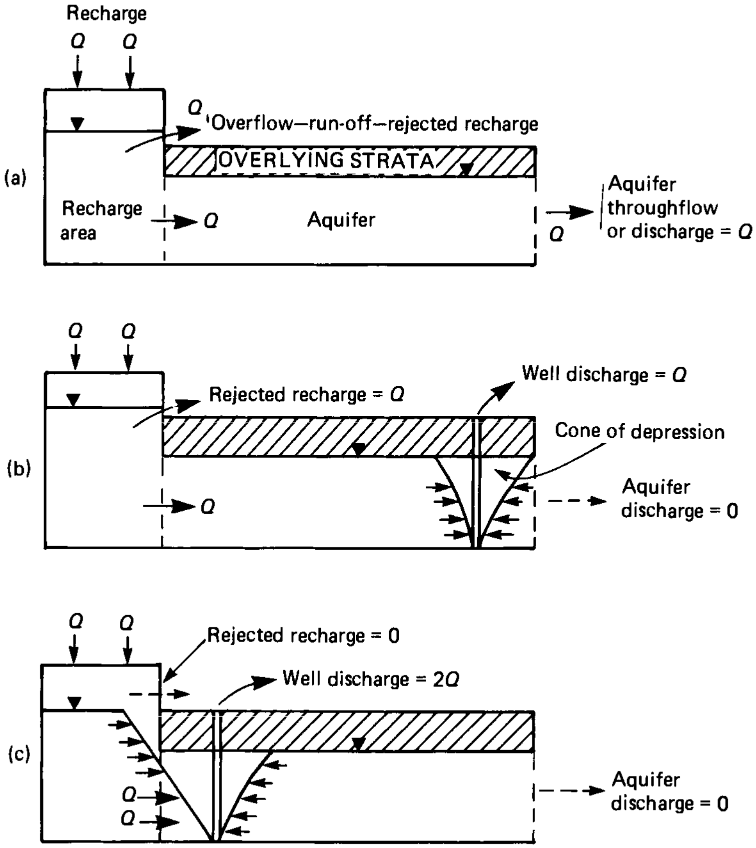

Figure 4.11 Schematic diagram of possible aquifer recharge–discharge mechanisms. (a) Aquifer-recharge area under natural conditions. Potential recharge rate is $2Q$, although the infiltration capacity of the recharge area is limited to Q so the remaining Q becomes rejected recharge such as run-off. The Q that is not rejected flows through the aquifer to the discharge area. (b) As (a), but a well located near to the discharge area intercepts the aquifer discharge which is reduced to zero. (c) The well is now located so that the cone of depression reaches the area of rejected recharge. As a result of the steeper hydraulic gradient the flow through the aquifer will be increased and rejected recharge will be reduced (in this example to zero). By this means the yield of a well or aquifer may be increased, although streamflow may be decreased so the net gain to a conjunctive use scheme could be negligible

from the aquifer it would be necessary to lower the water levels everywhere between the wells and the areas of natural discharge so that, by Darcy's law, flow is towards the wells. In practice it may be foolish to even attempt this, since the result could be contamination of an aquifer or some other unforeseen consequence (see Chapter 8).

Instead of trying to stop natural discharge, it is possible (although not certain) that yield could be increased by locating a well where its cone of depression can reach the recharge area. Whether or not the yield is increased depends upon which of the two following conditions prevail

1. The recharge area is extremely permeable and/or receives only very small amounts of precipitation, so that almost all of the water that falls on the recharge area eventually enters the aquifer. No potential recharge is rejected.

2. The recharge area receives large amounts of precipitation and/or its infiltration capacity is limited. Some potential recharge is rejected and lost (from the point of view of groundwater flow) as run-off or evapotranspiration.

In the first case aquifer recharge cannot be increased, since there is no surplus water available. In the latter case, however, recharge may be augmented if the well is located so as to increase the hydraulic gradient between itself and the recharge area (*Figure 4.11(c)*). By Darcy's law the flow in that part of the aquifer would be improved and a greater volume of recharge may result. Surplus water is available, so the volume of rejected recharge could be decreased in favour of increased aquifer flow. It should be noted, however, that if a conjunctive use scheme is to be adopted (see Chapter 9), where surface and groundwater resources are considered as one, this would not necessarily result in an increase in yield[28,29].

Theis[27] claimed that for most confined aquifers of small areal extent the perennial yield is equal to the amount of rejected recharge and natural discharge that can be diverted to the wells. Provided that this amount is not exceeded, the water level will eventually stabilize as equilibrium is achieved. If it is exceeded, water levels will continue to fall. If a well is situated in an unconfined aquifer and is remote from areas of rejected recharge or natural discharge, equilibrium may not be established in the foreseeable future so that any water that is abstracted is derived from storage. Under these conditions the concept of a perennial yield does not apply.

Even if the arguments put forward by Theis are accepted as giving an indication of perennial yield, many complications arise when attempting to make a quantitative assessment of aquifer yield. In practice it may prove impossible to estimate the reduction in natural discharge or the amount of rejected recharge that can be utilized. Consequently the assessment of aquifer yield tends to be a rather imprecise and crude process. However, even a crude estimate of potential yield is better than none at all, since this may at least indicate whether the yield is likely to be measured in tens, hundreds, or thousands of cubic metres per day. This information alone may be valuable during the preliminary stages of a groundwater resource investigation.

There are a variety of methods that may be used to assess aquifer yield. To reiterate a statement made in the introduction, these methods should be used in conjunction with each other and the *average* value of long-term yield calculated. They should not be used in isolation since all these techniques are subjective, and because of the indeterminate nature of some of the variables, liable to give answers which contain a significant error. This is particularly true if the data and maps described in Chapter 3, which form the basis of many of the methods described below, are not available.

The methods which may be used to assess potential aquifer or well yield can be divided into two broad groups. The meteorological–hydrological methods can be termed 'above ground' methods in that they use data or observations of phenomena which occur at or above the surface of the Earth. The essence of these methods is that they estimate the amount of water that is available for groundwater recharge in a given area, or the amount that has been discharged from an aquifer as springs or river baseflow. This does not, of course, necessarily imply that all this amount can be abstracted. The second group of methods are of a hydrogeological nature and can be thought of as 'below ground' or subsurface techniques. These aim to assess the yield of an aquifer from a consideration of fluctuations in the level of the water table or by the analysis of groundwater flow patterns.

When faced with the need to assess the yield of an aquifer, it is best to start with the methods that use the data which are most likely to be available during a preliminary desk study. Unless the aquifer has already been the subject of a detailed hydrogeological investigation, this generally means that the meteorological–hydrological methods will be employed first. In Britain at least, these type of data are usually available and have been collected over a period of many years. Even if they are not initially available, they are relatively cheap to collect.

Once the above ground methods have been exhausted, the hydrogeological group of methods may be employed. Hydrogeological data are usually more difficult and more expensive to acquire than that required for the above ground techniques, because observation boreholes may have to be constructed for the purpose. For this reason hydrogeological methods are more likely to be employed in the latter stages of groundwater resource investigation, after a preliminary desk study has indicated that the expense incurred will probably be justified.

The individual techniques that can be used to estimate the yield of an aquifer or a well are described in the following sections. Their data requirements and their advantages and disadvantages are compared in *Table 4.7*.

4.5.2 Water budget studies

Water budget studies attempt to quantify the average annual flow through the aquifer by considering the disposition of the rain that falls on the recharge area. Basically the rain must either run off the surface and become streamflow, be evapotranspired, or infiltrate the surface and thereby add to groundwater storage (see Section 4.3). The potential infiltration rate and subsequently the change in groundwater storage, can be estimated by rewriting Equation (4.4) so that

$$dS_g = (\mathrm{P} - \mathrm{AET} - \mathrm{R}_0) \tag{4.8}$$

where dS_g is the change in groundwater storage (m/year), P is the precipitation (m/year), AET is the actual evapotranspiration (m/year) and R_0 is the total run-off, including groundwater discharge and interflow (m^3/m^2 of catchment/year).

If the area of the recharge zone of the aquifer is known, the annual volume of recharge to the aquifer is given by

$$Q = (P - AET - R_0)A \tag{4.9}$$

where Q is the annual volume of groundwater recharge (m^3/year) and A is the area of the recharge zone (m^2).

If potential evapotranspiration (PET) is used in Equation (4.9) instead of AET, then a more conservative estimate of groundwater recharge is obtained since PET always exceeds AET. Alternatively, if the infiltration rate over the recharge area has been determined using a percolation gauge or by analysing groundwater discharge (Section 4.5.3) then

$$Q = f_e A \tag{4.10}$$

where f_e is the effective infiltration rate (m/year).

The reliability of water budget methods depends to some extent upon how accurately the values of the variables in the equation are known or can be estimated. For good results quite intensive instrumentation of the recharge area is necessary. Further errors will result from the fact that several terms of Equation (4.2) have been ignored, although this may be acceptable if the balance period has been carefully selected (Section 4.2). However, possibly the greatest limitation of this technique is the assumption that water infiltrating the surface of the recharge area (Equation (4.10)) will eventually reach any well-located down gradient. Some potential groundwater

TABLE 4.7. Comparison of methods of assessing aquifer and well yield

(A) *Meteorological/hydrological techniques*

Method and data requirement	*Comments*
1. *Water budget studies* Aquifer recharge area. Either precipitation, actual evapotranspiration and run-off, or known infiltration rate over recharge area. (See Section 4.5.2)	Uses data that are generally available or can be easily obtained or estimated and so is a quick, cheap, easy method to implement. Useful for desk and feasibility studies, but has a large data demand and may be of dubious accuracy. Estimates potential groundwater recharge not aquifer/well yield.
2. *River hydrograph analysis* River flow records for a number of years. (See Section 4.5.3)	The assumption that the baseflow component of river discharge is surplus groundwater that could be diverted to wells is not always valid. Uses records that are generally available or easy to obtain. Can be used to estimate groundwater discharge for use in other techniques.

(B) *Hydrogeological techniques*

Method and data requirement	*Comments*
3. *Groundwater level fluctuations* Area of aquifer, average seasonal rise in water level, average specific yield of aquifer material. (See Section 4.5.4)	The simplest and cheapest of the hydrogeological methods provided that boreholes do not have to be constructed to obtain the data. Uses actual changes in groundwater storage, so has a better basis than the techniques above. However, gives groundwater recharge not aquifer/well yield. Average values of the variables may be difficult to estimate.
4. *Darcy's Law and flow net analysis* Water levels in the aquifer. Transmissivity at one point in the aquifer, at least. (See Section 4.5.5)	Relatively quick, cheap method, although flow nets are of doubtful accuracy if the aquifer is complex. Assumes that adequate water level data are available. Does not indicate well yield, only total flow in the aquifer. The results may be of dubious accuracy.
5. *Pumping tests and flow net analysis* The fall in water level over a period of time around a well discharging at a known constant rate. (See Section 4.5.6 and Chapter 7)	Construction of a well and the conduct of a suitable test is very expensive. However, widely used in the final 'proving' stage of groundwater investigations. The only method that gives reliable data regarding well/aquifer yields and the effects of pumping. Limitations of flow nets as in (4).
6. *Models* Water levels in the aquifer, transmissivity, storativity, boundary conditions, knowledge of flow mechanisms, pumping test data. Possibly quantitative knowledge of recharge, discharge, and surface-groundwater interconnection. Plus suitable hardware and software to solve flow equations. (See Section 4.5.7 and Chapter 10)	Requires vast quantities of data to produce a reliable model, which may be available only after a groundwater investigation has been completed. Models make useful management tools and can be used to extend or extrapolate existing data and to make predictions. However, models can be expensive to construct, but cost may be offset against potential savings through improved management or reduced field investigations.

recharge may become interflow and be discharged to a surface watercourse without even reaching the water table. Added to this is the problem that some proportion of the water that has succeeded in percolating down to the zone of saturation may be lost as groundwater discharge to rivers and springs. The former may be included in the run-off term, R_0, in Equation (4.9), while the latter may require a survey of all the springs in the area (see Section 4.4.2). Of course, the perennial yield to wells will only be a fraction of the total recharge volume calculated from Equations (4.9) and (4.10), since it is unlikely that all the natural discharge from the aquifer could be stopped or that the wells could intercept the total flow through the aquifer. Conversely, it is also possible that the recharge volume could be greater than that indicated by Equation (4.10), since recharge may also take place as a result of seepage from rivers or other bodies of surface water. In such circumstances the use of river gauging or tracer techniques may be necessary to identify and quantify the leakage[1,30]. Several analytical techniques are also available for the evaluation of induced infiltration, where some proportion of well discharge is obtained from a surface source (see Chapter 8).

As a rough guide to the rate of infiltration that may be experienced in Britain, Skeat[7] suggested a 'rule of thumb' figure of 250 mm per year, while Boswell[31] proposed a figure of 40 per cent of the rainfall if no other data were available. These figures are supported by Aldrick[32] who calculated the recharge to the Magnesian Limestone in Yorkshire using a value of 250 mm for average annual infiltration, which was 38 per cent of the average rainfall of 660 mm/year. However, other investigators have used values of infiltration ranging from 150 mm/year to 300 mm/year[3,29,33]. As a general rule the figure of 40 per cent of average annual rainfall is to be preferred to a fixed value of 250 mm/year, since the percentage value does take into consideration the variation of rainfall (perhaps 500 mm to >2500 mm) across Britain.

4.5.3 Groundwater recharge from river hydrographs

The water budget technique as typified by Equation (4.9) attempts to quantify groundwater recharge by considering the balance between precipitation, actual evapotranspiration and surface run-off in a given area. However, in regions which have very permeable surface strata, such as limestone and chalk areas, there may be little or no surface run-off. Consequently any increase in groundwater storage is equal to the difference between precipitation (P) and actual evapotranspiration (AET) over the recharge area (A), so that the annual volume of recharge (Q) is given by

$$Q = (P - AET)A \text{ m}^3\text{/year} \tag{4.11}$$

Unfortunately, some water is likely to be lost as interflow or groundwater discharge to neighbouring catchments (Section 4.4.2), and this adds some element of uncertainty to the use of Equation (4.11).

If a region has very permeable surface deposits, but also has some sizeable streams and rivers, then it is quite possible (or even probable) that these watercourses are maintained by groundwater discharge. In an area where a very high proportion of the available groundwater resource discharges naturally to surface streams, it may be possible to quantify the surplus capacity of an aquifer from an analysis of river discharge hydrographs. The assumption inherent in this method is that the aquifer is overflowing into the surface watercourses and that this water could be diverted to wells instead. Thus, the amount of water that is available, assuming natural overflow is stopped, is equal to the total groundwater component of river discharge. This can be evaluated by separating the surface run-off, interflow and groundwater components of total river discharge using one of several available techniques[10,17,18]. These techniques can be applied to individual hydrographs (as in *Figure 4.2*), although in this instance it is much more enlightening to analyse a record of several months' duration. Since a groundwater recession curve is almost exponential in form (see Equation 4.6)) it will plot as a straight line on semi-logarithmic paper. Consequently a line representing the upper limit of groundwater discharge may be drawn through the lowest points on a river hydrograph (*Figure 4.12*) if it is assumed that the flow at these times was due entirely to groundwater discharge and that there was little or no surface run-off. The area of the hydrograph under the line so constructed is equivalent to the total groundwater discharge to the river over the period of the record, and thus may be calculated. If all the river records from the area under investigation are treated to a similar analysis, the total outflow from the aquifer may be estimated.

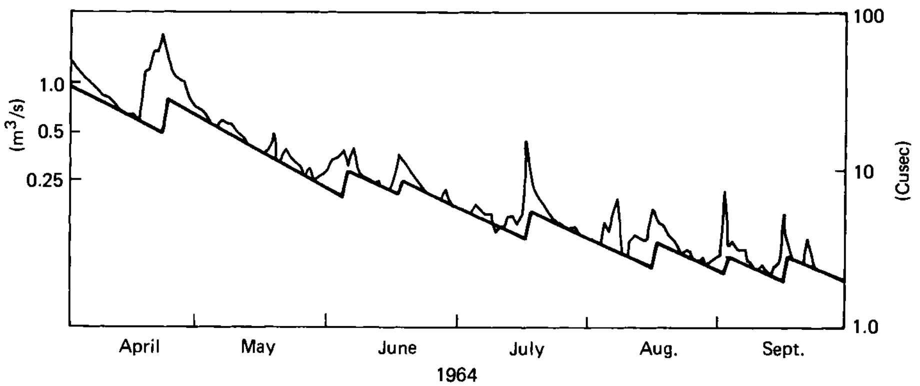

Figure 4.12 Semi-logarithmic plot of the mean daily flow of the Clow Beck, Yorkshire, during the summer half-year 1964. The lower line represents the upper limit of groundwater discharge (after Smith[12])

Two additional techniques which can be used to study the discharge of groundwater from a catchment are provided by Meyboom[20] and Kunkle[21]. Meyboom used the baseflow recession to estimate groundwater recharge and 'total potential groundwater discharge, Q_{tp},' which is defined as

$$Q_{tp} = K_1 K_2 / 2.3 \qquad (4.12)$$

where K_1 is groundwater discharge at the beginning of the baseflow recession and K_2 is the time increment corresponding to one log cycle change in the river baseflow.

Kunkle[21] described a method of estimating groundwater discharge from a catchment using a baseflow duration curve, which indicates the percentage of time that specified baseflows are equalled or exceeded during a given time period. The basis of this technique is similar to that previously described.

The evaluation of the groundwater contribution to riverflow may be necessary for several reasons. In some investigations it may be desirable to quantify all groundwater discharge so as to obtain a better estimate of the 'effective infiltration' over the recharge area (f_e in Equation (4.10)). In others, the objective may be to gain a better understanding of aquifer flow mechanisms, or to allow a more accurate solution of the water budget equation.

Frequently, the natural discharge of groundwater is considered as overflow from an aquifer or 'waste' water that can be diverted to wells and abstracted. In this case the consequences of such an action must be considered, particularly the possible adverse effect on surface water quantity and quality, the ecology of the area, and the abstraction of water from existing shallow wells. For this reason alone the quantification of groundwater discharge may be worthwhile.

4.5.4 Groundwater level fluctuations

Groundwater levels follow an annual cycle as a result of seasonal variations in the quantity of effective or excess rain in the recharge area (see Section 4.3 and *Figure 4.6*). These increases in water level can, therefore, be used to form an estimate of the annual volume of recharge to an unconfined aquifer. If the entire aquifer is to be considered, the first step is to draw a water level change map (see Chapter 3 and Davis and DeWiest[34]) that shows the area over which the increase in level occurs. The second step is to calculate the weighted average increase in water level over the area using the contours of the map. The product of the area and the average increase in level gives the change in the saturated volume of the aquifer. If the specific yield of the aquifer material (which is equal to the coefficient of storage for unconfined aquifers) is known, then the annual volume of recharge, Q, is given by

$$Q = AhS_y \tag{4.13}$$

where A is the plan area over which a rise in water level is observed, h is the weighted average increase in water level and S_y is the specific yield of the aquifer material.

If the annual recharge to a group of wells is of interest, as opposed to the whole aquifer, this can be evaluated using Equation (4.13), after first determining the area of diversion to the well group[35,36]. Using the groundwater contours recorded while the well group is discharging, the area of diversion can be drawn in much the same way as the watershed or catchment of a surface reservoir or river is delineated. In fact, the line marking the boundary of the area of diversion is the watershed of the underground catchment area of the wells (see Section 4.5.6).

The appeal of this technique is that it provides a very simple, quick estimate of annual recharge volumes. Unfortunately it may be rather inaccurate, particularly if the aquifer is non-uniform, as is usually the case, so that the average values of the variables in Equation (4.13) are difficult to determine. The method is not applicable to confined aquifers, in which the apparent annual increase in water level is due to a pressure effect; the actual saturated volume of the aquifer does not change. Under these conditions the specific yield of the aquifer material is not equal to its coefficient of storage.

4.5.5 Darcy's law and flow net analysis applied to two-dimensional flow

Evaluation of the flow in the aquifer commences with the construction of a flow net. A flow net consists of two sets of lines; flow lines which represent the paths of particles of water as they move through the aquifer, and equipotential lines, which are lines of equal drop in head that are equivalent to the water or piezometric surface contours. If the two sets of lines are arranged so as to form an orthogonal pattern of small squares, the result is a flow net (*Figure 4.13*). The section of the aquifer between adjacent flow lines is called a flow tube or a stream tube. Because the aquifer is assumed to be uniform, it follows that if the flow lines are evenly spaced the flow in each of the flow tubes so formed will be equal. Since no flow can cross a flow line (because by definition the velocity of a particle

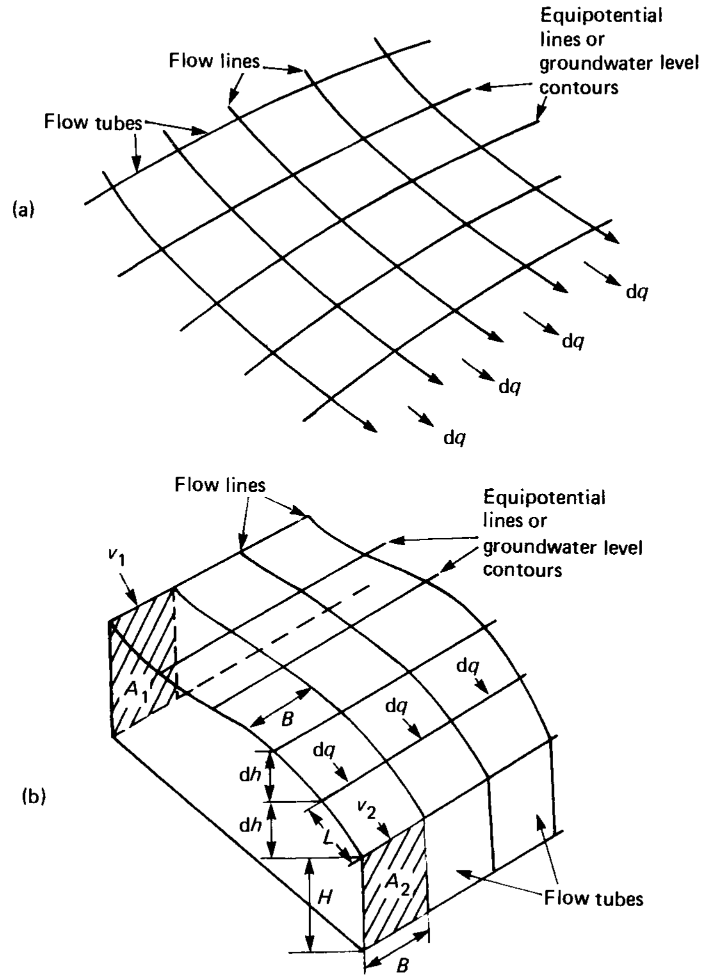

Figure 4.13 Flow net. (a) Two-dimensional approximation. (b) Three-dimensional section of aquifer showing flow tubes

at any point on a flow line is tangential to it) the flow, dq, at two sections in any particular flow tube (*Figure 4.13(b)*) is given by

$$dq = v_1 A_1 = v_2 A_2 \tag{4.14}$$

where v_1 and v_2 are the fluid velocities passing through sections of cross-sectional area A_1 and A_2. On the basis of Darcy's law of groundwater movement, Equation (4.14) can be written as

$$k_1 i_1 A_1 = k_2 i_2 A_2 \tag{4.15}$$

where k_1 and k_2 are the coefficients of permeability at the two sections and i_1 and i_2 the respective hydraulic gradients. Thus

$$\frac{k_1}{k_2} = \frac{i_2 A_2}{i_1 A_1} \tag{4.16}$$

Assuming that the aquifer is of near uniform thickness and that the flow lines are almost parallel, then $A_1 \doteqdot A_2$ and $i = \mathrm{d}h/L$ where $\mathrm{d}h$ is the interval between successive equipotential lines and L is the plan distance between them, so

$$\frac{k_1}{k_2} = \frac{\mathrm{d}h/L_2}{\mathrm{d}h/L_1} = \frac{L_1}{L_2} \tag{4.17}$$

In other words, the coefficient of permeability (or transmissivity) is proportional to the distance between the groundwater level contours. This fact can be used in several ways. For instance, if the permeability of the aquifer material is known at only one point, the permeability at any other location may be calculated from Equation (4.17). Alternatively, if the permeability is known from the results of laboratory or pumping tests or, as a first approximation from tables, it is possible to calculate the flow in a flow tube, $\mathrm{d}q$, from

$$\mathrm{d}q = kHB(\mathrm{d}h/L) \tag{4.18}$$

where H is the saturated thickness of the aquifer and B is the width between adjacent flow lines, that is, the width of the aquifer through which groundwater flow takes place. If the value of transmissivity is available as a result of pumping tests, this may be substituted for the product of kH in Equation (4.18). The total flow in the aquifer is the sum of the flows in the individual flow tubes. The number of flow tubes can, of course, be determined from the flow net.

Equation (4.18) also can be used to solve what is sometimes referred to as the inverse problem. Once the flow through each stream tube has been specified the transmissivity ($=kH$) may be calculated at the central point of each flow net element. These values of transmissivity may then be used in a finite difference model (see Chapter 10) to compute the variation in water level over the study area (*Figure 4.14*).

If a flow net is drawn so that the total drop in head, h_T, is divided into N_d potential drops, and there are N_f flow tubes, it can be shown that the total flow, Q, through a homogeneous isotropic aquifer is

$$Q = \frac{N_\mathrm{f}}{N_\mathrm{d}} kHh_\mathrm{T} \tag{4.19}$$

A proof of this formula can be found in most books on hydrogeology or soil mechanics[37,38,39].

In almost all localities an aquifer is not homogeneous or isotropic or of uniform thickness. Under these circumstances it is still possible to draw a flow net, although it may be difficult to draw a definitive set of flow lines and the 'squares' of the mesh will probably be rectangular (*Figure 4.14(b)*). However, the flow in a flow tube can be calculated, albeit with reduced accuracy, using Equation (4.18) provided that the permeability is known at some point. The total flow in the aquifer can then be estimated, because if the flow lines are evenly spaced initially, the flow in each flow tube should be the same. If the flow lines are not evenly spaced to begin with, which may be the case if the recharge or permeability is not uniform along the highest equipotential line, it may be necessary to calculate the flow in each individual flow tube using a combination of Equations (4.17) and (4.18) and one known value of permeability.

This method of aquifer evaluation incorporates many simplifying assumptions which may or may not be valid in a particular locality. It also relies upon a flow net, which is subjective and difficult to construct when the pattern of flow is complex. Consequently a wide range of answers may result from the analysis of the same

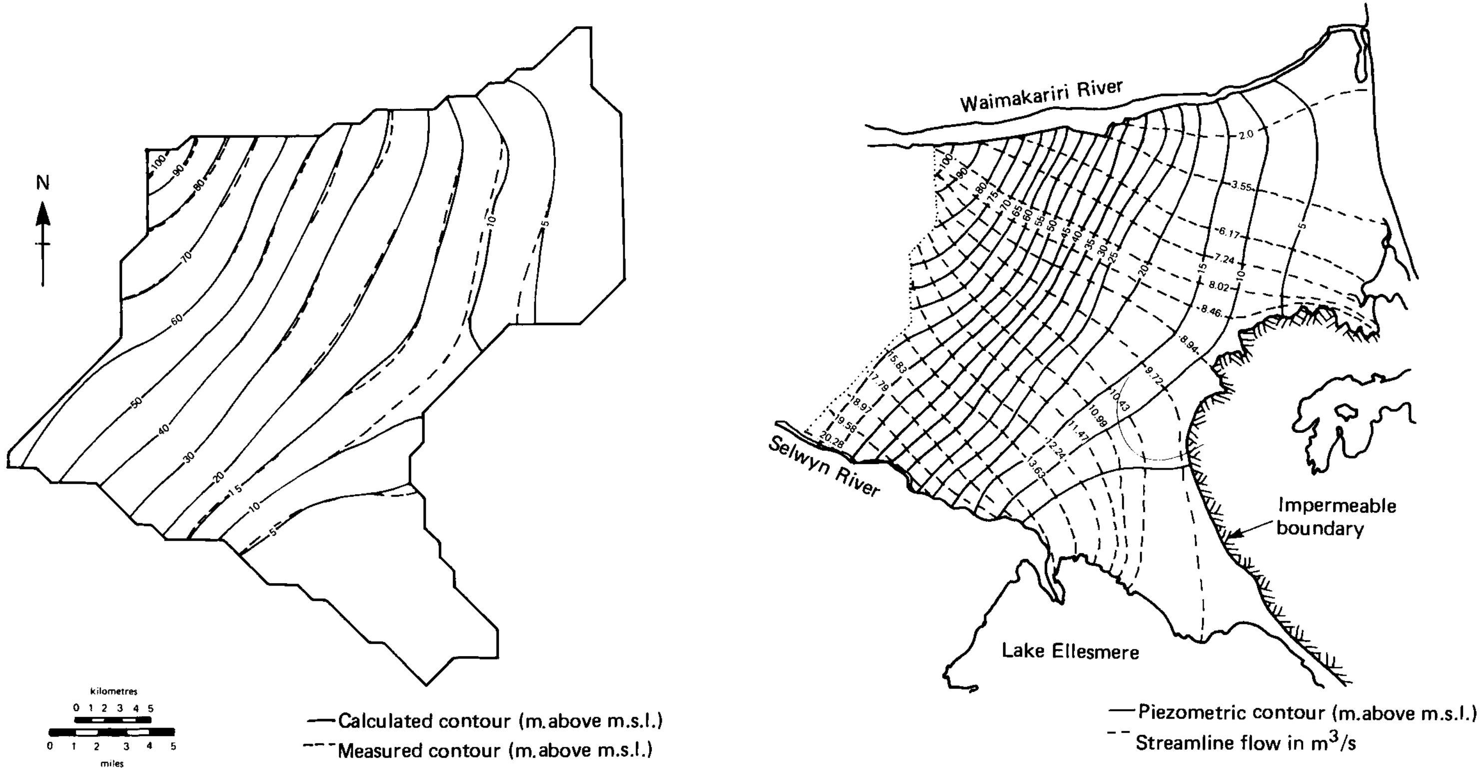

Figure 4.14 A comparison between measured piezometric contours and contours computed from a transmissivity distribution calculated by Day and Hunt[40] with Equation (4.18). The flow net that was used in this calculation is shown in *Figure 4.14(b)*

problem by different people. On the other hand, the advantage of the method is that it is relatively quick and inexpensive and that it can be conducted as part of a desk study, or perhaps as part of a feasibility study into which of several aquifers should be selected for more detailed investigation. Some interesting applications of the technique are provided by Walton[35,39] and Prickett *et al.*.[36]

So far it has been assumed that recharge occurs over an outcrop of the aquifer. However, it is possible that recharge to a leaky or semi-confined aquifer may be by way of vertical seepage through an aquitard, in which case the rate of flow, Q_v, may be calculated from the following modified form of the Darcy equation

$$Q_v = \frac{k_v \Delta h A}{H} \tag{4.20}$$

where k_v is the coefficient of vertical permeability of the aquitard, Δh is the difference in head between the groundwater level in the aquifer and the water level of the surface resource from which the seepage occurs, A is the plan area of the aquitard through which seepage occurs and H is the thickness of the aquitard through which seepage occurs. For further details see Walton[39].

4.5.6 Pumping tests and flow net analysis

When water is pumped from a well in an unconfined aquifer, there is a fall in the level of the water table in the area surrounding the well. The greatest decline in water level occurs at the well itself, while progressively smaller reductions are recorded as the distance from the well increases. The space between the standing and pumped water surfaces takes the form of an inverted cone and for this reason the volume of the aquifer that has been desaturated as a result of pumpage is called the 'cone of depression'.

The size and shape of the cone depends upon four factors: the transmissivity and the storativity of the aquifer, the pumping rate and the duration of pumping. The steady discharge from the well is equal to the amount of aquifer through-flow intercepted by the cone, so the cone either expands until it intercepts sufficient of this flow to satisfy the discharge, or until the rate of its expansion is so small as to go unnoticed. In either case the cone appears to have a constant size and shape. However, if the rate of abstraction is large with respect to the rate of flow of groundwater through the aquifer, the cone of depression will not be able to intercept enough of the flow to balance the discharge from the well. This means more and more water must be taken from storage and, consequently, the cone will continue to expand and deepen so that water levels in the vicinity of the well decline progressively until the aquifer is exhausted, the well is dry, or the pumping lift becomes uneconomic. If this occurs it can be said with some confidence that the abstraction rate was too high and that the perennial yield of the well or aquifer has been exceeded.

Thus, some information regarding the potential yield of a well can be obtained by gradually increasing its rate of discharge, say, by multiples of the initial pumping rate. If the drawdown at various observation points becomes stable, it could be tentatively concluded that the perennial yield has not been exceeded and that the abstraction rate could be increased. Conversely, if the water levels continue to decline, the perennial yield has been exceeded and the pumping rate should be reduced until such time as the cone of depression stabilizes (see Chapter 7).

A confined aquifer will behave in a similar manner to the unconfined aquifer described above, although in this case the aquifer will not be de-saturated because the cone of depression represents a decline in the piezometric or pressure surface, not the water surface itself. The saturated thickness of a confined aquifer remains constant, provided that the cone of depression does not extend beneath the level of the confining stratum. Usually wells are designed and operated with the objective of making sure that this does not happen (see Section 6.3.1). In general, the storativity of a confined aquifer is significantly less than that of an unconfined aquifer, typical values being in the range 0.00001–0.001 and 0.02–0.30, respectively. Consequently the cone of depression in a confined aquifer may extend a considerable distance before achieving equilibrium.

The term 'de-saturated', rather than 'de-watered', has been used above to describe the volume of an aquifer within the cone of depression. This is because some water will be retained in the material against the force of gravity so that it is not completely de-watered.

Although a pumping test may be used to investigate the perennial yield of a well, it will not necessarily give a direct indication of the perennial yield of the aquifer. This is especially true if the aquifer is large and much of it remains unaffected by the test. In this case recourse must be made to some form of flow net analysis to determine the proportion of the aquifer flow intercepted by the well. This technique is illustrated by *Figure 4.15*, which shows an aquifer whose recharge area (such as an outcrop of the aquifer or a river) is represented by a line. For the purpose of this example it is assumed that the flow from the recharge line to the aquifer is steady and uniform and that there is no other source of recharge. The elevation of the water surface in the aquifer is shown by the groundwater contours and it is apparent that the flow is from A to B. The diagram also shows that a cone of depression (indicated by the closed contour) has developed around the discharging well, with a subsequent re-alignment of the groundwater contours in part of the aquifer. The area of diversion, within which the general movement of groundwater in the aquifer is towards the well, has been defined by drawing limiting flow lines (dashed). Since, by definition, there can be no flow across a flow line, this means that, theoretically, all the flow within the area of diversion should enter the well[35,36]. The length of the recharge line intercepted by the two limiting flow lines can be measured and compared with the perennial yield of the well, estimated by pumping in the manner described above. The flow per unit length of the recharge line can then be calculated and thus the total yield of the aquifer.

Although the method, as applied to the problem in *Figure 4.15*, appears relatively straightforward, it contains many simplifying assumptions that may make it difficult to implement in some situations. For example, it is not easy to draw a flow net for a complex aquifer or to estimate the proportion of the aquifer yield represented by the well abstraction. It should also be remembered that flow nets are drawn in two dimensions, whereas the aquifer and groundwater flow are three dimensional. Further problems may arise due to the fact that a pumping test, with all its inherent complications (see Chapter 7), is a prerequisite of the method. This means that the technique is expensive and requires a considerable length of time to implement. It is also possible that if the well is over-pumped as part of the process of determining its perennial yield, the quality of the water in the aquifer could be affected adversely, possibly on a permanent basis. However, despite these limitations, pumping tests probably provide the best approach to assessing aquifer yield. If there are numerous wells in the area under investigation, they may be pumped simultaneously to determine the yield of the combined well field, or they may be pumped singly and their individual effects added to find the cumulative response to pumping, and hence the yield of the well field. Walton[39] gave some interesting case histories of aquifer tests and flow net analysis.

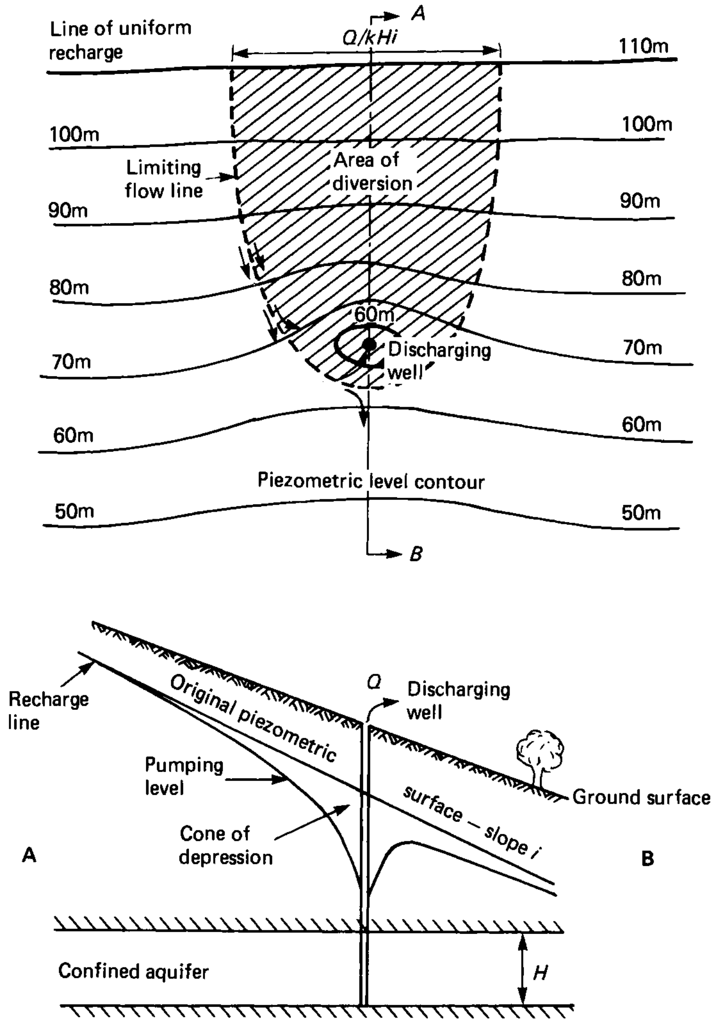

Figure 4.15 Area of diversion to a discharging well (above) and vertical section through A–B (below)

4.5.7 Model studies

It is possible to assess groundwater recharge and perennial yield of wells and aquifers using modelling techniques[41,42,43]. These techniques are considered in Chapter 10.

4.5.8 Supplementary techniques and considerations

An investigation of groundwater recharge may proceed using one or more of the above techniques, but there are alternative approaches that can be employed successfully. Some information regarding recharge mechanisms may be obtained by studying variations in groundwater chemistry[44,45,46]. Although this does not necessarily lead to a quantitative estimate of groundwater recharge, it may give valuable information regarding the preferred flow paths in the aquifer, since fissure flow is the dominant mechanism controlling groundwater movement in most rock aquifers[47]. Reeves *et al.*[29] estimated that preferential flow channels aligned towards the major springheads accounted for over half the total discharge in the limestone/sandstone Corallian Series aquifer in the western part of the Vale of Pickering, England. Headworth[48] also concluded that much of the flow to artesian boreholes in the Chalk of Hampshire was through fissures located in a relatively thin horizon of the aquifer. The presence of substantial fissure flow does, of course, make the use of flow nets and associated techniques a rather dubious way of assessing aquifer recharge.

Kriz[49] described a method of determining the long-term safe rate of abstraction by establishing the relationship between the weekly yield of a group of discharging wells and the groundwater level in an observation hole. In many localities it has been assumed that the perennial yield has not been exceeded so long as the reduction in groundwater level within the area of diversion to a well is proportional to the pumping rate. Unfortunately, under some circumstances, such as where boundary effects are apparent, the relationship between groundwater level and abstraction may be non-linear, or a second limb may develop at a higher rate of abstraction[7]. In these cases caution must be exercised until the reasons for these departures from linearity are established.

Sometimes it may be difficult to determine whether a decline in groundwater level is due to pumpage or to some natural fluctuations in elevation. In such circumstances a regression analysis on rainfall and water level data (see Chapter 10 and Brown *et al.*[1]) may give an equation that enables the natural groundwater level to be predicted.

4.6 Location of wells for maximum yield—summary of factors for consideration

There are many considerations that influence the siting of a well in an aquifer. Some may be concerned with politics, economics, access, or water distribution. However, from the hydrological point of view, the long-term yield of a well depends upon the following factors

1. The annual rate of groundwater recharge. This determines the rate of flow in the aquifer and thus the amount of water available for abstraction.
2. The location of the well in the aquifer. There may be some advantage in siting the well near to a recharge area, so that a surface water resource is diverted underground to augment aquifer flow by induced infiltration. This could increase well yields. Alternatively, the well could be sited near the discharge area of the aquifer with the objective of diverting as much of the natural discharge as possible to the well. Neither of these options should be undertaken without a careful evaluation of the possible consequences. An alternative strategy may be to exploit the storage of the aquifer without interfering with the flow of groundwater through the aquifer. In this case the wells should be sited some distance from the areas of natural recharge and discharge, so that it takes some significant time for the pumping effects to reach them. An indication of the delays to be expected is given by the 'response time', which is SL^2/T, where S is the storage coefficient, T is the transmissivity and L is the length of the flow path from the well to the zone of effected natural discharge[29,50].
3. The permeability of the aquifer in the area surrounding the well. The higher the permeability the easier it is for water to flow to the well during periods of abstraction.
4. The thickness of the aquifer at the well site. The well should be located where the

saturated thickness is greatest. If the aquifer material has only a small variation in permeability then transmissivity will increase with increasing saturated thickness. This again facilitates flow to the well.

5. The location and orientation of any faults or notable discontinuities. These may act as preferred flow channels and greatly increase the flow to a well (see Section 6.12.3). Many wells in relatively impermeable material have been successful as a result of flow through secondary permeability features. However, a fault may also act as a barrier to flow if it is filled with impermeable gouge material or if the throw of the fault places the aquifer against an impermeable stratum. Faults or other potential boundaries should be evaluated using pumping test analysis techniques.

6. The location of wells with respect to any features that may jeopardize the quality and quantity of the discharge, or the groundwater resource as a whole. It is important that a well should be able to operate at its design discharge and drawdown without the quantity and quality of the abstracted water being adversely affected, and without the abstraction having an adverse effect upon ecological or environmental features or resulting in the derogation of existing groundwater sources. For example, induced infiltration can be used to augment a groundwater supply (as in (2) above), but if the surface source is polluted, it can also have the effect of severely limiting the pumping rates of wells in the area[51]. In coastal aquifers saline intrusion may cause a progressive decrease in the quality of the water abstracted from wells and if the discharge becomes unacceptably saline the wells may have to be abandoned. Although these are management problems and as such is described in more detail in Chapter 9, their significance with respect to the location and operation of water wells should not be overlooked, even during the preliminary feasibility or planning stages.

Turc[52] described a systemized two-dimensional search technique for the location of high yield well sites. A less sophisticated procedure is to prepare maps of aquifer permeability, saturated thickness, depth to water, water quality and so on, and then to select the most suitable sites (taking everything into consideration) for investigation and development.

4.7 References

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Chapter 5

Groundwater quality

5.1 Introduction

In any evaluation of groundwater resources the quality of the water is of almost equal importance to the quantity available. In other words, the physical, chemical and biological characteristics of the water are of major importance in determining whether or not water is suitable for domestic, industrial or agricultural use. Furthermore, details appertaining to groundwater quality may throw some light on such factors as the interconnection between surface water and aquifers, groundwater movement and storage.

When water infiltrates into the ground its quality is modified by a number of processes. For example, the amount of precipitation which moves into and evaporation loss from the ground influence the character of the soil water, as does the reaction of the latter with the soil particles. Of particular importance is the role played by water as an agent of chemical weathering which leads to the dissolution of minerals.

Rainwater may leach salts which have accumulated at the surface into the groundwater. As a consequence the intensity is important because downward percolation of salts is much more significant as a result of infrequent heavy downpours rather than prolonged light showers. These salts may be concentrated in the groundwater as a result of evaporation and transpiration. Evaporation is most effective when the air is warm, dry and moving. Water loss by transpiration depends on the type and density of the vegetative cover, which is governed by climate and, to a lesser extent, soil type. Climate also affects the rate of chemical weathering and degree of dissolution of the soluble products. Micro-organisms and plants also influence the composition of groundwater.

The quality of water in the zone of saturation reflects that of the water which has percolated to the water table and the subsequent reactions between water and rock which occur. The factors which influence the solute content include the original chemical quality of water entering the zone of saturation, the distribution, solubility, exchange capacity and exchange selectivity of the minerals involved in reaction, the porosity and permeability of the rocks and the flow path of the water. Of critical importance in this context is the residence time of the water since this determines whether there is sufficient time for dissolution of minerals to proceed to the point where the solution is in equilibrium with the reaction. Residence time depends on the rate of groundwater movement and this usually is very slow beneath the water table.

Yet another factor which affects the quality of water in the zone of saturation is the fluctuation of the water table. In particular, if the water table occurs at shallow depth, then losses by transpiration, and possibly evaporation, will increase when it rises. This means that the salt content increases. Conversely when the water table is lowered this may cause lateral inflow from surrounding areas with a consequent change in salinity.

The uppermost layers of soil and rock act as a purifying agent. In the soil, organisms such as fungi and bacteria attack pathogenic bacteria, as well as reacting with certain other harmful substances. The other important factor in purifying groundwater is the filtering action of soil and rock. This depends on the size of the pores, the proportion of argillaceous and organic matter present and the distance travelled by the water involved. Unconfined water at shallow depth is highly susceptible to pollution but at greater depth recharge is by water which has been partially or wholly purified.

5.1.1 Reactions within the ground

As can be inferred from above, groundwater is a complex chemical solution which is in a dynamic state, the composition of which is in large part attributable to the solution of material in soils and rocks by percolating waters and to chemical reactions between this water and the host medium.

The dissolution of rock-forming minerals commonly is brought about by acid attack. Carbonic acid is formed by the solution of CO_2 in rain water. However, the amount of CO_2 present in the atmosphere is very small. Several hundred times that concentration is found within the soil due to CO_2 being given off by living organisms and by the oxidation of organic matter. For example, according to Thorne and Peterson[1], 2 to 10 l of CO_2/day/m^2 of surface is produced in soil where plants are growing vigorously. Such amounts of CO_2 if dissolved in water would give 550 to 2750 mg/l of HCO_3. By comparison, rain water in equilibrium with the partial pressure of CO_2 in the atmosphere could contain 60 to 100 mg/l of HCO_3. Other acids such as HNO_3, HNO_2 and humic acid are formed as a consequence of the decomposition of organic matter and sulphuric acid is produced when minerals such as pyrite (FeS_2) break down.

The chemical composition of groundwater may be altered by the precipitation of ions from solution to form insoluble compounds. These reactions typically are associated with changes in temperature and pressure. For instance, precipitation of calcium carbonate and release of dissolved carbon dioxide may result from a decrease in pressure and/or an increase in temperature. Precipitation of calcium carbonate may also be associated with increasing pH due, for example, to ion exchange or possibly to the activities of nitrate-reducing bacteria.

An important process affecting the quality of groundwater is ion exchange. Ion exchange involves the replacement of ions adsorbed on the surface of clay minerals, organic compounds and zeolites by ions in solution. Glauconite is also an important ion exchange mineral in some sedimentary rocks. The process usually is associated with cations and therefore is referred to as cation or base exchange. Cation exchange involving replacement of calcium and magnesium in groundwater by sodium takes place as the water flows below an argillaceous formation. For example, the change from hard calcium bicarbonate water to soft alkaline sodium-rich water due to ion exchange is typical of groundwater flowing through areas of the Chalk overlain by thick London Clay[2]. Such reactions produce a soft water in which sodium is the dominant cation. The cation exchange process is a reversible reaction. A further example from the Chalk serves as illustration. In a survey of the pore water in the Chalk at Lawford pumping station in Essex, Hoather[3] concluded that salt water percolating from the estuary of the river Stour had been subjected to cation exchange by which a substantial proportion of its sodium content was replaced by calcium or magnesium so that the hardness of the well was greatly increased.

The cation exchange process means that fine-grained formations, notably clays and shales, can behave like semi-permeable membranes, retarding the movement of ions. Some anions also get trapped in the adsorbed layers of ions. Such behaviour can produce osmotic pressure differences, salt sieving or ultrafiltration, and electric-potential differences. Indeed, it has been suggested that the high salt content of certain groundwaters that have not come in contact with evaporites is due to salt sieving, that is, as water flows out of a formation which behaves as a semi-permeable membrane the salts are retained.

5.1.2 Depth and groundwater zones

In general, quality variations are more likely to occur in shallow than deep aquifers since seasonal changes in recharge and discharge in the former create corresponding fluctuations in salinity.

Strong leaching may occur in areas of active groundwater recharge and consequently they may exhibit relatively low solute concentrations. Nonetheless, there is a frequent tendency for the salt content of groundwater to increase with depth. The reasons for this are, first, the greater the depth at which the groundwater occurs, the slower its movement and so it is less likely to be replaced by other water, especially that infiltrating from the surface. Secondly, the longer residence time of the water means more time for reaction with the host rock and so more material goes into solution until an equilibrium condition is attained. Also connate or fossil water may occur at greater depth. For example, Smith *et al.*[4] used isotope techniques to show that the age of the water in the Chalk of the London Basin increases towards the centre where it exceeds 25 000 years.

As the character of groundwater in an aquifer frequently changes with depth, it is possible at times to recognize zones of different quality of groundwater. With increasing depth, cation exchange reaction increases in importance, and there is a gradual replacement of calcium and magnesium in the water by sodium. As mentioned above, such reactions give rise to soft water. Any nitrates present near the surface of the aquifer invariably decrease with depth. On the other hand, sulphates tend to increase with depth. However, at appreciable depth, sulphates are reduced, which produces a low sulphate–high bicarbonate water. The chloride content also tends to increase with depth. In fact with increasing depth groundwater may become non-potable, due to its high chloride content. Most highly saline, chlorine groundwater (not associated with evaporites) occurring at depth, where groundwater circulation is restricted, is connate or fossil water.

5.2 Physical properties of groundwater

Temperature, colour, turbidity, odour and taste are the most important physical properties of groundwater in relation to water supply. Groundwater only undergoes appreciable fluctuations in temperature at shallow depth, beneath which temperatures remain relatively constant. In fact, the depth at which temperatures are more or less uniform occurs at about 10 m in the tropics, increasing to about 20 m in polar regions, although rock type, elevation, precipitation and wind can produce significant local deviations. Collins[5] recorded that in the USA the temperature of groundwater present within a depth of 10 to 20 m from the ground surface usually is about 1 to 2°C higher than the mean annual air temperature. Below the zone of surface influence, groundwater temperatures increase by approximately 1°C for every 30 m of depth, that is, in accordance with the geothermal gradient.

The colour of groundwater may be attributable to organic or mineral matter carried in solution. For example, light to dark brown discolorations can occur in groundwater which has been in contact with peat or other organic deposits. Brownish discoloration also can result from groundwater which contains dissolved ferrous iron being exposed to the atmosphere. This leads to insoluble ferric hydroxides being formed. Determination of the colour of groundwater is made by comparing a sample of water of given volume with that of another, of equal volume, possessing a standard colour rating of 500[6]. The colour of the water is expressed in units between 0 and 500.

Turbidity of groundwater is mainly caused by the presence of clay and silt particles derived from the aquifer. In other words, turbidity is a measure of the amount of suspended and colloidal material present in groundwater. This may be due to the well concerned being poorly developed or to the screen openings being too large. Oxidation of dissolved ferrous iron to form insoluble ferric hydroxides also contributes towards turbidity. The natural filtration which occurs when groundwater flows through unconsolidated deposits largely removes such material from groundwater.

Tastes and odours may be derived from the presence of mineral matter, organic matter, bacteria or dissolved gases. Most notable among the bacteria which can cause taste and odour problems are Crenothrix, Leptothrix and Gallionella (iron bacteria) and sulphate-reducing bacteria[7]. These problems are overcome by using disinfectant.

5.3 Biological character of groundwater

Standard tests used to determine the safety of groundwater for drinking purposes involve identifying whether or not bacteria belonging to the coliform group are present. One of the reasons for this is that this group of bacteria are relatively easy to recognize. The results of coliform tests are reported in terms of the most probable number (MPN) of coliform group organisms present in a given volume of water (see Section 5.6).

Viruses in groundwater are more critical than bacteria in that they tend to survive longer. In addition one virus unit (one plague-forming unit in a cell culture) may cause an infection when ingested. By contrast, ingestion of thousands of pathogenic bacteria may be required before clinical symptoms are developed.

Fortunately groundwater, except perhaps from very shallow aquifers, is generally free from pathogenic bacteria and viruses. In addition to the rate and direction of groundwater flow, the movement of micro-organisms, pathogenic bacteria and viruses depends on the size of the pores, on reactions within the media, on the amount of food available and on their life span.

Micro-organisms can be carried by groundwater but tend to attach themselves by adsorption to the surfaces of clay particles. In fine-grained soils bacteria generally move less than a few metres, but they can migrate much larger distances in coarse-grained soils or discontinuous rocks. The maximum rate of travel of bacterial pollution appears to be about two-thirds that at which the groundwater is moving. Viruses which retain all their characteristics for more than 50 days may migrate 250 m or more in soils where organic matter is present to supply a food source. The recommended safe distances between domestic wells and sources of pollution in non-karstic terrains are indicated in

Table 8.1. Nonetheless, biological pollution of groundwater generally does not occur because the soil represents a fairly effective filter between a source of pollution and the water table. Pathogenic bacteria, viruses and other micro-organisms not native to the subsurface environment generally do not multiply underground and eventually die.

Most cases of contamination result from poor well construction, from over-abstraction, or are associated with aquifers which posses large pores such as gravel deposits, or open discontinuities such as some limestones. Both afford connection between surface water, which may be polluted, and groundwater. In cavernous or fissured limestone the distances travelled may be several kilometres. Pollution of this type is only important, however, if the water is to be used for potable water supply or food processing.

Temperatures which permit micro-organisms to exist may extend to a depth of 2000 m. Also at this depth water pressures are not high enough to deter microbial activity. Sources of carbon, which are required by micro-organisms to exist, are present in the form of carbonates and other inorganic carbon in rocks. Sedimentary deposits may contain some organic carbon. Oxidation-reduction levels often are within the tolerance range of bacteria. Aerobic bacteria are usually absent in deeper regions as a supply of molecular oxygen is not usually available.

The total amount of organic material, exclusive of gases, is rarely more than 15 mg/l in water drawn from wells.

The biochemical oxygen demand (BOD) is a measure of the biodegradable material in water. As most organic matter has been removed from percolating water by micro-biological processes before it reaches the water table, groundwater BODs are essentially zero. Organic matter in peat deposits is relatively stable and so produces low BOD levels in groundwater.

5.4 Chemistry of groundwater

5.4.1 Major dissolved constituents in groundwater

The chemical elements present in groundwater are derived from precipitation, organic processes which go on in the soil and the breakdown of minerals in the rock through which the groundwater flows. Some of these constituents occur in significant amounts, whilst others are of minor importance or are present as trace constituents (*Table 5.1*).

Chemical constituents in rainwater are present only in small quantities. The principal components are: Cl which tends to vary from 0.1 to 100 mg/l, Na from 0.1 to 50 mg/l, Ca from 0.1 to 20 mg/l, SO_4 from 0.1 to 20 mg/l and Mg from 0.1 to 10 mg/l. But in shallow groundwater evapotranspiration may increase their proportions. The highest concentrations of salts in rainwater are found in coastal areas, that is, up to about 50 km inland from the sea. Moreover, rainwater in arid areas has a higher salt content than in humid areas, the reason being that most of the dissolved salts are of continental origin, being derived from dust in the atmosphere.

As noted above, the solution of carbonates, especially calcium and magnesium carbonate, is principally due to the formation of weak carbonic acid in the soil horizons where CO_2 is dissolved by soil water. Calcium present in groundwater in sedimentary rocks is derived from calcite, aragonite, dolomite, anhydrite and gypsum. Weyl[9] maintained that in the zone of saturation in limestone terrains the solution of calcite normally occurs rapidly enough so that water is always saturated with calcite. By contrast, precipitation of calcite is considerably slower. In igneous and metamorphic

TABLE 5.1. Relative abundance of dissolved solids in potable water (after Davis and DeWiest[14])

Major constituents (1.0 to 1000 mg/l)	*Secondary constituents* (0.01 to 10.0 mg/l)	*Minor constituents* (0.0001 to 0.1 mg/l)	*Trace constituents* (generally less than 0.001 mg/l)
Sodium	Iron	Antimony*	Beryllium
Calcium	Strontium	Aluminum	Bismuth
Magnesium	Potassium	Arsenic	Cerium*
Bicarbonate	Carbonate	Barium	Cesium
Sulphate	Nitrate	Bromide	Gallium
Chloride	Fluoride	Cadmium*	Gold
Silica	Boron	Chromium*	Indium
		Cobalt	Lanthanum
		Copper	Niobium*
		Germanium*	Platinum
		Iodide	Radium
		Lead	Ruthenium*
		Lithium	Scandium*
		Manganese	Silver
		Molybdenum	Thallium*
		Nickel	Thorium*
		Phosphate	Tin
		Rubidium*	Tungsten*
		Selenium	Ytterbium
		Titanium*	Yttrium*
		Uranium	Zirconium*
		Vanadium	
		Zinc	

* These elements occupy an uncertain position in the list

rocks calcium is supplied by the feldspars, pyroxenes and amphiboles and the less common minerals such as apatite and wollastonite.

Calcium carbonate and bicarbonate are the dominant constituents found in the zone of active circulation and for some distance under the cover of younger strata. The normal concentration of calcium in groundwater ranges from 10 to 100 mg/l. Such concentrations have no effect on health and it has been suggested that as much as 1000 mg/l may be harmless.

The reactions involved in the solution of magnesium from carbonate minerals are similar to those for solution of calcite, that is, the solubility of magnesium carbonate is also controlled by the amount of carbon dioxide in the groundwater. However, Garrels *et al.*[10] noted that the solubility of magnesium carbonate is greater than that of calcium carbonate and that the precipitation of dolomite from solution is extremely slow. They went on to suggest that equilibrium conditions with respect to magnesite or dolomite probably do not occur at low temperature–pressure conditions.

Dolomite is the common source of magnesium in sedimentary rocks, the rarer evaporate minerals such as epsomite, kierserite, kainite and carnallite are not significant contributors. Olivine, biotite, hornblende and augite are among those minerals which make significant contributions in the igneous rocks and serpentine, talc, diopside and tremolite are among the metamorphic contributors.

Despite the higher solubilities of most of its compounds (magnesium sulphate and magnesium chloride are both very soluble), magnesium usually occurs in lesser concentrations in groundwaters than calcium. This is probably due to the fact that the dissolution of dolomite is a slow process and that calcium is more abundant in the

Earth's crust. Common concentrations of magnesium range from about 1 to 40 mg/l, concentrations above 100 mg/l are rarely encountered except in sea water and brines.

Most bicarbonate ions in groundwater are derived from carbon dioxide and the solution of carbonate rocks. Bicarbonate concentrations of between 50 and 400 mg/l are commonly found in groundwater. On the other hand, carbonate concentrations in groundwater generally are not greater than 10 mg/l. This is because dissolution of bicarbonate ions to give carbonate ions primarily is effective above a pH value of 8.2. Below this value the ratio of bicarbonate to carbonate ions frequently exceeds 100 to 1.

Sodium salts are highly soluble and will not precipitate unless concentrations of several thousand parts per million are reached. The only common mechanism for removal of large amounts of sodium ions from water is through cation exchange which operates if the sodium ions are in great abundance. The conversion of calcium bicarbonate to sodium bicarbonate no doubt accounts for the removal of some sodium ions from sea water which has invaded freshwater aquifers[3]. This process is reversible.

Sodium does not occur as an essential constituent of many of the principal rock-forming minerals, plagioclase feldspar being the exception. Consequently plagioclase is the primary source of most sodium in groundwaters. Under certain conditions, some clay minerals may release exchangeable sodium ions. Obviously in areas of evaporitic deposits halite is an important source.

All groundwaters contain measurable amounts of sodium, up to 200 mg/l being the most common concentrations. However, in some natural brines concentrations as high as 100 000 mg/l have been recorded.

Common sources of potassium in groundwater are the feldspars and micas of the igneous and metamorphic rocks. Potash minerals such as sylvite occur in some evaporitic sequences but their contribution is not important. Like sodium, potassium is highly soluble and therefore is not easily removed from water except by ion exchange. Although the abundance of potassium in the Earth's crust is similar to that of sodium its concentration in groundwaters is usually less than a tenth of that of sodium. Most groundwaters contain less than 10 mg/l.

Sedimentary rocks such as shales and clays may contain pyrite or marcasite from which sulphur can be derived. However, most sulphate ions are probably derived from the solution of calcium and magnesium sulphate minerals found in evaporitic sequences, gypsum and anhydrite being the most common. The concentration of the sulphate ion in water can be affected by sulphate-reducing bacteria. The products of sulphate reduction are hydrogen sulphide and carbon dioxide, hence a decline in sulphate ion is frequently associated with an increase in the bicarbonate ion. Concentrations of sulphate in groundwaters are usually less than 100 mg/l and may be less than 1 mg/l if sulphate-reducing bacteria are active. In magnesium sulphate brines the concentration sometimes exceeds 100 000 mg/l.

The chloride content of groundwater may be due to the presence of soluble chlorides from rocks, saline intrusion, connate and juvenile waters, or contamination by industrial effluent or domestic sewage. In the zone of actual circulation the chloride ion concentration is normally relatively small. Chloride is a minor constituent in the Earth's crust, sodalite and apatite are the only igneous and metamorphic minerals containing chloride as an essential constituent. Halite is one of the principal mineral sources. As in the case of sulphate, the atmosphere probably makes a significant contribution to the chloride content of surface waters, these, in turn, contributing to groundwaters. Leaching of chlorides that have accumulated in the upper layers of the soil may be a significant source of chloride in dry climates. Groundwaters containing significant amounts of chloride also tend to have high amounts of sodium. Usually the concentration of chloride in groundwater is less than 30 mg/l but concentrations of 1000 mg/l or more are common in arid regions, whereas in brines over 150 000 mg/l have been noted.

Most of the nitrogen in groundwater is probably derived from the biosphere. For example, nitrate ions are derived from the oxidation of decaying organic material, particularly that with a high protein content. In the latter case the presence of nitrate ions may be indicatve of a source of pollution and their occurrence is usually associated with shallow groundwater sources. In addition, molecular nitrogen from the atmosphere can be transformed into organic matter by nitrogen-fixing bacteria which live in nodules on the roots of legumes. When the plants die and decay, the nitrogen is mineralized by soil bacteria into ammonia.

Under anaerobic conditions ammonia can be absorbed by clay minerals and organic matter in soil, otherwise it is oxidized by nitrifying bacteria to form nitrate. Infiltrating water may leach nitrate ions out of the upper layers of the soil.

If the natural nitrogen enrichment processes have been active for some time, nitrate–nitrogen levels in groundwater may be around 1 to 50 mg/l. Kreitler and Jones[11] found NO_3—N concentrations varying from less than 0.2 mg/l to more than 690 mg/l in groundwater in west central Texas. They concluded that most of this nitrate was derived from natural soil nitrogen developed under dry-land farming with minimal use of fertilizer. This example illustrates how nitrate can be concentrated in water leached from the root zone.

Concentrations of nitrate in fresh water do not generally exceed 5 mg/l, although in rural areas where nitrate fertilizer is liberally applied, concentrations may exceed 600 mg/l. Nitrite occurs in groundwater at much lower concentrations than nitrate. Ammonia–nitrogen levels in groundwater seldom exceed a few milligrams per litre. Organic nitrogen occurs in negligibly small concentrations.

Although silicon is the second most abundant element in the Earth's crust and is present in almost all the principal rock-forming minerals, it is not one of the most abundant constituents of groundwater. This is because silica (SiO_2) is only slightly soluble in water. The dissolution of silicate minerals other than quartz and clay minerals, accounts for most of the silica present in solution in groundwater. At pH values of less than 9 and at a temperature of 25°C, silicate material occurs in solution as dispersed molecules of silicic acid (H_4SiO_4) in a non-ionized state, when its concentration is less than 100 to 140 mg/l[12]. At higher concentrations, silicic acid polymerizes to form colloidal gels. Above a pH value of 9, silicate ions, because of the dissociation of silicic acid, do occur in solution in significant amounts and the solubility of silicates increases rapidly, it becoming very high, rising from 200 to more than 2000 mg/l of SiO_2 at a pH value of 12. Silica also becomes more soluble when the temperatures are above 25°C. Groundwater, however, generally contains between 5 to 40 mg/l of silica, although high values may be recorded in water from volcanic rocks.

Iron forms approximately 5 per cent of the Earth's crust and is contained in a great many minerals in rocks as well as occurring as ore bodies. The chemical weathering of these minerals makes available large amounts of iron but this is usually converted into insoluble iron oxides.

Normally iron occurs in groundwater in the form of Fe^{2+}, $Fe(OH)_3$ or $FeOH^+$ (see Hem[6]). The form and amount of iron in solution in groundwater at chemical equilibrium is controlled by the nature of the iron minerals present, the pH value and redox potential (Eh), and the activity of other ions in solution, notably the concentration of HCO_3^-. When iron occurs in groundwater in concentrations of 1 mg/l or above, it does so in the ferrous state and it is retained in solution by a pH appreciably below 7.0 or a low redox potential. The latter commonly occurs in groundwater which is not in contact with air. When such groundwater is exposed to air, oxygen raises the Eh of the solution, iron is then oxidized to the ferric condition and precipitated as ferric hydroxide, causing a brown coloration to the water. The solubility of ferric iron only exceeds 1 mg/l when the pH value falls below 3.8 or at a high value of Eh. Concentrations of ferrous iron in groundwater are normally in the range 1 to 10 mg/l.

5.4.2 Minor elements, trace elements and radionuclides

Durum and Haffty[13] defined minor elements in groundwater as those which generally do not exceed 1 mg/l. The presence of minor and trace elements in water may affect the health of animals and plants.

Several minor elements are a matter of concern because of their toxic effects. For instance, arsenic should not exceed 0.01 to 0.1 mg/l in drinking water, neither barium nor copper should exceed 1 mg/l, neither chromium nor lead 0.05 mg/l, cadmium 0.01 mg/l or mercury 0.002 mg/l.

Although one of the most abundant elements in the Earth's crust, the amount of aluminium, because of its low solubility, usually ranges between 0.005 and 0.3 mg/l in normal groundwater and it rarely contains more than 0.5 mg/l.

Manganese is released into groundwater by the weathering of minerals such as biotite and hornblende. When exposed to the atmosphere Mn^{2+} is oxidized to form much less soluble hydrated oxides. Most waters contain less than 0.2 mg/l of manganese.

Fluorite, apatite and some amphiboles act as sources of fluoride for groundwater. Apatite, however, has a very low solubility. Natural concentrations of fluorine in groundwater frequently range from about 0.01 to 10 mg/l. Indeed, fluoride concentrations sometimes exceed 10 mg/l and have been known to reach more than 30 mg/l. These high concentrations tend to be associated with high pH. The amount of fluorine present appears to be limited by the solubility of fluorite (CaF_2) which is approximately 9 mg/l fluoride in pure water. However, water with a high content of calcium does not contain more than about 1 mg/l of fluoride.

Most water contains only about 1 mg/l of bromide for every 300 mg/l of chloride. Bromide does not adversely affect the health of animals or plants when present in normal concentrations in groundwater.

Iodine is concentrated by plants and animals so that after death decomposition releases it into groundwater.

Boron occurs in tourmaline and is emitted as boron fluoride and boric acid during volcanic eruptions. Hence water in volcanic areas may contain high concentrations of boron. Although boron is essential to healthy plant growth, it is injurious if present in groundwater in significant quantities. However, the sensitivities of plants to boron vary widely. For example, citric trees may be damaged by as little as 0.5 mg/l whereas alfalfa can tolerate 10 mg/l if soil drainage is good. The normal amount contained by groundwater varies from 0.01 to 1.0 mg/l.

The concentration of strontium in groundwater is probably limited by cation exchange with calcium-rich clays. In most groundwaters its concentration ranges between 0.01 and 1.0 mg/l.

The largest number of natural radionuclides in groundwater are called primordial radionuclides and these have exceptionally long half-lives[14]. Some of these radionuclides break down to give daughter products that are also radioactive. The most abundant radionuclides are K^{40}, Rb^{87}, Th^{233}, U^{235} and U^{238}. Uranium concentrations in groundwater usually range between 0.0001 and 0.01 mg/l. The daughter products of importance are Rn^{222} and Ra^{226} which are derived from U^{238}. Another group of natural radionuclides originates from cosmic ray activation of the stable nuclides N^{14}, O^{16} and Ar^{40}. The best known products of this activation are tritium (H^3) and carbon-14 (C^{14}). Radionuclides tend to be readily adsorbed by clay minerals and organic matter.

5.4.3 Total dissolved solids and hardness of groundwater

The total dissolved solids (TDS) in a sample of groundwater includes all solid material in solution, whether ionized or not. The amount of dissolved solids in natural waters ranges from less than 10 mg/l in rain and snow to over 300 000 mg/l in some brines. Water for most domestic and industrial uses should contain less than 1000 mg/l and the TDS content of water for most agricultural purposes should not be above 3000 mg/l. Groundwater has been classified by Hem[6] according to its total dissolved solids content as follows

Fresh	Less than 1000 mg/l
Moderately saline	3000 to 10 000 mg/l
Very saline	10 000 to 35 000 mg/l
Briny	Over 35 000 mg/l

The hardness of water relates to its reaction with soap and to the scale and incrustations which form in boilers and pipes where water is heated and transported. It is attributable to the presence of divalent metallic ions, calcium and magnesium being the most abundant in groundwater. Waters derived from limestone or dolostone aquifers containing gypsum or anhydrite may contain 200 to 300 mg/l hardness or more. Water for domestic use should not contain more than 80 mg/l total hardness.

Hardness, H_T, generaly is expressed in terms of the equivalent of calcium carbonate, hence

$$H_T = Ca \times (CaCO_3/Ca) + Mg \times (CaCO_3/Mg) \tag{5.1}$$

where H_T, Ca and Mg are measured in mg/l and the ratios in equivalent weights. This equation reduces to

$$H_T = 2.5Ca + 4.1Mg \tag{5.2}$$

The degree of hardness in water has been described as follows

Description	*Hardness* (mg/l as $CaCO_3$)	
	(after Sawyer and McCarty[15])	(after Hem[6])
Soft	Below 75	Below 60
Moderately hard	75–150	61–120
Hard	150–300	121–180
Very hard	Over 300	Over 180

5.4.4 Gases in and pH of groundwater

The solubility of most gases in water is directly proportional to the pressure and inversely proportional to the temperature conditions prevailing. Rainwater contains dissolved gases in concentrations that are in approximate equilibrium with the gases in the atmosphere. But when rainwater infiltrates through the soil, the content of dissolved oxygen is reduced by bacterial activity and oxidation of minerals. Nitrogen also may be depleted by certain bacteria. However, once the water has percolated to the zone of saturation, its gas content tends to remain more or less unchanged. Dissolved gas occurs in concentrations of 1.0 to 100.0 mg/l in most groundwater.

Certain dissolved gases, notably oxygen and carbon dioxide, affect groundwater chemistry. Moreover, dissolved oxygen promotes corrosion of metals. Methane coming out of solution may accumulate and present a fire or explosion hazard. The minimum concentration of methane in water sufficient to produce an explosive methane–air mixture above the water from which it has escaped depends on the volume of air into which the gas evolves. Theoretically water containing as little as 1 to 2 mg/l of methane can produce an explosion in poorly ventilated air space.

Hydrogen sulphide in concentrations exceeding 1.0 mg/l renders water unfit for consumption because of the objectionable odour. What is more, hydrogen sulphide promotes the growth of bacteria which block well screens and pipes.

When gases are liberated from solution they form bubbles which may obstruct the flow of water in the pores of aquifers.

Acid waters normally are comparatively rare. Most natural waters are within a pH range of 6 to 8.5[6]. Acidity can be caused by the solution of carbon dioxide, by the oxidation of sulphide minerals, by the leaching of organic acids from decaying vegetation, by the hydrolysis of iron and aluminium or by the presence of hydrochloric acid or sulphuric acid in areas of volcanic activity.

5.5 Rock types and groundwater

The chemical quality of water abstracted from plutonic igneous or metamorphic rocks is nearly always excellent with salt concentrations commonly below 100 mg/l and seldom over 500 mg/l (*Table 5.2*). Exceptions are found in arid areas where salts may be concentrated in the recharge water by evaporation. The principal reason for the generally low salt content of such waters is that silicate minerals normally are resistant to solution.

The ratio of calcium to sodium is often lower in groundwater in granites than in other rocks and the occasional tendency for sodium to predominate over other cations depends on the type of feldspars present. The magnesium content, like that of calcium, generally is low. As no bicarbonate ions are dissolved from granitic rocks, the HCO_3/free CO_2 ratio is low. Hence the bicarbonate content of the groundwater normally is about half that of the average of other rocks. This in turn, means that the water has a pH value usually between 4 and 6. When the groundwater comes to the surface, CO_2 escapes, the pH value rises and the water may become alkaline through the predominance of sodium. The proportion of silica contained in these groundwaters may be high. In diorites and syenites the silica content may range up to 55 mg/l. Groundwater from granitic terrains normally contains little Cl and SO_4. However, if the rocks are rich in pyrite, its oxidation gives rise to SO_4 ions. Iron from ferromagnesium minerals then precipitates.

Water derived from basic plutonic igneous rocks normally has high ratios of calcium

TABLE 5.2. Examples of groundwater quality from different types of rocks (mg/l)

Rock type	TDS	Ca	Mg	HCO_3	Na	K	Cl	SO_4	Fe	SiO_4	*Location*
Granite	223	27	6.2	93	9.5	1.4	5.2	32	1.6	39	Maryland
Rhyolite	148	12	2.2	80	6.8	0.6	2.0	0.1	1.1	39	N. Carolina
Gabbro	359	32	16	203	25	1.1	13	10	0.06	56	N. Carolina
Basalt	505	62	28	294	24	—	37	30	—	30	Hyderabad
Diorite	347	72	4.1	114	10	2.8	6.5	115	0.04	22	N. Carolina
Syenite	80	9.5	2.3	38	2.8	0.6	2.1	2.8	0.14	19	New York
Andesite	70	12	0.5	38	1.8	2.6	—	6.3	—	8.9	Idaho
Quartzite	52	1.6	5.8	18	2.8	—	9.9	2.0	—	8	Transvaal
Marble	236	39	10	162	2.7	0.3	3.8	2.4	0.03	9.9	Alabama
Schist	221	27	5.7	138	16	0.7	2.5	9.6	0.11	21	Georgia
Gneiss	135	19	5.1	39	4.4	3.2	5.8	30	0.09	13	Connecticut
Sandstone	210	40	12	67	7.6	0.4	19	26	—	12	Worcestershire
Sandstone	439	65	38	326	44	—	63	79	—	14	France
Arkose	101	9.6	1.9	38	5.1	—	1.8	7.4	0.2	35	Colorado
Greywacke	553	74	20	381	34	1.2	2.7	26	0.62	12	New York
Limestone	247	48	5.8	168	4	0.7	0.1	4.8	0.05	8.9	Florida
Limestone	720	124	28	460	14	3	18	57	0.22	9.2	Tennessee
Chalk	384	115	5	152	10.2	1.2	20	39	—	1.1	Hertfordshire
Dolostone	546	67	39	390	7.6	0.4	—	17	—	24	Transvaal
Gypsum	2480	636	43	143	16.1	0.9	24	1570	—	29	New Mexico
Lignite	2580	74	53	702	624	5.4	25	1080	0.9	11	North Dakota
Shale	260	29	16	126	12	1.1	12	22	0.02	16	New Jersey
Shale	1100	123	70	539	61	2.2	3.5	283	1.3	19	Ohio
Alluminum	371	45	20	207	16	2.6	17	35	0.05	25	Nevada
Glacial deposits	548	86	27	33	5.1	3	6	60	—	24	Minnesota

to sodium and of magnesium to calcium. The magnesium and silica content of water from gabbro usually is higher than that of water from acidic rocks because its component minerals break down more readily. Where the pH and Eh are low, the iron and magnesium content is relatively high. However, water from terrains of gabbroic rocks may be slightly alkaline with total dissolved solids averaging 230 mg/l.

Groundwater which occurs in marble is normally similar to that present in limestone and it may have a moderate to high hardness. The water from quartzite is similar to that of water from silica-rich sandstone. Many waters from quartzite are low in silica and total dissolved solids, and have a high ratio of potassium to sodium. Their pH value generally is low, because there are very few minerals to react with dissolved carbon dioxide. The groundwater from serpentinite and amphibolite usually contains moderately high concentrations of magnesium.

Most water issuing from volcanic rocks is of good to excellent chemical quality (*Table 5.2*). Generally it tends to be a calcium–magnesium–bicarbonate water, or in the case of acid volcanic rocks a sodium bicarbonate water with relatively large amounts of silica. But in the neighbourhood of hot springs and fumaroles the chemical quality may be poor, the water may have a high sodium and chloride content or the amounts of fluorine may be unacceptable.

Siliceous rocks such as very pure sandstone contain few soluble substances. Water passing through these rocks has a low salt content (20 to 300 mg/l with the average of 220 mg/l). The calcium content frequently ranges from 4 to 60 mg/l, with an average value of 25 mg/l, and the HCO_3 content is about half of that normally found in water from other rocks. Usually the values vary between 12 and 160 mg/l, the average value being around 60 mg/l. The amount of Cl and SO_4 present is very low. All the dissolved matter present in the groundwater may not be derived from the host rocks, for example, its initial source may be rainwater, subsequent evapotranspiration concentrating the salt content.

In other sandstones the ratio of calcium to sodium, potassium to sodium, bicarbonate to chloride and sulphate to chloride frequently are a little higher than those for most waters from igneous rocks, but the content of silica is generrally less[16]. Occasionally waters from sandstones contain notable amounts of iron. For example, in several areas groundwater from the Lower Greensand is ferruginous.

Generally, the chemical quality of water obtained from sandstones at or near the surface is good, it usually containing less than 100 mg/l total dissolved solids (*Table 5.2*). Some waters, for example, from the Millstone Grit and the Bunter Sandstone, are noted for their softness. On the other hand, where sandstones are in contact with beds of marl, such as the Keuper Sandstone, the groundwater may be moderately to very hard. Groundwater at depth may be saline. This generally is either fossil or connate water, as for example, occurs in the Permo-Triassic sandstones in the north of the Cheshire basin where fresh water, usually less than 300 m in depth, rests upon salt water.

Most fine-grained sediments were deposited in saline environments. Because of their low permeability the original water in which these deposits accumulated may have been retained. Such connate waters generally possess more than 100 mg/l dissolved matter.

Groundwater in contact with argillaceous rocks may become charged with salts. Cation exchange may occur, sodium, calcium and magnesium being interchanged. Sulphate may be derived from the breakdown of pyrite or gypsum.

When water under pressure infiltrates through a bed of argillaceous rock, the clay minerals present may act as a filter. Cations pass through if their diameter is sufficiently small, as do bicarbonate molecules, but anions are rejected. Hence the concentration of salts in the groundwater moving out of the clay is diluted, the amount of chloride, calcium and magnesium it contains being reduced, but it is richer in sodium and bicarbonate ions.

Limestone is dissolved when attacked by acids, the most important of which is carbonic acid. In the latter case the essential factor regulating solution is the partial pressure of CO_2, the ionic strength of the water and its temperature also being important. Additional factors influencing solution include the velocity of groundwater flow and the area of contact between the water and rocks concerned[17]. In massive limestones, movement via open discontinuities, is rapid but the area of contact with water is small in relation to the volume circulating. Since such limestone are often relatively pure, the water has a very low salt content. The predominant constituents are bicarbonate and calcium (*Table 5.2*), there being relatively little sulphate, chloride or sodium. The magnesium content depends upon the amount of magnesium in the carbonate. In addition to calcium carbonate, many limestones contain dolomite, anhydrite, gypsum, clay minerals and silica in varying amounts and these may influence the chemical character of the groundwater. Water from limestone is usually hard and is generally slightly alkaline. For example, water drawn from the Carboniferous Limestone, the Lincolnshire Limestone and the Great Oolite Limestone is usually hard to very hard. Fossil water may be present in limestones, for instance, Edmunds and Walton[18] reported the occurrence of saline water in the Lincolnshire Limestone to the east away from the outcrop.

Like other discontinuous rock masses, limestones may be susceptible to pollution as discontinuities and sinkholes provide easy access for both organic and inorganic contaminants, natural filtration being absent.

Water from chalk is usually hard, the total hardness varying between 200 and 300 mg/l. The presence of overlying deposits can effect the hardness, for example, in parts of East Anglia, the total hardness of groundwater from the Chalk beneath the chalky boulder clay may exceed 500 mg/l. By contrast, beneath the London Clay it may only amount to 20 mg/l. In fact, much of the groundwater in the Chalk beneath the London Clay shows an excess of alkalinity over hardness due to the presence of sodium bicarbonate. This is due to cation exchange softening, the calcium in the bicarbonate having been replaced by sodium. Another characteristic of these waters is that the chloride content is higher than that normally found in groundwater from the Chalk, although it varies from a fairly low figure of about 120 mg/l (expressed as chlorine) in the central London area to as much as 750 mg/l in West Mersea in Essex. Hoather[3] suggested that this may be due to the presence of connate water.

The principal carbonate mineral in dolostone is dolomite which means that magnesium, calcium and bicarbonate are the most important ions present in groundwater derived therefrom. The water generally is very hard. For example, water from the Magnesium Limestone is very hard, the average hardness tends to vary between 300 and 450 mg/l.

The groundwater in rock sequences containing gypsum or anhydrite, because of the solubility of these minerals[19] may contain large quantities of calcium and sulphate. About 660 mg/l of calcium can be contained in water saturated with gypsum. If the concentration of calcium is high, this may lead to a reduction in the amount of bicarbonate present.

Groundwater which has been in contact with carbonaceous rocks, for example, peat, lignite or coal, is distinguished by a negative redox potential. This commonly gives rise to an abundance of iron, the reduction of sulphates with a consequent decrease in the amount of sulphate, the possible presence of sulphur (S^{2-}) and hydrogen sulphide (H_2S), and possibly to an increase in bicarbonate. When such groundwater is exposed to the air, ferrous iron is oxidized to the ferric state and $Fe(OH)_3$ precipitates. Unless present in large quantities, the hydrogen sulphide content does not necessarily affect potability adversely, because it rapidly escapes into the air. Nevertheless, its presence, together with that of CO_2 and a negative redox potential, make such water very corrosive.

The chemical quality of water from most river deposits is good. However, where groundwater development is intense, the aquifers are subject to direct recharge from the rivers so that their water quality is influenced by the quality of the river water. A high content of dissolved matter can be present in such interrelated systems. Connate water may occur in alluvial deposits, especially near their base and this is likely to be of poor quality. Bacterial pollution of wells in aluvium is rare due to the filtration qualities of the sands, silts and muds.

The chemical quality of water in active circulation in glacial deposits is usually good. By contrast, where it has remained stagnant it may contain so many dissolved salts that it is unusable. It is almost always free from pathogenic organisms.

Because dune sands are often clean and composed of relatively inert materials, the water extracted from them is of good quality. The quality of water found in marine and coastal deposits varies. Near the surface, water may have a low content of dissolved solids, sodium, calcium and bicarbonate being the most abundant ions. Such water is of good quality. With depth the content of sodium carbonate frequently increases.

Connate water may also be present at depth, as may be sea water which has invaded the aquifer.

5.6 Uses of groundwater

5.6.1 Drinking water

Water for human consumption must be free from organisms and chemical substances in concentrations large enough, to affect health adversely[20]. In addition, drinking water should be aesthetically acceptable in that it should not possess unpleasant or objectionable taste, odour, colour or turbidity. For example, the maximum concentration of chloride in drinking water is 250 mg/l, primarily for reasons of taste. Again for reasons of taste, and also to avoid straining, the recommended maximum concentration of iron in drinking water is 0.3 mg/l. Oxidation of manganese in groundwater can produce black stains and therefore the maximum permitted concentration of manganese for domestic water is 0.05 mg/l. Although the temperature of drinking water is not too critical, cool water generally is preferred.

European and international standards of drinking water, laid down by the World Health Organisation, are given in *Table 5.3*. Most drinking water in the USA conforms with standards established by the US Environmental Protection Agency (*Table 5.4*). The permissible and mandatory limits of trace elements present in drinking water are recorded in *Table 5.5*. The values quoted cannot be universally applied.

The bacterial quality of drinking water which has been established by the US Environmental Protection Agency, requires that tests reveal no more than one total coliform organism per 100 ml as the arithmetic mean of all water samples examined per month, with no more than 4 per 100 ml in more than any sample if the number of samples is less than 20 per month, or no more than 4 per 100 ml in 5 per cent of the samples if the number of samples is more than 20 per month. Samples should be taken at frequent intervals of time. These vary according to the population supplied, for example, one sample per month if less than 1000 is supplied to 300 per month if 1 000 000 is supplied.

Drinking water should contain less than 1 virus unit per 400 to 4000 l[21]. Viruses can be eliminated by effective chlorination. The water should be free from suspended solids in which viruses could harbour and thereby be protected against disinfectant.

The pH value of drinking water should be close to 7 but treatment can cope with a range of 5 to 9. Beyond this range treatment to adjust the pH to 7 becomes less economical.

See Appendix II for a brief note on groundwater treatment processes.

5.6.2 Industrial water

The quality of water required in different industrial processes varies appreciably, indeed it can differ within the same industry. Nonetheless, salinity, hardness and silica content are three parameters that usually are important in terms of industrial water. On the one hand, water used in the textile industries should contain a low amount of iron, manganese and other heavy metals likely to cause staining. Hardness, total dissolved solids, colour and turbidity also must be low. On the other hand, the quality of water required by the chemical industry varies widely depending on the process involved. Similarly water required in the pulp and paper industry is governed by the type of products manufactured. Limits of salt concentrations, recommended by the American

TABLE 5.3. Standards for drinking water (after WHO[8])

	European standards (1971)	*International standards (1972)*
*Biology**		
Coliform bacteria	Nil	Nil
Escherichia coli	Nil	Nil
Streptococcus faecalis	Nil	Nil
Clostridium perfringens	Nil	
Virus	Less than 1 plaque forming unit per litre per examination in 10 l of water	
Microscopic organisms	Nil	
Radioactivity		
Overall α radioactivity	<3 pCi/l	<3 pCi/l
Overall β radioactivity	<30 pCi/l	
Chemical elements	(mg/l)	(mg/l)
Pb	<0.1	<0.1
As	<0.05	<0.05
Se	<0.01	<0.01
Hexavalent Cr	<0.05	<0.05
Cd	<0.01	<0.01
Cyanides (in CN)	<0.05	<0.05
Ba	<1.00	<1.00
Cyclic aromatic hydrocarbon	<0.20	
Total Hg	<0.01	<0.01
Phenol compounds (in phenol)	<0.001	<0.001–0.002
NO_3 recommended	<50	
acceptable	50–100	
not recommended	>100	
Cu	<0.05	0.05–1.5
Total Fe	<0.1	0.10–1.0
Mn	<0.05	0.10–0.5
Zn	<5	5.00–15
Mg if $SO_4 > 250$ mg/l	<30	<30
if $SO_4 < 250$ mg/l	<125	<125
SO_4	<250	250–400
H_2S	0.05	
Cl recommended	<200	
acceptable	<600	
NH_4	<0.05	
Total hardness	2–10 meq/l	2–100 meq/l
Ca	75–200 mg/l	75–200 mg/l

F In the case of fluorine the limits depend upon air temperature:

Mean annual maximum day time temperature (°C)	*Lower limit* (mg/l)	*Optimum* (mg/l)	*Upper limit* (mg/l)	*Unsuitable* (mg/l)
10–12	0.9	1.2	1.7	2.4
12.1–14.6	0.8	1.1	1.5	2.2
14.7–17.6	0.8	1.0	1.3	2.0
17.7–21.4	0.7	0.9	1.2	1.8
21.5–26.2	0.7	0.8	1.0	1.6
26.3–32.6	0.6	0.7	0.8	1.4

* No 100 ml sample to contain *E. coli* or more than 10 coliform

TABLE 5.4. Drinking water standards in the United States[a]

Physical characteristics		
Criterion	*Recommended limit*[b]	*Tolerance limit*[c]
Colour, units	15	—
Odour, threshold number	3, inoffensive	—
Residue:		
Filtrable, mg/l	500	—
Taste	Inoffensive	—
Turbidity, units	5	—

Inorganic chemicals (mg/l)		
Substance	*Recommended limit*[b]	*Tolerance limit*[c]
Alkyl benzene sulphonate (ABS)	0.5	—
Arsenic (As)	0.01	0.05
Barium (Ba)	—	1.0
Cadmium (Cd)	—	0.01
Carbon chloroform extract (CCE)	0.2	—
Chloride (Cl)	250	—
Chromium, hexavalent (Cr^{+6})	—	0.05
Copper (Cu)	1.0	—
Cyanide (CN)	0.01	0.2
Fluoride (F)	0.8–1.7[d,e]	1.4–2.4[e]
Iron (Fe)	0.3	—
Lead (Pb)	—	0.05
Manganese (Mn)	0.05	—
Mercury (Hg)	—	0.002
Nitrate (as N)	10	—
Phenolic compounds (as phenol)	0.001	—
Selenium (Se)	—	0.01
Silver (Ag)	—	0.05
Sulphate (SO_4)	250	—
Zinc (Zn)	5	—

Organic chemicals (mg/l)	
Substance	*Tolerance limit*
(A) Chlorinated hydrocarbons	
Endrin	0.0002
Lindane	0.004
Methoxychlor	0.1
Toxaphene	0.005
(B) Chlorophenoxys	
2,4-D	0.1
2,4,5-TP Silvex	0.01

TABLE 5.4. *continued*

TABLE 5.4. *continued*

Biological standards	
Substance examined	*Maximum permissible limit*
Standard 10 m/ portions	Not more than 10 per cent in one month shall show coliforms[f]
Standard 100 m/ portions	Not more than 60 per cent in one month shall show coliforms[f]

Radioactivity ($\mu\mu c/l$)	
Substance	*Recommended limit*
Radium 226 (Ra^{226})	3
Strontium 90 (Sr^{90})	10
Gross beta activity	1000[g]

[a] Based on maximum contaminant levels of the US Environmental Protection Agency (*Federal Register*, v. 40, no. 248, pp. 59566–59588, December 24, 1975)
[b] Concentrations that should not be exceeded where more suitable water supplies are available
[c] Concentrations above this constitute grounds for rejection of the supply
[d] Dependent on annual maximum daily air temperature
[e] Where fluoridation is practiced, minimum recommended limits are also specified
[f] Subject to further specified restrictions
[g] In absence of strontium 90 and alpha emitters

Water Works Association[22], in water used in a number of industries are given in *Table 5.6*.

The quality of water used by a particular industrial process may need to be relatively constant. If water is of poor quality, it can be treated to bring it to the standard required by the process concerned. If, however, the quality of the water used varies widely from time to time, this means that the treatment becomes more expensive. Fluctuations in water temperature also can prove troublesome. Groundwater generally is preferable to surface water since it shows less variation in chemical and physical quality.

5.6.3 Agricultural uses

Maximum concentrations of total salt and specific ions in drinking water for farm animals are given in *Table 5.7(a)* and *(b)*. In terms of total dissolved solids the US standard is more rigorous than the Australian. Nonetheless, a higher total dissolved solids content than the 3000 mg/l probably is suitable for animals other than poultry.

The suitability of groundwater for irrigation depends on the effects that the salt concentration contained therein has on plants (*Table 5.8*) and soil[23]. Salts may harm plant growth in that they reduce the uptake of water either by modifying osmotic processes or by metabolic reactions such as those caused by toxic constituents. The effects that salts have on some soils, notably the changes brought about in soil fabric which, in turn, affect permeability and aeration, also influence plant growth. In other words, cations can cause deflocculation of clay minerals in a soil which damage its crumb structure and reduce its infiltration capacity. The quality of water used for irrigation varies according to the climate, the type and drainage characteristics of the soil and the type of crop grown. Plants growing in adverse climatic conditions, are susceptible to injury if poor-quality water is used for irrigation. Also in hot, dry climates plants abstract more moisture from the soil and so tend to concentrate dissolved solids in the soil moisture quickly. Clayey soil may cause problems when poor-quality water

TABLE 5.5. Minor and trace elements and compounds in drinking water (from UNESCO, 1972[8])

	Comment	*Recommended limit* (mg/l)	*Mandatory limit* (mg/l)	*Unofficial limit* (mg/l)
Alkyl benzine Sulphonate (ABS)		0.5		
Arsenic (As)	Serious cumulative systemic poison. 100 mg usually causes severe poisoning	0.01	0.05	
Antimony (Sb)	Similar to As but less acute. Recommended limit 0.1 mg/l, routinely below 0.05 mg/l; over long periods below 0.01 mg/l	0.05	0.05	
Barium (Ba)	Muscle (including heart) stimulant. Fatal dose is 550–600 mg as chloride		1	
Beryllium (Be)	Poisonous in some of its salts in occupational exposure			None
Bismuth (Bi)	A heavy mineral in the arsenic family—avoid in water supplies			None
Boron (B)	Ingestion of large amounts can affect central nervous system	1	5	1
Cadmium (Cd)	13–15 ppm in food has caused illness		0.01	
Carbon chloroform extract (CCE)	At limit stated, organics in water are not considered a health hazard	0.200		
Chromium (hexavalent)	Limit provides a safety factor. Carcinogenic when inhaled		0.05	
Cobalt (Co)	Beneficial in small amounts; about 7 μg/day			None
Copper (Cu)	Body needs copper at level of about 1 mg/day for adults; not a health hazard unless ingested in large amounts	1.0		
Cyanide (CN)	Rapid fatal poison, but limit set provides safety factor of about 100	0.01	0.20	
Fluoride (F^-)	Beneficial in small amounts; above 2250 mg dose can cause death	0.7–1.2	1.4–2.4	
Iron (Fe)		0.3		
Lead (Pb)	Serious cumulative body poison		0.05	
Manganese (Mn)		0.05		
Mercury (Hg)	Continued ingestion or large amounts can damage brain and central nervous system			0.005
Molybdenum (Mo)	Necessary for plants and ruminants. Excessive intakes may be toxic to higher animals; acute or chronic effects not well known			None
Nickel (Ni)	May cause dermatitis in sensitive people; doses of 30–73 mg of $NiSO_4 \cdot 6H_2O$ have produced toxic effects			None
Radium (Ra-226)	A bone-seeking, internal alpha emitter that can destroy bone marrow	3 pc/l		
Selenium (Se)	Toxic to both humans and animals in large amounts. Small amounts may be beneficial		0.01	
Silver (Ag)	Can produce irreversible, adverse cosmetic changes		0.05	
Sodium (Na)	A beneficial and needed body element, but can be harmful to people with certain diseases			200
Strontium-90	A bone-seeking internal beta emitter	10 pc/l		
Zinc (Zn)	Beneficial in that a child needs 0.3 mg/kg/day; 675–2280 mg/l may be an emetic		5	

TABLE 5.6. Ranges in recommended limiting concentrations for industrial process waters (except where noted, units are in mg/l) (after Am. Water Works Ass.[22])

Use	*Turbidity, units,*	*Colour, units*	*Taste and odour threshold*	*Dissolved solids*	*Hardness, as $CaCO_3$*	*Alkalinity, as $CaCO_3$*	*pH, units*	*Chlorides, as Cl*	*Sulphates, as SO_4*	*Iron, as Fe*	*Manganese, as Mn*	*Iron plus manganese*	*Hydrogen sulphide*	*Fluorides, as F*	*Other requirements*
Air-conditioning	—	—	Low	—	—	—	—	—	—	0.5	0.5	0.5	—	—	Not corrosive or slime-promoting
Baking	10	10	None–low	—	[a]	—	—	—	—	0.2	0.2	0.2	0.2	—	Potable
Boiler feed															Potable if steam is used for food preparation
Brewing	0–10	0–10	None–low	500–1500[b]	[c]	75–80[d]	6.5–7.0[e]	60–100	—	0.1	0.1	0.1	0.2	1.0	Potable, numerous other requirements
Carbonate beverages	1–2	5–10	None–low	850	200–250	50–130	—	250	250	0.1–0.2	0.2	0.1–0.4	0.02	0.2–1.0	Potable; COD, 1.5; organic matter, infinitesimal; algae and protozoa, none
Confectionery	—	—	Low	50–100	Soft	—	> 7.0	—	—	0.2	0.2	0.2	0.2	—	Potable
Dairy	—	None	None	500[f]	180	—	—	30	60	0.1–0.3	0.03–0.1	—	—	—	Potable; NO_3-N, 5.5; NO_2-N, 0; NH_3N, trace only; COD as $KMnO_4$, 12
Drinking	5	15	3, inoffensive	500	—	—	—	250	250	0.3	0.05	—	—	1.4–2.4[g]	Potable
Food canning and freezing	1–10	—	None–low	850	[h]	30–250	> 7.5	—	—	0.2	0.2	0.2–0.3	1.0	1.0	Potable; free from saprophytic organisms; NaCl, 1000–15000; NO_3-N, 2.8; NH_3-N, 0.4
Food equipment, washing	1	5–20	None	850	10	—	—	250	—	—	—	0.1	—	1.0	Potable; organic matter, infinitesimal
Food processing,	1–10	5–10	Low	850	10–250	30–250	—	—	—	0.2	0.2	0.2–0.3	—	1.0	Potable
Ice manufacture	5	5	Low	170–1300	—	—	—	—	—	0.2	0.2	0.2	—	—	Potable; SiO_2, 10
Laundering	—	—	—	—	0–50	60	6.0–6.8[e]	—	—	0.2–1.0	0.2	0.2–1.0	—	[i]	
Paper and pulp, fine	10	5	—	200	100[j]	75	—	—	—	0.1	0.05	—	—	—	Soluble SiO_2, 20; free CO_2, 10; residual Cl_2, 2
Paper, groundwood	50[k]	30	—	500	200	150	—	75	—	0.3	0.1	—	—	—	Soluble SiO_2, 50; free CO_2, 10
Paper, kraft, bleached	40	25	—	300	100	75	—	200	—	0.2	0.1	—	—	—	Soluble SiO_2, 50; free CO_2, 10
Paper, kraft, unbleached	100	100	—	500	200	150	—	200	—	1.0	0.5	—	—	—	Soluble SiO_2, 100; free CO_2, 10
Paper, soda, and sulphate pulps	25[k]	5	—	250	100[l]	75	—	75	—	0.1	0.05	—	—	—	Soluble SiO_2, 20; free CO_2, 10
Rayon and acetate fibre pulp production	5	5	—	100[l]	8	50–75	—	—	—	0.05	0.03	0.05	—	—	Al_2O_3, 8; Si, 25; Cu, 5
Rayon manufacture	0.3	—	—	—	55	—	7.8–8.3	—	—	0.0	0.0	0.0	—	—	

Sugar	—	—	—	Low	Low	—	—	20	20	0.1	—	—	—	—	Ca, 20; Mg, 10; bicarbonate, as $CaCO_3$, 100; sterile, no saprophytic organisms
Tanning	20	10–100	—	—	50–500	130	6.0–8.0	—	—	0.1–0.2	0.1–0.2	0.2	—	—	Bicarbonate hardness, low; COD, 8; heavy metals, none; Ca, 10; Mg, 5; bicarbonate, as $CaCO_3$, 200
Textile	0.3–25	0–70	—	—	0–50	—	—	100	100	0.1–1.0	0.05–1.0	0.2–1.0	—	—	

[a] Some calcium is necessary for yeast action. Too much hardness retards fermentation, but too little softens the gluten to produce soggy bread. Water of zero hardness is required for some cakes and crackers
[b] Not more than 300 mg/l of any one substance
[c] $CaSO_4$ less than 100 to 500 mg/l; $MgSO_4$ less than 50 to 200 mg/l
[d] For dark beer alkalinity as $CaCO_3$ may be 80 to 150 mg/l
[e] Range, lower to upper limits
[f] Total solids
[g] Tolerance limit depends on annual average of maximum daily air temperatures for a minimum of 5 years
[h] For legumes, 25 to 75; for fruits and vegetables, 100 to 200; for peas, 200 to 400
[i] 1.5 mg/l of fluoride has been reported to cause embrittlement and cracking of ice
[j] Calcium hardness, 50
[k] No gritty material
[l] Calcium hardness, 50; magnesium hardness, 50

TABLE 5.7a. Australian standards for livestock water (mg/l)

Livestock	*Maximum total dissolved salts (TDS)*	
	Western Australia	*Victoria*
Poultry	2 900	3 500
Pigs	4 300	4 500
Horses	6 400	6 000
Dairy cows	7 100	6 000
Beef cattle	10 000	7 000
Sheep on dry feed	13 000	14 000
Ewes with lambs	10 000	4 500
Ewes in milk	10 000	6 000

TABLE 5.7b. Water quality criteria for livestock (after UNESCO, 1972[8])

Substance	*Upper limit*	*Substance*	*Upper limit*
Total dissolved salts (TDS) (mg/l)	10 000[1]	*Hazardous trace elements* (mg/l)	
		As	0.05
(Radionuclides (pCi/l)		Cd	0.01
Sc^{90}	10	Cr	0.05
Ra^{225}	3	F	2.40
Activity	1 000	Pb	0.05
		Se	0.01
Chemical elements (mg/l)			
$MgSO_4$	2 050		
NaCl	2 000	*Organic substances*	
SO_4	1 000	Algae (water bloom)	—[2]

1. Depending upon animal species and ionic compositions of the water
2. Avoid abnormally heavy growth of blue-green algae. Parasites and pathogens conform to epidemiological evidence. Dissolved organic substances

is used for irrigation since they are poorly drained which means that the capacity for removing excess salts by leaching is reduced. On the other hand, if a soil is well drained crops may be grown on it with the application of generous amounts of saline water.

In addition to the potential dangers due to high salinity, a sodium hazard sometimes exists in that sodium in irrigation water can bring about a reduction in soil permeability and cause the soil to harden. Both effects are attributable to cation exchange of calcium and magnesium ions by sodium ions on clay minerals and colloids. Sodium content can be expressed in terms of per cent sodium as follows

$$\%\mathrm{Na} = \frac{(\mathrm{Na}+\mathrm{K})100}{\mathrm{Ca}+\mathrm{Mg}+\mathrm{Na}+\mathrm{K}} \tag{5.3}$$

where all ionic concentrations are expressed in milli-equivalents/l. The extent of replacement of calcium and magnesium ions by sodium ions, that is, the amount of sodium adsorbed by a soil, can be estimated from the sodium absorption ratio (SAR), which is defined as

$$SAR = \frac{\mathrm{Na}}{\sqrt{(\mathrm{Ca}+\mathrm{Mg})/2}} \tag{5.4}$$

TABLE 5.8. Relative tolerance of crops to salt concentrations expressed in terms of specific electrical conductance (after US Dept. of Agriculture[23])

Crop division	*Low salt tolerance*	*Medium salt tolerance*	*High salt tolerance*
Fruit crops	Avocado Lemon Strawberry Peach Apricot Almond Plum Prune Grapefruit Orange Apple Pear	Cantaloupe Date Olive Fig Pomegranate	Date palm
Vegetable crops*	3000 μS/cm Green beans Celery Radish 4000 μS/cm	4000 μS/cm Cucumber Squash Peas Onion Carrot Potatoes Sweet corn Lettuce Cauliflower Bell pepper Cabbage Broccoli Tomato 10 000 μS/cm	10 000 μS/cm Spinach Asparagus Kale Gardet beet 12 000 μS/cm
Forage crops*	2000 μS/cm Burnet Ladino clover Red clover Alsike clover Meadow foxtail White Dutch clover 4000 μS/cm	4000 μS/cm Sickle milkvetch Sour clover Cicer milkvetch Tall meadow oatgrass Smooth brome Big trefoil Reed canary Meadow fescue Blue grama Orchard grass Oats (hay) Wheat (hay) Rye (hay) Tall fescue Alfalfa Hubam clover Sudan grass Dallis grass Strawberry clover Mountain brome Perennial rye grass Yellow sweet clover White sweet clover 12 000 μS/cm	12 000 μS/cm Bird's-foot trefoil Barley (hay) Western wheat grass Canada wild rye Rescue grass Rhodes grass Bermuda grass Nuttall alkali grass Salt grass Alkali sacaton 18 000 μS/cm

TABLE 5.8. *continued*

Crop division	Low salt tolerance	Medium salt tolerance	High salt tolerance
Field crops*	4000 μS/cm Field beans	6000 μS/cm Castorbeans Sunflower Flax Corn (field) Sorghum (grain) Rice Oats (grain) Wheat (grain) Rye (grain) 10 000 μS/cm	10 000 μS/cm Cotton Rape Sugar beet Barley (grain) 16 000 μS/cm

* Specific electrical conductance values represent salinity levels at which a 50 per cent decrease in yield may be expected as compared to yield on non-saline soil under comparable growing conditions. Concentrations refer to soil water. Specific electrical conductance is measured in microsiemens/cm (μS/cm) which is equivalent to micromhos/cm

where, again, the concentrations are expressed in milli-equivalents/l. The sodium hazard as defined in terms of the SAR is shown in *Figure 5.1*.

5.7 Investigation of groundwater quality

If wells exist in the area under investigation, then the level of the water in them and its temperature should be recorded. Samples of water taken from the wells permits determination of its pH value, the hardness and the concentration of major ions. Samples of water for analysis can also be taken from springs. Again the temperature of the water can be recorded.

Figure 5.1 Classification of quality criteria for irrigation water (after Richards, L. A. (ed.), *Diagnosis and Improvement of Saline and Alkali Soils*, Agricultural Handbook 60, US Dept. Agric., Washington DC (1954))

* Description of sodium hazard

S_1 Low sodium water can be used for irrigation on almost all soils with little danger of the development of harmful levels of exchangeable sodium. However, sodium sensitive crops such as stonefruit trees and avacado may accumulate injurious concentrations of sodium.

S_2 Medium sodium water will present an appreciable sodium hazard in fine textured soils having high cation exchange capacity, especially under low leaching conditions, unless gypsum is present in the soil. This water may be used on coarse-textured or organic soils with good permeability.

S_3 High sodium water may prodoce harmful levels of exchangeable sodium in most soils and will require special soil management—good drainage, high leaching and organic matter additions. Gypsiferous soils may not develop harmful levels of exchangeable sodium from such water. Chemical amendments may be required for replacement of exchangeable sodium, except that amendments may not be feasible with waters of very high salinity.

S_4 Very high sodium water is generally unsatisfactory for irrigation purposes except at low and perhaps medium salinity, where the dissolving of calcium from the soil, or the use of gypsum or other additives may make the use of these waters feasible.

** Description of salinity hazard

C_1 Low salinity water—can be used for irrigation with most crops on most soils with little likelihood

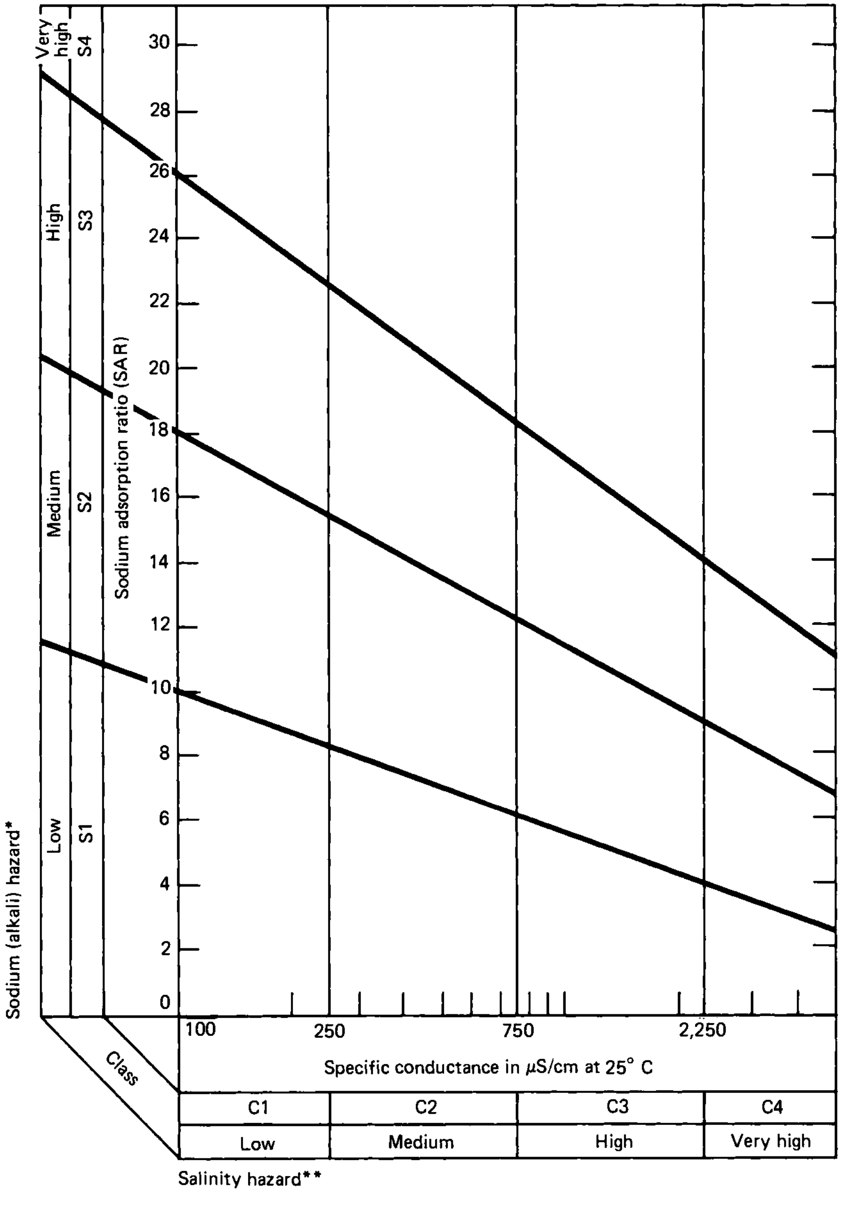

Figure 5.1

that a salinity problem will develop. Some leaching is required, but this occurs under normal irrigation practices except in soils of extremely low permeability.

C_2 Medium salinity water—can be used if a moderate amount of leaching occurs. Plants with moderate salt tolerance can be grown in most instances without special practices for salinity control.

C_3 High salinity water—cannot be used on soils with restricted drainage, special management for salinity control may be required and plants with good salt tolerance should be selected.

C_4 Very high salinity water—is not suitable for irrigation under ordinary conditions but may be used occasionally under very special circumstances. The soil must be permeable; drainage must be adequate; irrigation water must be applied in excess to provide considerable leaching and very salt tolerant crops should be selected.

Electrical techniques provide an indirect method of investigating groundwater quality. For example, the measurement of resistivity provides a means of detecting and outlining variations in groundwater quality where such quality variations are accompanied by changes in the electrical resistivity or conductivity of the groundwater[24]. For example, the resistivity method can be used in sandstones in which groundwater flow is almost entirely pore-controlled to locate salt water–fresh water interfaces such as occur in the Bunter Sandstone of the Cheshire Basin. The results of such surveys are useful when planning the test drilling and water sampling programmes which are required.

The ionic content of the groundwater in a drillhole can be monitored by measuring the resistivity of the fluid, at a short electrode spacing, or by determining its electrical conductivity. The latter is a function of the number and kind of ions present, their relative charge and their mobility[6,25]. Hence such drillhole logs are of particular importance where adverse mineralization or saline conditions exist.

Brown[26] used resistivity logging to help determine the quality of groundwater in a thick succession which consisted mainly of sands and silts. He obtained an approximation of the dissolved solids and chloride content by using the expression

$$\rho_w = \rho_0 / F \tag{5.5}$$

where ρ_w is the resistivity of the interstitial water in a porous rock, ρ_0 is the resistivity of the saturated rock and F is the formation factor (see Chapter 3), and a number of graphs. In order to find F in the above expression, the porosity, n, was obtained from a gamma–gamma log. This enabled the formation factor to be derived from *Figure 5.2*. The resistivity of the pore water depends upon the type of salt present in solution. For example, if sodium bicarbonate is present in solution, then the resistivity is 1.75 times

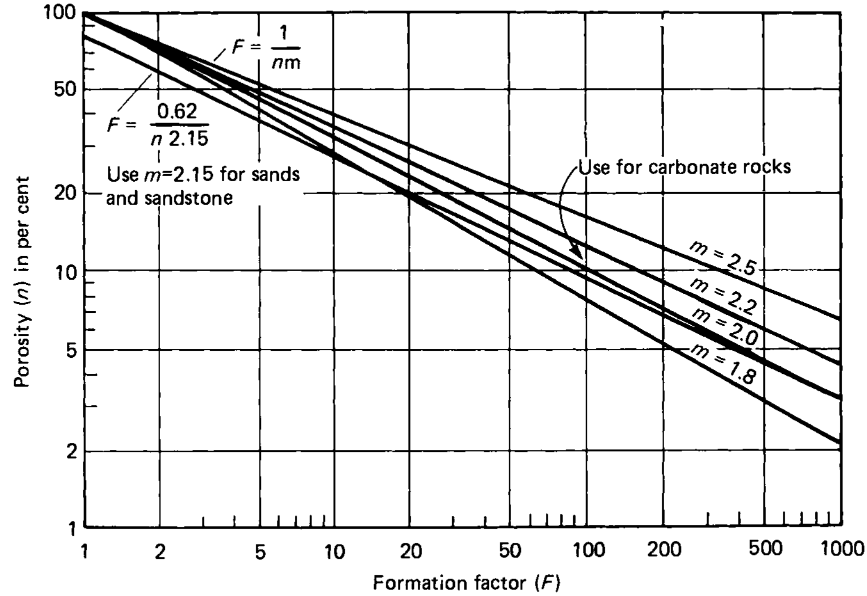

Figure 5.2 Relation of formation factor, *F*, to porosity, *n* (after Schlumberger Well Surveying Corporation)

greater than when sodium chloride is in solution, the sodium concentration being identical.

According to Patten and Bennett[27], the quantitative interpretation of the spontaneous potential log relates the magnitude and direction of the spontaneous potential deflection, ΔSP, from a shale base line to the difference in sodium chloride ion concentration between the fluids in the formation and in the drillhole. This analysis led to the relationship

$$\Delta SP = -71 \log_{10}(a_w/a_{dh}) \tag{5.6}$$

where a_w and a_{dh} are the mean ionic activities of sodium chloride in the formation water and in the drillhole fluid respectively[28]. The numerical coefficient (-71) is a constant for a specific temperature of 25°C (the standard temperature for the specific conductance measurements used). Because the ionic activity of a solution is inversely proportional to its resistivity, for practical purposes, the ratio a_w/a_{dh} may be replaced by ρ_{dh}/ρ_w[29]. Similarly as the resistivity of a solution is the reciprocal of its specific electrical conductance, ρ_{dh}/ρ_w may be replaced by C_w/C_{dh}. Equation (5.6), therefore, may be rewritten as

$$SP = -71 \log_{10}(C_w/C_{dh}) \tag{5.7}$$

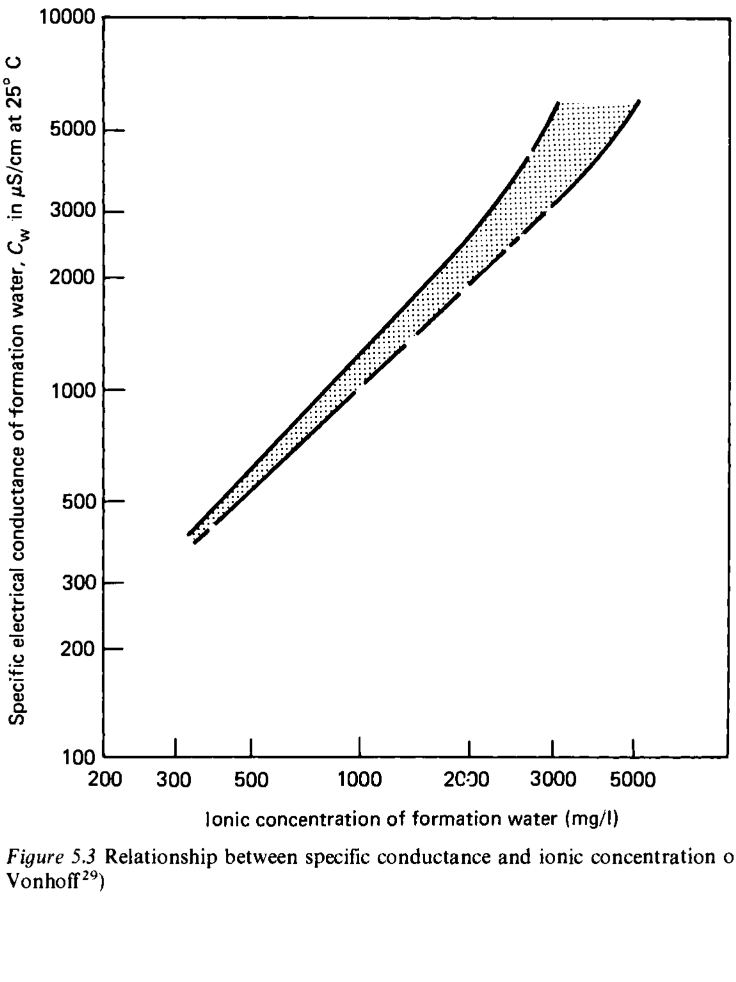

Figure 5.3 Relationship between specific conductance and ionic concentration of pore water (after Vonhoff[29])

in which C_w and C_{dh} are the specific electrical conductances for the pore water and the drillhole fluid respectively.

Hence, the quality of groundwater can be estimated from the spontaneous potential deflection on an electric log, provided the specific conductance of the drilling mud is less than the specific conductance of the pore water. Indeed, the method tends to give better results as the salinity of the pore water increases. *Figure 5.3* can be used to estimate the ionic concentration of a pore water from its specific conductance even if its ionic composition is unknown. An empirical method of interpreting *SP* logs in order to determine the quality of pore water also has been provided by Brown[26].

Ineson and Gray[30] carried out a number of investigations in which resistivity and electrical conductivity methods were used to recognize the chemical characteristics of the groundwater. In particular, these methods were used to study the movement of saline interfaces associated with saline infiltration into an aquifer, the movement of salt water between a tidal river and a well and the movement of groundwater between different aquifers in the same well. In the latter situation, movement of groundwater can be investigated by inducing artificial changes in the natural profile of electrical conductivity in the well. First, the natural profile is determined over either the entire depth of the well or drillhole or at specific depths of interest. Then the electrical conductivity is artificially increased by the introduction of a saline solution at a particular depth, thereby increasing the electrical conductivity of the fluid. As soon as the increased salinity has been recorded, electrical conductivity profiles are determinded at as frequent time intervals as possible. It is generally adequate to carry out these traverses only over the zone of increased salinity, provided that the upper and lower limits can be defined. When a pattern emerges, occasional determinations of the electrical conductivity profile should be made over the total depth of the hole.

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Chapter 6

Well design and construction

6.1 Well design and construction

There is no simple answer to the question 'how is a well designed and constructed?' because no two wells are identical. *Figure 6.1* shows a typical water production drillhole. This consists basically of a screen through which water passes into the well and a pump casing that connects the screen to the surface and which houses the pump or pump intake and the delivery pipe. The size of these components depends upon many factors such as the geology of the area, the thickness and type of strata encountered, the position of the water table and constraints upon construction such as the availability of material. Thus, any well design should reflect local conditions and be based upon common sense and the use of tried and trusted procedures.

There is one other basic question that must be answered before designing a water well, and that is 'why is the well being constructed and what precisely is required?' In England, most groundwater abstraction schemes are managed by nine regional water authorities who are charged with the responsibility of supplying water to the public. Consequently their usual aim is to obtain a large amount of water from the resource as cheaply as possible. In other parts of the world, however, a borehole may be constructed to supply only one homestead. Obviously these two differing situations would call for very different sizes and types of well, so it is important that the actual requirement is kept firmly in mind while the well is being designed.

Having established what is required, there are still various ways to approach the problem. For example, is water so scarce that the objective is to obtain the maximum yield regardless of cost, or to obtain as much as possible at a reasonable cost? The answer, of course, depends upon the circumstances. Nonetheless, the price of water from a well depends on the cost of its construction (see Section 1.6, pp. 6–9) and its running costs during its useful life. Sometimes it may be worth spending extra money on the construction of the well if this is going to significantly reduce the recurring costs of pumping and maintenance over, say, the following 25 years. Stoner *et al.*[1] described a method of determining the 'least cost solution' for a chosen design parameter, such as drawdown, screen length, or well diameter.

Whatever the requirements of an individual well, it is true to say that a major water production drillhole is an expensive venture that should not be undertaken without prior investigation. In particular, geological and hydrological data are required[2] to produce the detailed design of a suitable drillhole, the most important of which are

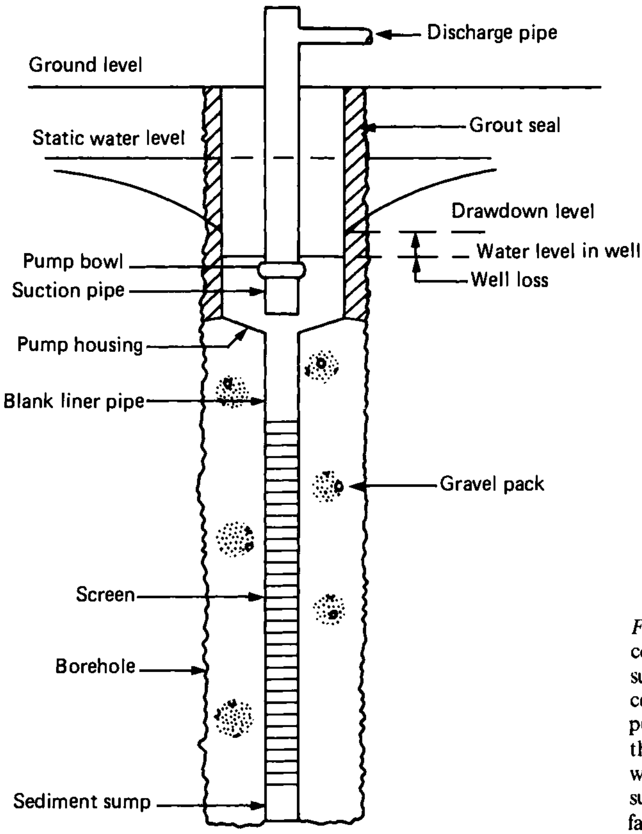


Figure 6.1 Schematic configuration of a typical water supply well. The basic components of a well are the pump casing and the screen, through which water enters the well. The screen may be surrounded by a gravel pack to facilitate flow near the well

1. The thickness, character, and sequence of strata to be drilled through.
2. The thickness, character (particularly permeability and storativity) and sequence of the aquifer(s).
3. The degree of confinement of the aquifer(s).
4. The permeability of the enclosing strata.
5. Water levels and water level trends in the aquifer(s).
6. The quality of the water.
7. The yield required from the well.
8. The probable repercussions of reduced water levels in the vicinity of the well.
9. The case histories of any wells previously constructed in the area.

In most cases all this information will not be available, so before commencing the construction of a large diameter drillhole some sort of preliminary investigation should be carried out (see Chapter 3). This often takes the form of a small diameter pilot hole, which can be pump tested using, for example, an air lift technique. It must be stressed that a pilot hole is an essential pre-requisite of well construction. The pilot hole may even pay for itself as a result of improved well design, or it may prevent a full-size water production well being drilled in a formation which is unsuitable for the purpose.

The procedure for designing a water well is summarized in *Figure 6.2*, which gives an indication of the sequence in which the various parameters influencing the design may be considered. The various steps shown in the figure are discussed below.

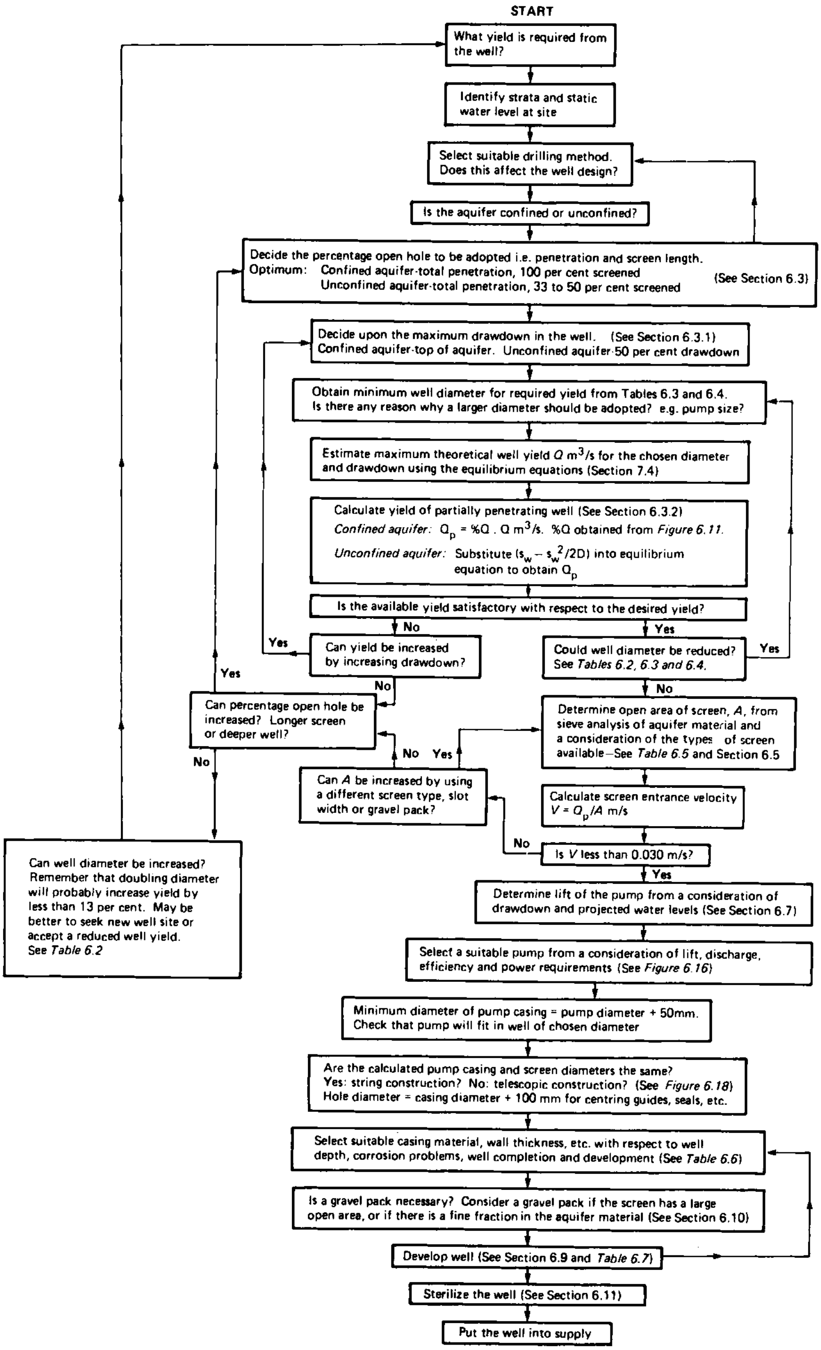


Figure 6.2 Flow diagram showing the possible steps in the design and construction of a water well

6.2 Well drilling methods

The method used for sinking a well for water supply depends upon the geological conditions at the site. The rate of advance in particular is governed by the hardness of the ground penetrated and progress may be as little as 1 m/day in hard rock. As a result a well extending to 300 m depth is an expensive proposition.

The method of drilling or boring may affect the design of a well. For instance, the percussion technique (or as it is often called, the cable-tool method) may necessitate a telescopic design, where the well diameter decreases with depth, because of the need to reduce friction on the tool string during boring. Rotary drilling methods, on the other hand, allow the construction of wells of a constant diameter, but the drilling fluid used during construction may pollute or block off the aquifer. Consequently the drilling technique must be considered as part of the well design process.

There are far too many variables involved in constructing a water well for any one method or procedure to be universally applicable. Instead there are a variety of techniques which are used according to local conditions and requirements. For example, in Britain, because groundwater is mainly the responsibility of the regional water authorities, there are relatively few new wells drilled each year, and the average size of such a hole is 600 mm diameter and 90 m depth. In America, on the other hand, individual dwellings frequently have their own wells. Indeed in 1977 some 730 000 were sunk, usually of 100 to 150 mm diameter.

Most water wells are constructed using one of three basic techniques, namely, percussion (or cable-tool), direct circulation hydraulic rotary and reverse circulation hydraulic rotary drilling. Down-the-hole-hammer drilling is a relatively recent innovation that, under suitable conditions, may become more widely used in the future for well drilling. These techniques are described below, but it should be appreciated that there are other methods, some, such as turbine drilling[3,4], that are still under development and which may eventually replace the older, more established techniques[3,4,5].

6.2.1 The percussion (or cable-tool) method

The great advantage of the percussion or cable-tool method is its relative simplicity. Drilling is accomplished by the alternate lifting and dropping of a string of tools, which, from top to bottom, consist of a rope socket, a set of jars, a drill stem and a drill bit (*Figure 6.3*). The drill bit has a relatively sharp chisel edge which breaks the rock by impact, and may vary between 1 to 3 m in length and weigh up to 1400 kg. The function of the drill stem is to add weight to the bit and, by virtue of its length, keep the string vertical. Drill stems vary from 2 to 15 m in length, from 60 to 150 mm in diameter and weigh between 450 to 1400 kg. An important element of the tool string is the jars, although during most of the drilling they are not brought into action. The jars consist of a pair of sliding connecting links (like a chain), which are included for the purpose of giving a jerking or jarring action to the string should it become stuck in the hole. During normal operations the jars are in tension and are partially or fully extended, but when the tools become stuck the line can be slackened to allow the links to extend (by about 150 to 500 mm), after which a pull on the cable causes the jars to impart an upward blow to the tools. Alternatively, the jars may be kept in tension and a coaxial weight (a 'jar bumper') run down the drilling cable, so that the impact of the weight on the top of the rope socket momentarily closes the jars which then snap back smartly, giving an upward thrust to the string.

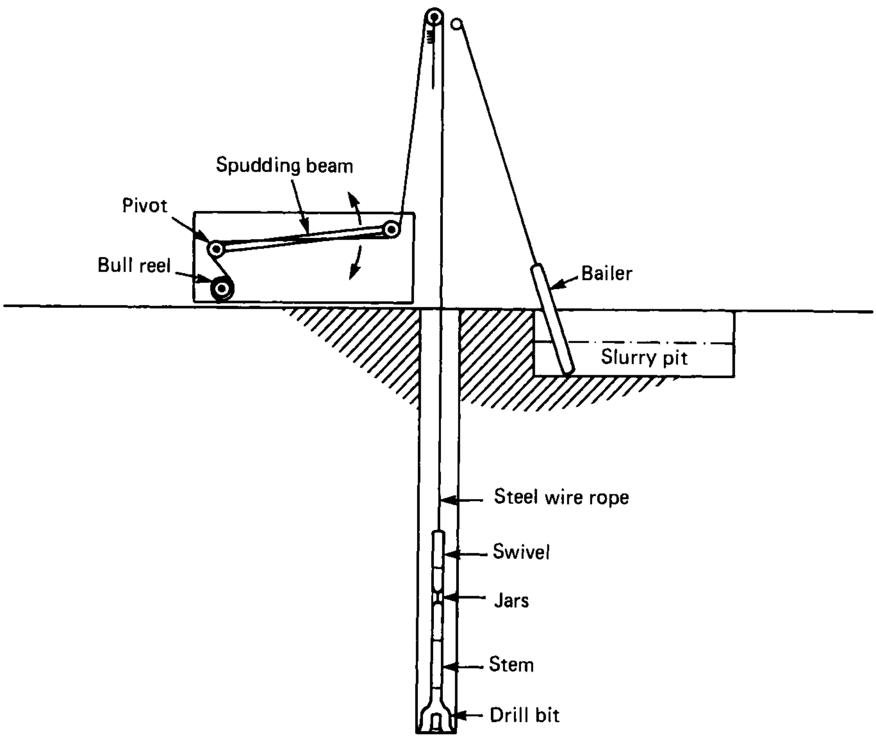


Figure 6.3 Percussion or cable tool rig (after Cruse[3])

The purpose of the swivel rope socket is to attach the wire drilling rope to the string of tools and to allow the string to rotate so that the bit impacts in a different position each time, thus ensuring a circular drillhole. The rotation of the string is achieved by using a left-hand lay steel wire rope that unwinds under the weight of the string and which counters the tendency of the right-handed tool joints to unscrew.

On most cable tool rigs, the cable from the rope socket runs over a rubber cushioned sheave at the top of the tower, down through a sheave on the spudding beam, and is then wound around the bull reel. The spudding beam imparts a reciprocation motion to the cable and is thus responsible for the drilling action of the tool string. The tools typically make 40 to 80 strokes per minute which range from 0.4 to 1.2 m in length. Additional line is let out as necessary so that the bit always strikes the bottom of the hole. Thus, the driller has two important adjustments to make, that is, the number of blows per minute, which must be set to avoid snatching of the drilling line and the length of the line which must be just right to achieve maximum penetration rates. The driller literally makes these adjustments by feel through the drilling cable. Occasionally water will be added to the hole to move the cuttings away from the bit face, reduce friction and make a paste of the rock fragments so that they are easier to extract. When the driller feels that progress is being slowed by the cuttings in the hole, the drill string is removed and a bailer is run down the hole. This is usually necessary at intervals of about 1.5 to 2.0 m. The bailer is essentially a section of pipe with a valve at the bottom, which permits cuttings to enter but prevents them from escaping. The bailer is then hauled to the surface and emptied. Bailers vary in length from 2 to 10 m, and in capacity from 10 to 45 l.

Cable-tool drilling is a very versatile method that can be used almost anywhere. Such rigs can even be horse or man powered if necessary. To get the best performance from cable-tool rig, a good and experienced driller is required, although the penetration rate may still be slow compared to some of the modern rotary and percussion techniques. This is offset to some extent by the fact that a cable-tool rig is reliable, robust, costs about 50 to 60 per cent of a rotary rig of equivalent capacity and is cheap to run.

The cable-tool method is suitable for drilling holes of 75 to 600 mm diameter in both consolidated and unconsolidated strata (see *Table 6.1*). However, in unconsolidated ground slumping and caving around the bit may retard progress, but it is still possible to construct a hole by employing a bail and drive technique. This consists of using the bailer to remove the loose material (a bit is probably not needed) while the casing is maintained to the full depth of the hole to support the formation.

6.2.2 Direct circulation hydraulic rotary methods

The direct circulation rotary drilling technique has been widely adopted for drilling water wells. Drilling using this system is accomplished using a string of tools comprising, from top to bottom, drill pipes, a drill collar and a drill bit (*Figure 6.4*). Rotary bits are of the roller cutter type in hard rock or the plain drag bit type in soft strata[6,7,8]. The bit is rotated at speeds of 30 to 300 rev/min depending upon diameter and type of strata, while a downward force of 250 to 2750 kg/25 mm of diameter is applied[3]. Under these conditions the teeth of a suitable roller cutter bit will crush hard rock and tear soft rock, while the hard faced blades of a drag bit behave somewhat like an auger.

The drill collar consists of extra heavy pipes fitted above the drill bit. The purpose of the collar is to add weight to the bit and to prevent bending of the drill stem near the bit, thus keeping the drillhole vertical. The collar is attached to the surface via drill pipes which are of between 3 and 10 m in length. The uppermost length of pipe is termed the kelly, through which the rotational drive is transmitted. The drill pipes also convey drilling fluid to the bit, from which it is extruded through openings in the bit. The fluid, being pumped from the surface and thus under pressure, then flows back to the surface by way of the annulus formed between the outside of the drill collar and the well wall or casing. The fluid acts as a lubricant, cools the bit, removes cutting from the bottom of the hole and carries them to the surface where they can be collected for inspection and provides support to the sides of an uncased hole, thereby preventing caving.

One of the key features of the hydraulic rotary technique is the drilling fluid[3,4,6,8]. At one time this was nearly always a clay or bentonite suspension, but in recent years organic polymers have become increasingly popular. Drilling fluid is an expensive substance, so losses during circulation should be kept as small as possible to minimize costs and also to afford the maximum support to the well. Collapse of a well during construction is often the result of a sudden loss of fluid into a permeable or porous formation. One of the functions of the drilling fluid should be to prevent such a loss. Bentonite achieves this by invading the formation and so blocking the voids. In fine-grained material only the water phase of the suspension enters the formation, while the bentonite forms a cake on the inside of the hole. This process, known as mudding off, prevents further fluid losses and, in conjunction with hydrostatic pressure, helps support the hole. In coarse-grained material, however, both the water and the clay particles invade the formation and they may prove difficult or impossible to remove despite pumping or the use of dispersant chemicals. This may mean that the very formation that the well was constructed to exploit has been polluted, mudded off, or

TABLE 6.1. Comparison of common water well drilling techniques, and range of suitability

Drilling method	*Range of diameter and depth possible*	*Formations which suit the method*	*Formations which do not suit the method*	*Typical penetration rates*	*Advantages of the method*	*Disadvantages of the method*
Percussion (or cable tool)	75–600 mm. Possibly up to 900 mm. Depth from 100 to 600 m depending on rig weight and hole diameter. Record depth 3397 m	Consolidated rock, clay. Unconsolidated material, sands and gravels if cased	Running sands may cause problems. Hole must be cased to full depth to prevent caving	Hard rock 1–3 m/day. Soft sandstone and sandy clay 15–30 m/day. Sticky clay and shale 5–15 m/day. Loose fine sand or quicksand 3–6 m/day	Relative simplicity. Low initial equipment cost. Low daily operating cost. Low rig set up time. Suitable for almost all conditions. Drilling rates comparable to rotary methods in some types of rock at shallow depths. Good cutting samples with no contamination of rock fragments. Easy identification of aquifers and easy water quality sampling. Low water requirement. Minimum contamination of aquifers	Slow penetration in hard rock. Limited economical depth. Lack of control over water flow from penetrated formations. Lack of control over borehole stability. Unconsolidated formations have to be supported with multiple columns of casing. Recovery of casing may be difficult or impossible if well aborted. Frequent drill line failures. Lack of experienced personnel—an art not a science. Usually necessitates a telescopic design to reduce friction during drilling
Direct circulation rotary drilling using mud as drilling fluid	75–1500 mm. Depth 00's to 000's of metres	Consolidated soft and hard rock formations. Unconsolidated material	Difficulty encountered in formations which are highly permeable, cavernous, or fractured due to fluid loss causing possible collapse of hole. Boulders prove difficult to drill through	Consolidated rock 10–15 m/day. Soft unconsolidated sediments 100–150 m/day	Borehole can be drilled to great depth at quite good penetration rates in most materials. Unconsolidated formations are supported without the use of casing	Relatively high cost of equipment—perhaps twice that of cable tool rig. Risk of losing all mud into fissures. Risk of mudding-off or contaminating aquifers. Considerable quantities of make-up water required. Cuttings recovery not good, and fragments are contaminated by mud. Difficulty in recognizing aquifers. Boulders difficult to drill through
Reverse circulation rotary drilling using water as drilling fluid	400–1800 mm. Depth 120–350 m	Loose formations such as soft unconsolidated sand and silt. Soft clay. Also stiff clay and soft rock, but at reduced efficiency	A suction dredging technique rather than a true drilling method, so not effective on hard rock or boulders. Water losses may cause collapse as in rotary drilling above. Water table should be 3 m or more below ground	Rates of 12 m/h are quite common. Occasionally up to 0.6 m/min. Under favourable conditions	Very rapid drilling possible even at large diameters, particularly in sands and gravel. Rapid production of very good samples. No risk of mudding-off or contaminating aquifer if circulating water kept clean. Casing unnecessary except at surface to control drilling fluid	High cost of equipment. Large quantities of make-up water required—perhaps 9–70 m^3/h. Essentially a suction-dredging technique so material larger than the drill bit openings or drill stem (such as boulders, cobbles) may have to be fished out
'Down the hole' hammer drill using air	50–375 mm. Possibly up to 750 mm. Depth depends on conditions	Hard dry rock	Will not operate in unconsolidated materials or clays. May be defected by water	Hard dry rock—3 m/h or better	Fast penetration in hard rock. Drilling mud not required. Aquifer not mudded off or contaminated. Good return of uncontaminated cuttings	Inability to support hole-mud back-up may be required. Efficiency falls with depth below water

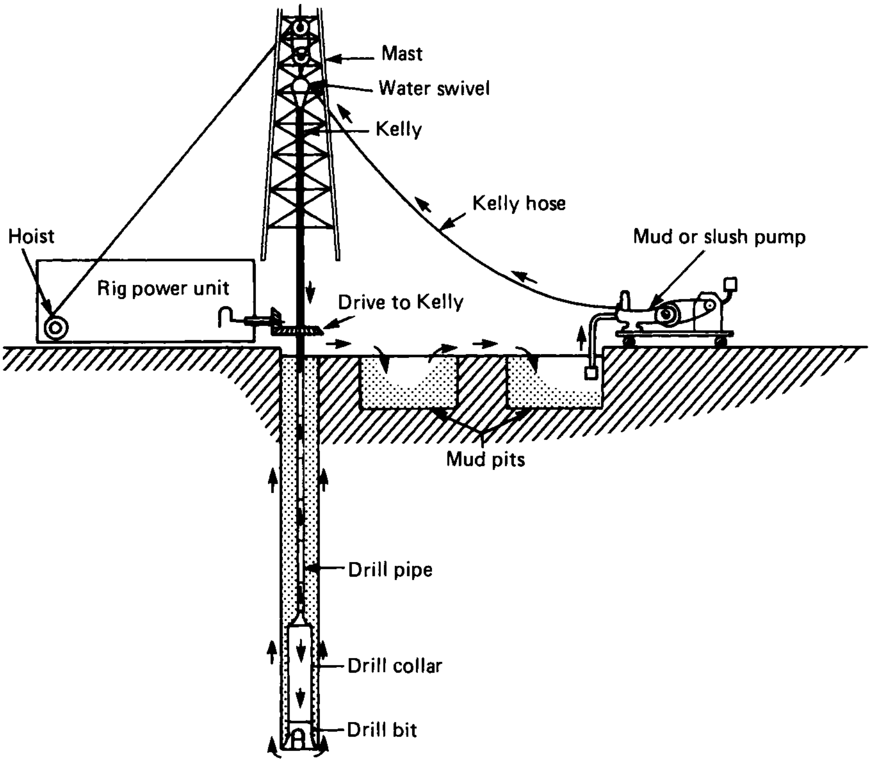


Figure 6.4 Normal direct circulation rotary drill (after Cruse[3])

rendered inviable as a result of the construction process. On the other hand, water-based organic polymers have similar properties to bentonite, but possess the additional advantage that they break down into a water substance that can be easily removed from the formation. Hence, the threat to an aquifer is eliminated. Organic polymers have a working life of three days or more, although some retain their properties indefinitely, that is, until broken down by a heavy dose of chlorine[3].

Although hydraulic rotary drilling is a widely used technique, there are some inherent difficulties involved with its use under certain conditions. If the formation to be drilled consists of highly permeable unconsolidated material or heavily jointed rock, the loss of drilling fluid may prove very expensive. In the former case in particular, the well would have to be cased to prevent the possibility of a sudden loss of fluid causing collapse[4]. A record should be kept during construction of the losses of drilling fluid with depth since this gives an indication of where the most permeable formations are located.

The direct circulation rotary drilling technique can be used to construct wells of up to 1500 mm diameter in most types of ground (see *Table 6.1*). However, problems may be encountered in heavily fissured strata and ground which contains boulders, as the latter may prove very difficult to drill through. In addition, rotary drilling is technologically more complex than the cable-tool method, which may not make it suitable for areas where skilled labour and sophisticated materials (such as bentonite and organic polymers) are in short supply or unavailable. In some parts of the world there may not even be an adequate source of make-up water for the drilling fluid.

6.2.3 Reverse circulation hydraulic rotary methods

As the name suggests, reverse circulation rotary drilling has a reversed flow of drilling fluid compared to the system used in the direct method (*Figure 6.5*). Consequently the drilling fluid and cuttings move upwards inside the drill pipe, through the suction end of the pump and then into a settling or mud pit. The fluid returns to the drillhole by gravity flow, moving down the annular space around the drill pipe to the bottom of the hole, picking up cuttings and re-entering the drill pipe through openings in the bit. The reverse circulation method is primarily a suction dredging technique, with the drilling action being of secondary importance. This means that only material and cuttings smaller than the openings in the drill bit can be removed from the hole, so if cobbles or boulders are encountered, progress may be stopped unless they can be fished out.

The drilling fluid used with this method is water, usually without any additives. This means that the hole is easier to clean and develop, but the problems associated with fluid loss are increased. Water is lost into all the permeable formations that are encountered, which means that while an indication of the most permeable strata is obtained, considerable quantities of make-up water are required (perhaps 9 to 70 m^3/h). In addition, to prevent caving of the hole the fluid level must be kept at least 3 m above the water table to ensure an adequate hydrostatic pressure to support the drillhole. In some instances the well casing may have to be extended above the ground surface to achieve this. However, the presence of a high static water level or the lack of an adequate water supply may rule out the possibility of using this method.

The principal advantage of this technique is that it provides an inexpensive method of drilling holes of up to 1.5 m diameter, especially in sand, silt, or soft clay. The smallest diameter of hole that can be drilled using the reverse rotary method is approximately 0.45 m.

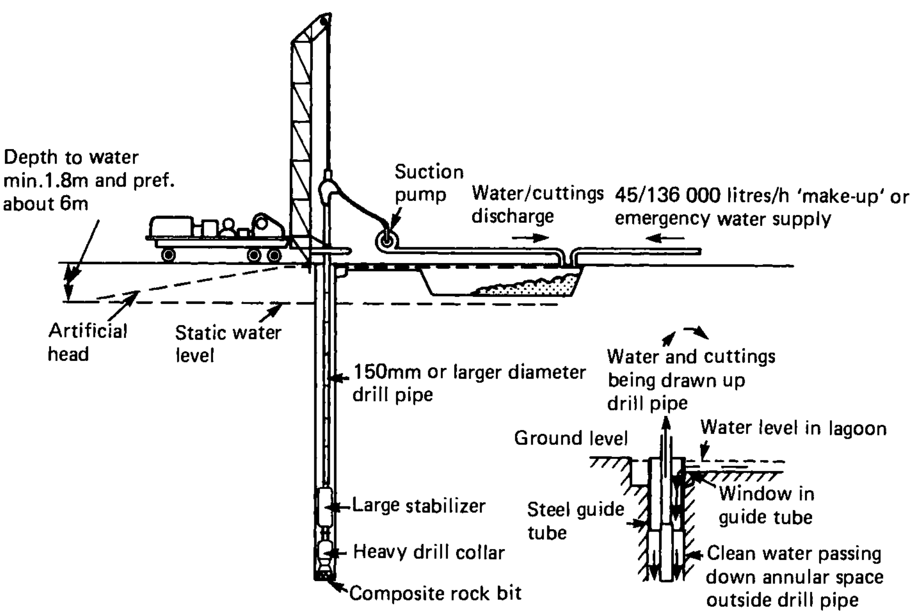


Figure 6.5 Reverse circulation rotary drill (after Cruse[3])

6.2.4 'Down-the-hole' hammer methods

The down-the-hole hammer drill is an air-actuated single piston device similar to the pneumatic road drill[3]. A tungsten carbide bit is attached to the hammer which in turn is attached to drill pipes leading to the surface (*Figure 6.6*). The circulation of drilling fluid, in this case air, is the same as in direct circulation rotary drilling.

The hammer is capable of delivering 500 to 1000 blows/min and should be rotated at between 20 to 50 rev/min to ensure even cutting. The high rate of strike gives the hammer drill an impressive rate of penetration in hard rock. Cruse[3] reported that under suitable conditions the hammer drill may be over thirty times quicker than a cable-tool rig. Unfortunately, a hammer drill is almost useless in unconsolidated ground or clays and small quantities of water can cause problems. Large quantities of water are ejected as in air lift pumping, and do not cause a problem until the depth becomes too great for the air compressor to operate the hammer and lift the water to the surface.

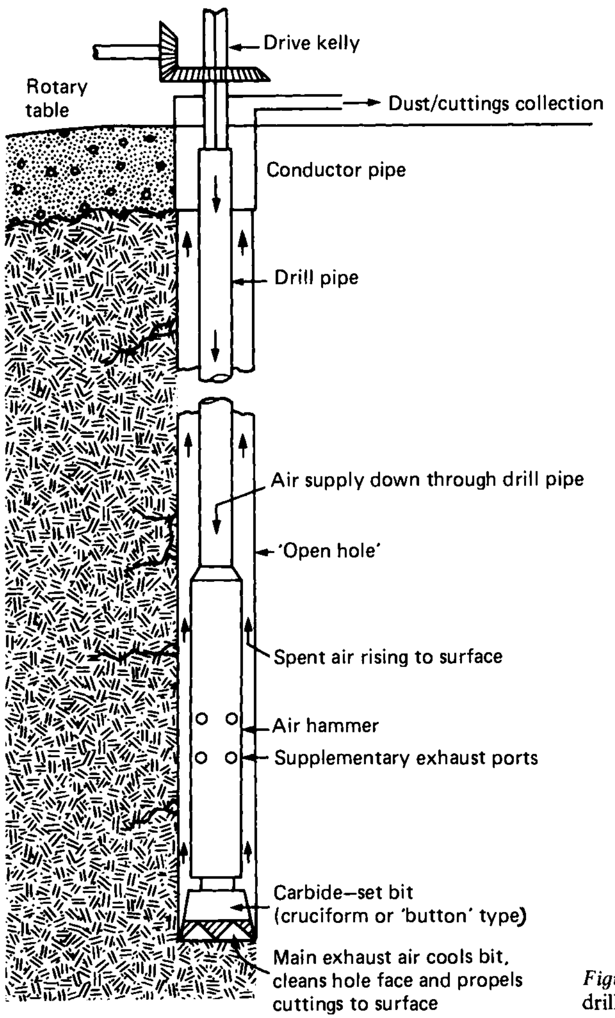


Figure 6.6 Down-the-hole hammer drill (after Cruse[3])

The hammer drill represents one of the more recent developments in drilling technology, and is suitable for drilling holes of between 50 to 375 mm diameter. Tools over 375 mm diameter are available, but are still somewhat unproven.

6.2.5 Pilot hole drilling methods

The objective of a pilot hole is to obtain data relating to the geology and hydrogeology of the site (see Chapter 3). It is, therefore, important that good samples of the penetrated strata are obtained. Additionally the hole should be quick, cheap and easy to construct since it will have to be abandoned should the site prove unsuitable for development. The diameter of pilot holes generally range between 100 and 165 mm.

Core drilling techniques are often used for the construction of pilot holes (see Chapter 3), although conventional rotary drilling techniques using direct water or air flush are also suitable. If water circulation is used, the hole may be self supporting and not need casing. This is desirable because temporary casings can sometimes prove to be difficult or impossible to extract. Unfortunately, the circulating water often picks up clay or other impurities so that rock cuttings and aquifers become contaminated and difficult to identify.

Percussion methods provide reasonable cuttings thereby allowing easy identification of aquifers. This drilling system is relatively slow, but its simplicity makes it useful in remote areas.

In hard dry rock, hammer drills are capable of sinking small diameter holes very rapidly. Using air flush, a good return of uncontaminated cuttings can be obtained.

To summarize, there is no universally acceptable method of drilling pilot holes. Each case must be decided individually, as outlined above. Sometimes more than one technique may be employed.

6.3 Well design for confined and unconfined aquifers

Perhaps the most important factor that influences well design is whether the aquifer is confined or unconfined, because this will determine the pattern of flow to the well and hence its design. If the aquifer is confined, flow to the well is horizontal (*Figure 6.7(a)*) provided that the well is screened throughout the entire thickness of the aquifer. In an unconfined aquifer, the flow to the well will never be horizontal, since water tends to be drawn down from the water table to the well, as shown in *Figure 6.8(a)*. Consequently the design of wells in confined and unconfined aquifers differs according to the anticipated flow conditions.

6.3.1 Percentage open hole of a well and maximum drawdown

The percentage of open hole depends upon two factors, first, the percentage penetration of the well into the aquifer (*Figures 6.7(b)* and *6.8(b)*) and secondly, the proportion of the well that is screened where it is in contact with the aquifer. If the entire thickness of the aquifer is screened, this is called a 100 per cent open hole. Anything less than a 100 per cent open hole as a result of either of the two factors just described is termed *partial penetration*. The effect of partial penetration is to reduce the efficiency of a well so that either the yield is reduced for a given drawdown, or the drawdown is increased for a given yield. This can be explained most clearly by comparing the pattern of flow to a totally and a partially penetrating well in a confined aquifer (*Figures 6.7(a)* and *(b)*). In

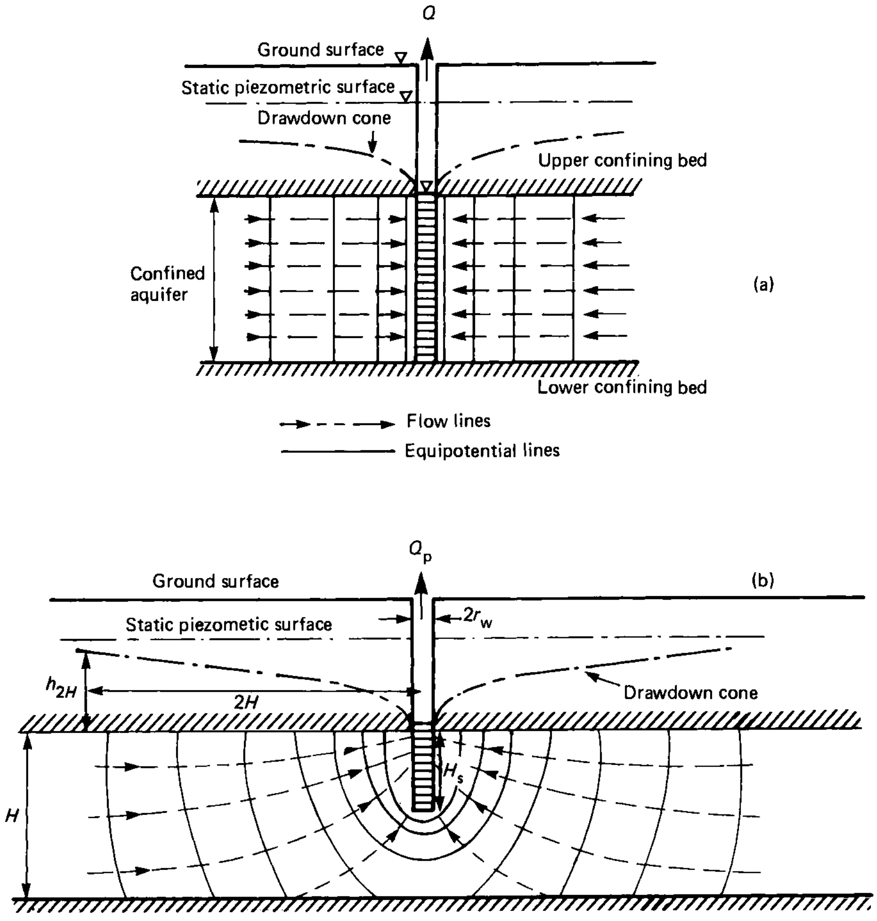


Figure 6.7 Flow to a well in a confined aquifer with (a) total penetration and (b) partial penetration (from Anon[2]). (a) Diagrammatic representation of flow to a fully penetrating and 100 per cent open hole in a confined aquifer. (b) Diagrammatic representation of flow to a 50 per cent penetrating and open hole in a confined aquifer

the case of the totally penetrating hole there is horizontal radial flow to the well, whereas with partial penetration water is drawn up into the well and the flow is no longer horizontal. The fact that the flow is no longer horizontal means that the flow path of a particle travelling to the well has been increased, which means that the head loss and therefore the drawdown also increases.

The vertical flow component has an added significance in that the vertical permeability of most aquifers is quite small in comparison with that in the horizontal plane. The effect of partial penetration is most pronounced near the well, while at a distance of twice the aquifer thickness from the well the difference in drawdown observed for total and partial penetration is negligible (*Figure 6.7(b)*) since the flow is almost horizontal. In an unconfined aquifer the flow to a well is never horizontal since water tends to be drawn down from the water table, as shown in *Figure 6.8(a)*. However, partial penetration causes water to be drawn up into the well (*Figure 6.8(b)*) so that the drawdown is increased and the efficiency reduced. Again the effect of partial

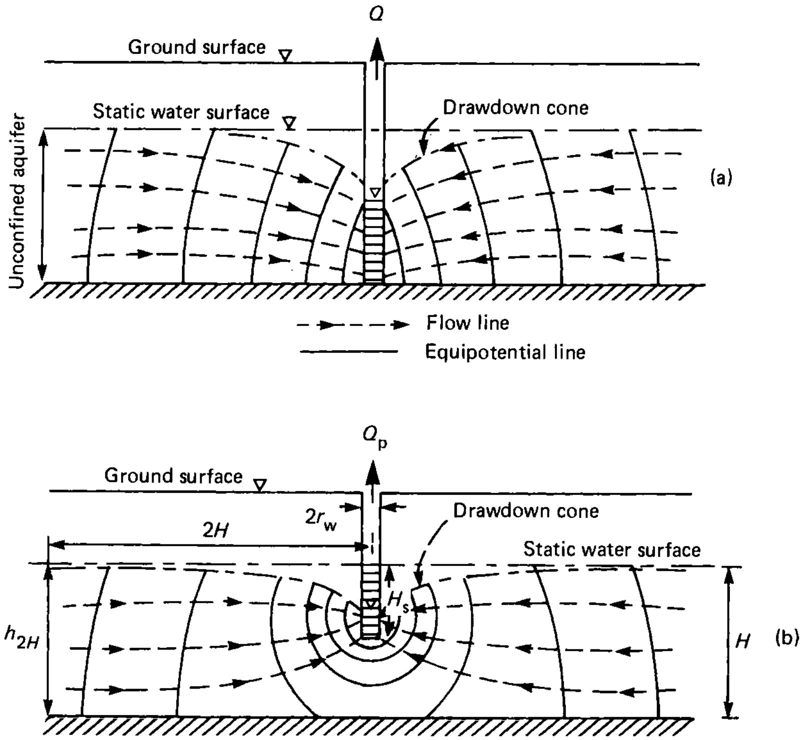


Figure 6.8 Flow to a well in an unconfined aquifer with (a) total penetration and (b) partial penetration (from Anon[2]). (a) Diagrammatic representation of flow to a fully penetrating and 50 per cent open hole in an unconfined aquifer. Note that this is technically a partially penetrating well because only 50 per cent of the saturated thickness is screened. However, in this case the upper 50 per cent of the hole is dry, so the effect of partial penetration can be ignored. (b) Diagrammatic representation of flow to a 50 per cent penetrating and open hole in an unconfined aquifer

penetration is greatest near the well and decreases with distance, until at twice the saturated thickness of the aquifer from the well the flow can be regarded as horizontal (*Figure 6.8(b)*). An additional consequence of partial penetration is that the assumptions made in the well discharge formulae (such as the equilibrium equations in Section 7.4.1) that the well totally penetrates the aquifer and that flow is horizontal are not valid. Hence, the theoretical equations are not valid and some other means of predicting the discharge from a partially penetrating well must be found. One such method is described in Section 6.3.2.

Whenever possible, wells in confined aquifers are designed to minimize the effects of partial penetration. This can be done relatively easily by screening the aquifer throughout its thickness to achieve a 100 per cent open hole and limiting drawdown to the top of the aquifer. There are two reasons why drawdown should be limited to the top of the aquifer. First, the wetted length of the screen remains constant and hence the yield per unit of drawdown does not vary, so the specific capacity is almost constant. Secondly, a larger drawdown would cause 'dewatering' of the formation within the cone of depression so that part of the aquifer would be unconfined and the rest confined. Under these conditions the well discharge equations would be inapplicable.

Although a 100 per cent open hole is preferable in a confined aquifer, about 90 per

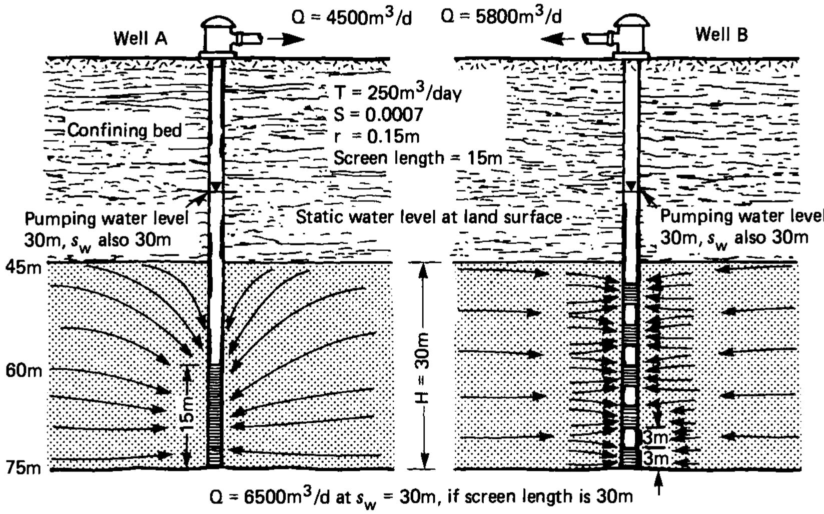


Figure 6.9 Example of the use of multiple screen section to reduce the effect of partial penetration in a thick confined aquifer. The total screen length is the same in both wells (after Anon[9])

cent of the maximum yield can be obtained if 70 to 80 per cent of the formation is screened, the percentage screened increasing above 70 per cent if the aquifer is thicker than about 8 m. In homogeneous aquifers the best results are obtained by centring the screen section in the formation, or by dividing the screen into sections of equal length and interspacing them with sections of ordinary pipe (*Figure 6.9*). This technique gives a significantly higher yield for a given screen length than a partially penetrating well with a low percentage of open hole. In non-homogeneous confined aquifers the screens should be divided into sections and placed opposite the most productive parts of the formation.

With a well in an unconfined aquifer, water tends to be drawn down from the surface of the water table so that the cone of depression forms within the aquifer material itself. The most obvious implication of this is that there is no point screening the entire thickness of the aquifer, because under operational conditions the upper part of the screen would be dry (*Figure 6.8(a)*). Consequently it is usual to screen only the bottom 33 to 50 per cent of the hole. Screening the bottom third is often regarded as the optimum design for this situation, since this gives about 90 per cent of the maximum yield. However, this does mean that there is considerable flow convergence near the bottom of the well which reduces efficiency. If efficiency is considered more important than yield, then the bottom half of the hole should be screened since this will reduce flow convergence and still give about 80 per cent of the maximum yield. In non-homogeneous unconfined aquifers the screens should be located in the most permeable strata of the lower part of the formation to allow for maximum available drawdown.

Since the wetted length of screen decreases as the drawdown in an unconfined aquifer increases, the situation is reached where with a 100 per cent drawdown the screens would be completely dry and there would theoretically be no flow. For this reason the well yield per unit of drawdown decreases as the total drawdown increases. In other words, the specific capacity is not constant and decreases with drawdown (*Figure 6.10*). Thus only a small gain in yield is obtained by increasing the drawdown from 65 to 100 per cent.

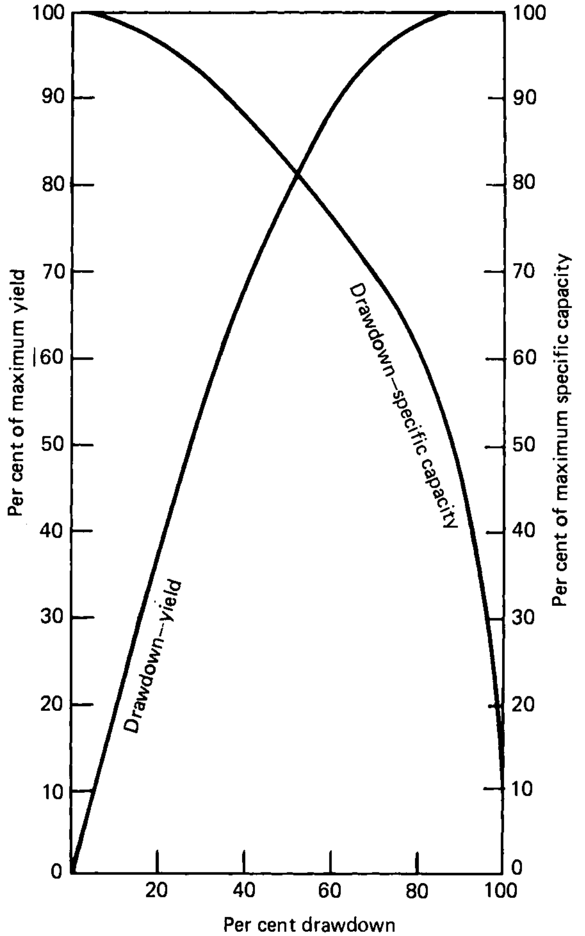


Figure 6.10 Relationship between yield and drawdown in an unconfined aquifer with 100 per cent penetration (after Anon[2])

6.3.2 The effect of percentage open hole and well diameter on available well yield

As explained above, the effect of partial penetration is always to reduce the available well yield for a given drawdown so that only a proportion of the theoretical maximum yield is actually obtained. The approximate yield of a partially penetrating well (expressed as a percentage of the maximum yield) is shown in *Figure 6.11* which is generally applicable to confined aquifers, where H_s, H and r_w are as shown in *Figures 6.7(b)* and *6.8(b)*. The graph is useful because it illustrates the relationship between the parameters frequently encountered in the design of a well. The values obtained from *Figure 6.11* are, however, approximate even if the aquifer is relatively uniform and homogeneous and its hydraulic characteristics and thickness are known.

To use the diagram, the screen length or the length of open hole is expressed as a percentage of the aquifer thickness, and the 'slimness' of the well, H/r_w, is calculated. After the appropriate value of the percentage screened has been located on the horizontal scale, move vertically upwards to intersect the line representing the well slimness. Then move horizontally to the vertical scale to obtain the percentage of the theoretical maximum yield that is actually available from the partially penetrating well.

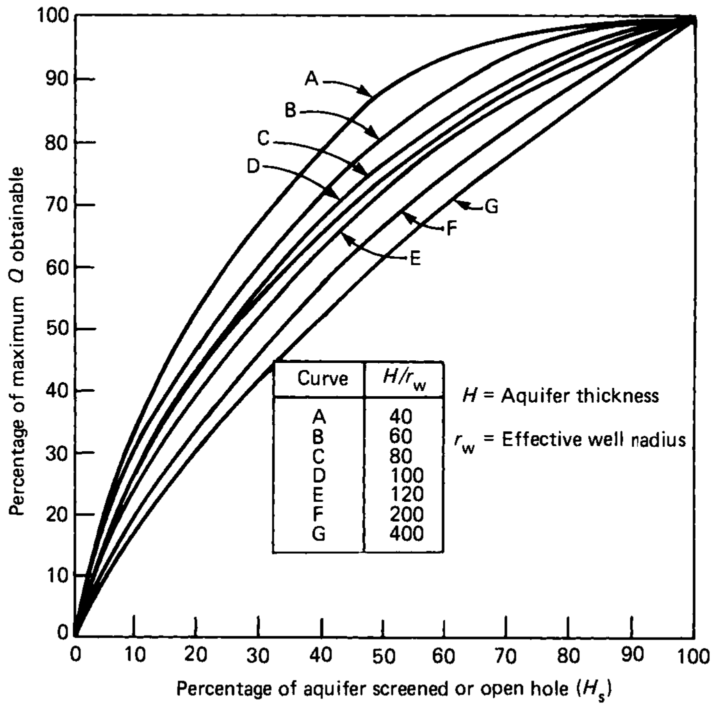


Figure 6.11 Graph of Kozeny's equation for the relative yield of a partially penetrating well in an ideal confined aquifer (after Anon[2]) which approximates the valid range of the equation

$$\frac{Q_P}{Q} = H_s[1 + 7(r_w/2H \,.\, H_s)^{1/2} \cos \pi H_s/2]$$

where Q_P is the yield of a partially penetrating well, Q is the yield of a fully penetrating well, H_s is the length of open hole as a fraction of aquifer thickness, r_w is the well radius and H is the aquifer thickness. The equation is not valid under all conditions (see Anon[9])

For example, if a 0.3 m diameter well penetrates 6 m into a 15 m thick confined aquifer, then $H_s/H = 40$ per cent and $H/r_w = 100$, so $Q_p/Q = 66$ per cent. For this particular well the actual discharge will be approximately 66 per cent of the theoretical maximum yield of a similar totally penetrating well. This is true regardless of whether the screens are located at the top of the aquifer, as in *Figure 6.7(b)*, or at the bottom.

Partial penetration of a confined aquifer results in a vertical component of flow to the well, but in the case of wells penetrating unconfined aquifers there is always a vertical flow component anyway. This is because the aquifer is dewatered by pumpage from the well and it is normal practice to screen only the bottom 33 to 50 per cent of the hole. Hantush[10] pointed out that provided the observed drawdown, s_w, is replaced by $s_w - s_w^2/2D$ where D is the depth of penetration, then the normal methods of analysing pumping test data from fully penetrated aquifers can be applied to partially penetrated aquifers when the pumping time is relatively short and the aquifer relatively thick.

The methods of calculating the effects of partial penetration outlined above are not very precise, but in many situations the errors incurred as a result of anisotropic conditions, heterogeneity, faulting or unknown aquifer characteristics will be as great or greater. Consequently these methods may suffice. If a better estimate of the

TABLE 6.2. Percentage increase in well yield with increasing well radius

Confined aquifer ($r=2500$ m)								
Well radius (m)	0.15	0.30	0.60	0.90	1.20	1.50	1.80	2.10
Well yield (%)	100	108	117	123	127	131	134	137
		100	108	114	118	122	125	127
			100	105	109	112	115	118
				100	104	106	110	112
					100	103	106	108
						100	103	105
							100	102
Unconfined aquifer ($r=150$ m)								
Well radius (m)	0.15	0.30	0.60	0.90	1.20	1.50	1.80	2.10
Well yield (%)	100	111	125	135	143	150	156	162
		100	113	122	129	135	141	146
			100	108	114	120	125	129
				100	106	111	116	120
					100	105	109	113
						100	104	108
							100	104

The above tables are based on the equation

$$Q=\frac{C}{\ln(r/r_W)}$$

where Q is the well discharge, r is the radius of cone of depression, r_W is the effective radius of the well or hole and C is a constant representing the other terms in the well discharge equations (see Chapter 7)

additional drawdown resulting from partial penetration is required, then Huisman[5] provided a suitable technique.

It is apparent from *Figure 6.11* that the available yield depends to some extent upon the diameter or slimness of the well. It is a common mistake to think that the yield of a well is directly proportional to the well diameter and that doubling the diameter will also double the yield. This is not the case because inspection of the well discharge formulae shows that yield is *inversely* proportional to $\log_e(r/r_w)$. Assuming the radius of the cone of depression to be 2500 m in a confined aquifer and 150 m in an unconfined aquifer, the effect of increasing the radius of the well is shown in *Table 6.2*, assuming that everything else remains constant. For instance, increasing the radius from 0.3 to 0.6 m in a confined aquifer will increase the yield by about 8 per cent and by about 13 per cent in an unconfined aquifer. However, it should be remembered that if the diameter and the yield increase, the drawdown and the radius of the cone of depression are also affected. It has been calculated that to theoretically double the yield of a well, the initial diameter would have to be increased by a factor of around 90 in the case of a confined aquifer and about 45 in the unconfined case[2]. Nonetheless very occasionally, doubling the diameter may increase the yield by as much as 25 per cent as a result of reduced turbulence and increased well efficiency (see Chapter 7). But in most instances any attempt to significantly increase yield by increasing the diameter of a well is likely to be expensive and not very successful. A much better approach is to increase the percentage of open hole by either increasing the screen length or by increasing the depth of the well, or both (see *Figure 6.2*). If this is not possible, the effective radius and specific capacity of the well might be increased by the use of a gravel pack (see Section 6.10), but this should be used as a last resort because gravel packing increases both construction and maintenance costs.

6.4 Selection of a suitable well diameter with respect to yield

The selection of a suitable diameter for a production well depends on several considerations. The flow to the proposed well will be radial and the velocity will increase progressively towards the well as the cylindrical flow area is reduced. The highest velocity will occur at the screens where the entire flow must pass through a vertical cylinder of relatively small diameter. The high flow velocities cause turbulence, which results in a head loss through the screen so that the water level on the outside is higher than the static water level observed inside and a departure from the assumption made in the well discharge equations that the flow is laminar. An additional result may be that the pick-up velocity of the finer fraction of the aquifer is exceeded and consequently this material is carried into the well so that it becomes a 'sand pumper'. This is obviously undesirable, so from the point of view of efficiency and operational problems the well diameter must not be too small. On the other hand, the diameter must not be too large otherwise construction would be uneconomic. An approximate guide to the minimum well screen diameter suitable for a particular yield is given in *Table 6.3*. This screen diameter can be used as a first approximation and modified by subsequent considerations such as entrance velocities (see below). The remainder of the well can then be sized relative to the screen diameter and the size of the pump. In general, the pump casing diameter should be two nominal sizes larger than the bowl size of the pump (or at least 25 to 50 mm) to prevent binding and reduce head losses. Suggested casing diameters for various pumping rates are shown in *Table 6.4*. The casing diameter may be reduced to that of the screens below the maximum anticipated pump setting depth if desired. The final configuration of the well depends upon many factors such as the location of the static and pumped water levels with respect to the ground surface and the bottom of the well, the depth of the well and the drilling method, whether it is necessary to reduce the hole diameter with depth to facilitate drilling and the strata penetrated. Briefly the options are to adopt a single string construction in which the screens and pump casing have the same diameter and are welded together, single string construction with a reducer so that part of the well (such as the screens) can

TABLE 6.3. Suggested minimum screen diameter (after Anon.[2])

Proposed well discharge	*Minimum nominal screen diameter*	
(m^3/day)	(mm)	(inches)
Up to 270	50	2
270–680	100	4
680–1 910	150	6
1 910–4 350	200	8
4 350–7 650	250	10
7 650–13 600	300	12
13 600–19 000	350	14
19 000–27 000	400	16
27 000–38 000	450	18
38 000–49 000	500	20

The table indicates the minimum diameter that should be considered. Keeping the diameter as small as possible reduces construction costs. However, if the well is to be pumped for long periods on a regular basis, running costs may be more significant in the long term than construction costs. Therefore, it may be more economic to adopt a larger diameter so as to improve efficiency by reducing the entrance and subsequent vertical flow velocities in the well (see Stoner *et al.*[1])

TABLE 6.4. Suggested minimum pump casing diameter*

Proposed well discharge	*Minimum pump casing diameter*	*Suggested surface casing diameter*	
		Naturally developed wells	*Gravel packed wells*
(m^3/day)	(mm)	(mm)	(mm)
Up to 490	150	200–250	450
270–820	200	250–300	500
550–2 725	250	300–350	550
1 635–8 175	300	400–450	600
2 725–10 900	400	400–450	650
8 175–16 350	400	450–500	700
10 900–27 250	500	500–550	750
16 350–27 250	600	600–650	850
21 800–43 600	700	650–700	900

* Based on the use of the most suitable turbine pumps after Anon[2]. Check the size of available pumps with manufacturers before finalizing design

be of a smaller diameter, or a telescoping design where the diameter may be reduced several times with depth, the individual lengths of casing being overlapped and sealed with a packer. Typical examples of well construction are given in Section 6.8.

Sometimes a temporary or permanent casing may be used to support unconsolidated or unstable ground near the surface while the well is under construction, since the upper part of the formation may be subjected to considerable disturbance as a result of drilling, construction and completion processes. The surface casing prevents caving and reduces loss of drilling fluid. Obviously the surface casing must be large enough to allow all the other components required for well construction to pass through it. *Table 6.4* suggests a minimum suitable diameter for various discharges.

6.5 Well screens

Well screens serve two basic functions. First, they support the sides of the hole in unconsolidated or fractured ground thereby preventing collapse. Secondly, they keep sediment out of the well while still offering the largest practical open area to the aquifer to minimize resistance to flow. Hence the requirements of a good screen are that it should be strong and have a large open area. Since these two requirements interfere with each other, some compromise may be necessary at times.

The screen is sometimes referred to as the business end of the well. Hence it is very important that care is taken to obtain a screen that is suitable for the formation in which it will be set. An inappropriate choice could result in an inefficient well that will be both expensive and difficult to operate.

6.5.1 Entrance velocity

As mentioned above, the velocity of the water as it enters the screens must not be too high otherwise the well would be inefficient and might also be a sand pumper. The entrance velocity depends upon the yield of the well, Q m^3/s, and the open area of the screen, A m^2. The relationship

$$V = Q/A \text{ m/s} \tag{6.1}$$

is adequate for the calculation of the average entrance velocity, V m/s, which should be 0.03 m/s or less for optimum efficiency[2,9]. If the entrance velocity exceeds 0.045 m/s, then steps should be taken to reduce this to a more acceptable value by increasing the open area of the screen.

6.5.2 Open area of a well screen

The open area of a well screen, A m^2, depends upon its diameter, D m (or to be more correct, its circumference), its length, Lm, and the percentage of open area afforded, a per cent, thus

$$A = \pi DLa \text{ m}^2 \tag{6.2}$$

This expression can be used to calculate the length of screen required for various diameters or, if the diameter of the well has already been decided, the entrance velocity can be controlled by varying either the length or the percentage open area of the screen. For example, the entrance velocity could be decreased by increasing the percentage open area, provided of course, that screens with the required open area can be manufactured and would be strong enough to support the aquifer. Depending upon the material and the method of manufacture, well screens have percentages of open area ranging from about 1 to 62 per cent. As a rough guide, the percentage of open area should not be less than 15 per cent to minimize headloss, while 15 to 25 per cent is a reasonable compromise between efficiency and strength, if strength is important. There is some advantage in using open areas of up to 40 per cent since resistance to flow is reduced if the open area of the screen is near the porosity of the aquifer, but there are few good reasons for exceeding 40 per cent. It should also be remembered that part of the screen may become blocked during its operational life and typically up to half the open screen area may be lost. Consequently the entrance velocity should be kept low initially to allow for blockages.

6.5.3 Selection of screen slot width

The well screen slot width plays an important part in determining whether the formation can be successfully developed and how much of the formation material can pass into the well. If the slot width is too large, the formation may not stabilize so that fine sediment is continually drawn into the well and it becomes a sand pumper. On the other hand, slots that are too narrow may cause clogging, high friction losses and possibly encrustation problems.

For naturally developed wells in unconsolidated deposits (see Section 6.9), the screen slot width is usually selected from a sieve analysis. The results of the analysis are recorded on a graph of cumulative per cent retained against particle size. In the case of fine uniform sand, the slot width is chosen so that between 40 and 60 per cent of the particles are retained[9,11]. The slot width required is found by obtaining from the cumulative curve the particle size corresponding to, for instance, 40 per cent retention. If the groundwater is not corrosive and if the sieve analysis gives a true reflection of the aquifer material, then the 40 per cent value is usually used. If, however, the groundwater is corrosive and it is probable that the slot width will be enlarged with time, or if there is any doubt about the reliability of the sample, then a more conservative retention value of around 50 per cent should be adopted.

In homogeneous aquifers of coarse sand or gravel, the slot width is generally selected so as to retain about 30 to 50 per cent of the material. In non-homogeneous aquifers the slot width may have to be varied to suit the strata concerned. Generally slot sizes of 0.15 to 0.5 mm should be limited to use with small, low capacity wells[2]. If a large capacity well requires a slot size of less than 0.75 mm to stabilize the formation, then it is probably advisable to use a gravel pack thereby allowing an increased slot width.

6.5.4 Screen types and materials

The type of screen used in the construction of a well varies according to individual requirements and conditions. For example, if a well has to be constructed cheaply in a relatively remote part of the world then a screen may be manufactured on site out of ordinary steel pipe, but if the requirement is for a highly efficient long-lasting well then a more sophisticated screen is used. There is a large range of screen types and materials some of which are described below and compared in *Tables 6.5* and *6.6*.

The simplest and most basic type of screen is that fabricated from plain steel pipe. The crudest form is that produced by a casing perforator, a tool that punches holes in the casing after it has been placed in the well. This gives jagged openings of uncontrolled width and size, while the percentage open area is low. Better results can be obtained by machining or torch-cutting slots in the casing, although the open area may still be low despite the fact that the slots may be quite wide (*Figure 6.12*). Slotted screens also may be made out of other materials such as stainless steel, which gives them a greater resistance to corrosion but does not make them any more efficient. Gauze screens use a perforated pipe as a base, but have coarse wire gauze and filter mesh wrapped around the pipe to produce a screen that is more efficient as a strainer, but which is easily blocked and not very strong.

Screens that have long narrow slits provide a greater resistance to blocking, and these can be manufactured in various ways in several different styles. The slits may be continuous or intermittent, horizontal or vertical and formed by cutting, punching or other means. In general these manufacturing techniques tend to produce rather wide slits, unless the material is only partially removed by pressing it out to give a bridge or louvre type screen in which case finer slits can be produced. However, the percentage open area of these screens is not large and the shape of the louvre openings makes them totally unsuitable for naturally developed wells. As a consequence they must always be used with a gravel pack.

The continuous slot type of well screen is made by winding wire, which is approximately triangular in cross section, spirally around a circular array of longitudinal rods. At each point where the wire crosses the rods, the two members are securely fastened, typically by welding. The well screen then becomes a rigid one-piece unit with V-shaped continuous openings which widen inwardly making them less prone to clogging, increasing hydraulic efficiency (see *Figure 6.13*) and allowing effective development of the well. In addition, it is possible to vary the pitch of the wire during the winding process to obtain screens with different slot widths, ranging from 0.25 to 6.0 mm. Such screens may be fabricated from various types of metal such as steel, galvanized iron and stainless steel (see *Table 6.6*) and are available in standard sizes suitable either for telescopic or continuous string construction[9].

A relatively new type of well screen has been introduced that incorporates a prefabricated pack of epoxy-cemented sand grains around a perforated pipe base.

TABLE 6.5. Comparison of common screen types

Type of screen or method of manufacture	*Typical fabrication material*	*Range of slot widths* (mm)	*Maximum open area* (%)	*Comments*
Pipe perforated *in situ*	Low-grade carbon steel. Possibly coated	3–12 wide by 25–50 long. Difficult to control	3–4	Cheap, but ragged edges cause corrosion and encrustation. Low open area, no control over slot width, poor hydraulic efficiency, difficult to develop well
Perforated casing torch-cut, sawed or machined slots	Carbon or stainless steel	0.25–6.0	Up to 12	Torch-cut slots similar to above. Machined slots more uniform but open area still low and well still difficult to develop
Punched or stamped perforations	Carbon or stainless steel	1.5–6.0	4–20	Open area increased, but still similar to above
Louvre screens	Carbon or stainless steel	0.75–5.0	3–33	Slot sizes accurate and uniform. Openings are the preferred continuous slot type. Shape of openings makes well difficult to develop
Wire wound cage screens	Commonly stainless steel or galvanized iron or steel. Also special alloys (see *Table 6.6*)	0.25–6.0	2–62	Round or specially shaped wire designed to prevent clogging can be used. Continuous slots aid well development. Good hydraulic efficiency. Largest open area available. Acid and corrosion resistance good. Expensive but good value
Plastic	Plastic	0.15–1.25	3–24 Generally low	Collapse resistance of unreinforced plastic doubtful, particularly below 50 m. Generally limited to shallow small wells of up to 150 mm diameter. Good corrosion and acid resistance
Epoxy/reinforced plastic	Plastic	0.15–1.25	Low	More desirable than pure plastic. Have been used in wells up to 100 m deep. High corrosion resistance. Not always competitive economically

TABLE 6.6. Details of metals used in fabricating well screens, and suggested applications (after Anon.[9])

Metal or alloy	*Nominal composition*	*Cost factor*	*Suggested applications*
Monel	70% nickel 30% copper	1.5	High sodium chloride combined with dissolved oxygen as in sea water. Usually not needed for potable groundwater
Stainless steel	74% steel 18% chromium 8% nickel	1.0	Hydrogen sulphide. Dissolved oxygen. Carbon dioxide. Iron bacteria. Excellent strength
Everdur	96% copper 3% silicon 1% manganese	1.0	High total hardness. High sodium chloride where dissolved oxygen is absent. High iron. Extremely resistant to acid treatment
Silicon red brass	83% copper 16% zinc 1% silicon	0.9	Used for same conditions as Everdur, but not quite as good. Not as strong as Everdur. Used in relatively inactive waters
Armco iron	99.84% pure iron (double galvanized)	0.6	Not corrosion resistant, but functions satisfactorily in some areas. Used for irrigation wells in areas where waters are relatively neutral
Steel	99.35/99.72 iron 0.09/0.15 carbon 0.20/0.50 manganese (double galvanized)	0.5	Not corrosion resistant. Generally used only in temporary wells, such as test wells or wells for dewatering. Satisfactory service life in some areas of southwestern United States where waters are non-corrosive and non-encrusting

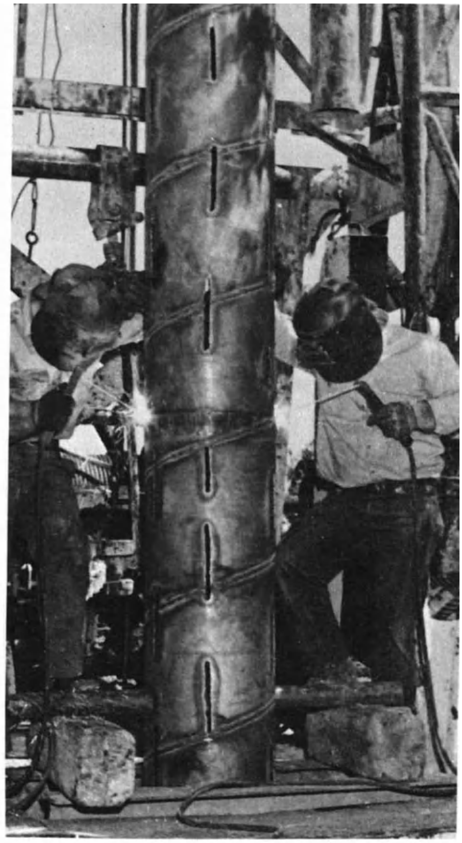
Plastic screens are another relatively recent addition to the range, but are generally limited to shallow wells of small diameter, perhaps up to 150 mm. These screens offer good resistance to aggressive groundwaters but their strength is somewhat suspect. Reinforced and epoxy plastic screens are available that have a greater collapse resistance and these have been used to depths of around 100 m.

The types of screen described above are some of those that are in common use, but screens have been made out of wood, asbestos, cement, concrete, cast iron, glass, vitrified clay and porcelain. It is also possible to coat a screen with one of the following substances, chlorinated rubber paint, hard rubber, bitumen, or plastic in an attempt to prolong its life. This is especially important if the screen is made from a material that is not naturally resistant to corrosion. Unfortunately, it is very difficult to complete and develop a well without chipping or damaging the protective coating. Plastic is probably the most resistant to chipping, but there is no substitute for using a screen that is suited to the local conditions.

6.5.5 The sediment sump

An essential part of the screen is the sediment sump. This is a blank section of pipe, 1 to 2 m long, secured to the lower end of the screen, the purpose of which is to collect the sediment that enters the well through the screen and then settles to the bottom[11]. In this way the lower part of the screen is prevented from becoming blocked. The accumulated sediment should be removed periodically as part of the regular maintenance of the well. The bottom of the sump is sealed commonly with a metal or wooden plug.

(a)



(b)

Figure 6.12 Types of well screen. (a) Casing perforator produces crude jagged openings of uncontrolled size. (b) In torch-cut, slotted pipe, per cent open area is low and width of slots is too large to permit developing well to sand-free condition.

6.6 Well straightness, verticality, hole diameter and formation fillers

It is practically impossible to drill a well that is perfectly straight and truly vertical. During drilling the alignment of the hole is influenced by changes in strata, in particular variations in hardness, which tend to deflect the bit from a vertical course. In unconsolidated deposits the edge of a boulder may deflect the drill bit and may have the same effect on a well casing. As a result the hole may drift increasingly with depth. Obviously the alignment of a hole should be checked periodically during drilling and it is necessary to have some idea of what does and does not constitute an acceptable deviation. Quoted specifications are that the deviation should be less than 100 mm or 150 mm from the vertical per 30 m, or that the deviation from the vertical should be not more than two-thirds of the inside casing diameter per 30 m[2,12,13,14]. In general it is more important that the hole should be straight than truly vertical, because if the hole is not reasonably straight it may be difficult or impossible to install the screen without having to drive it down, which could damage it. To overcome this problem the hole is

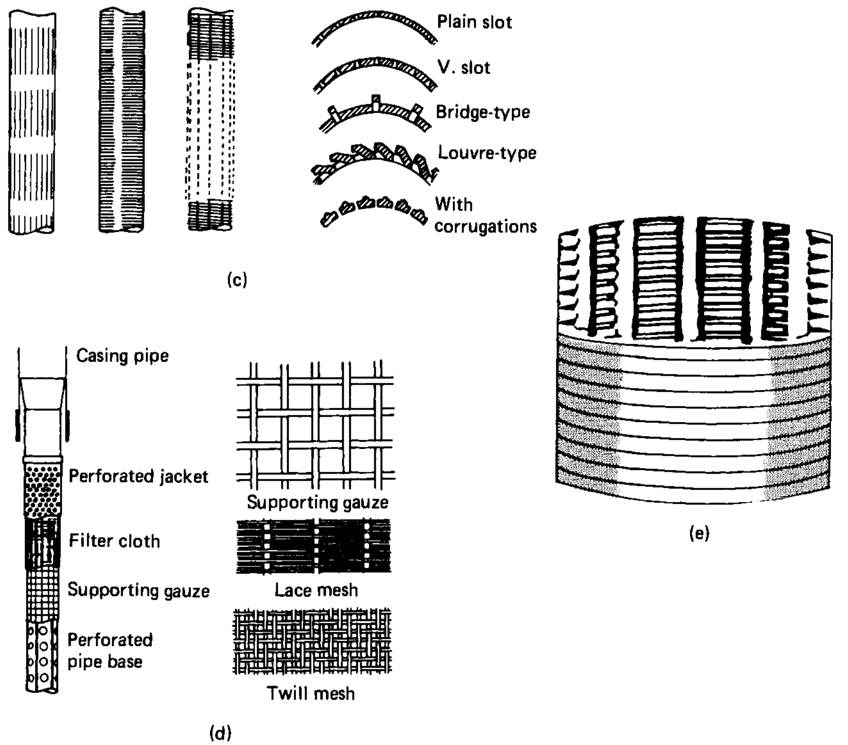
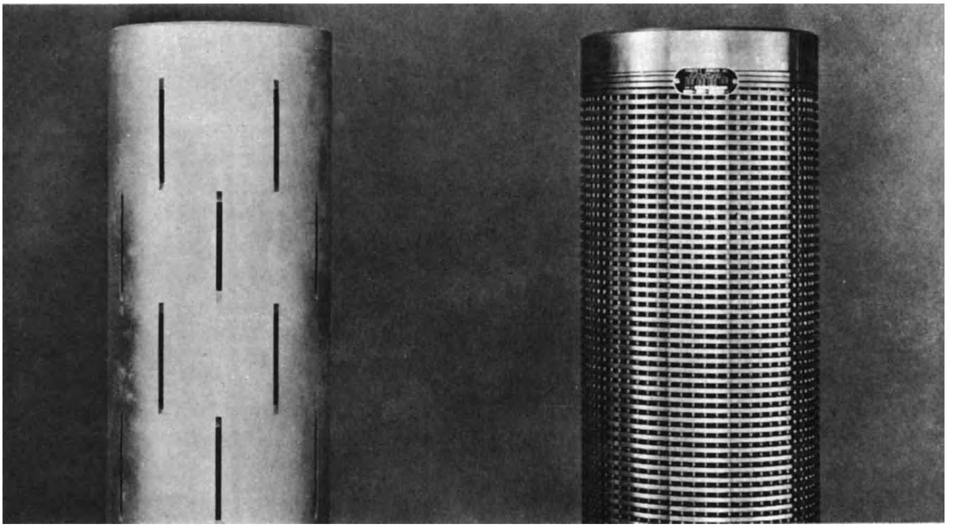


Figure 6.12 (c) Construction of slotted screens. (d) Construction of gauze screens. (e) Section of continuous slot screens shows heavy, welded construction with V-shape openings. ((a), (b) and (e) after Anon[9], (c) and (d) after Huisman[5])

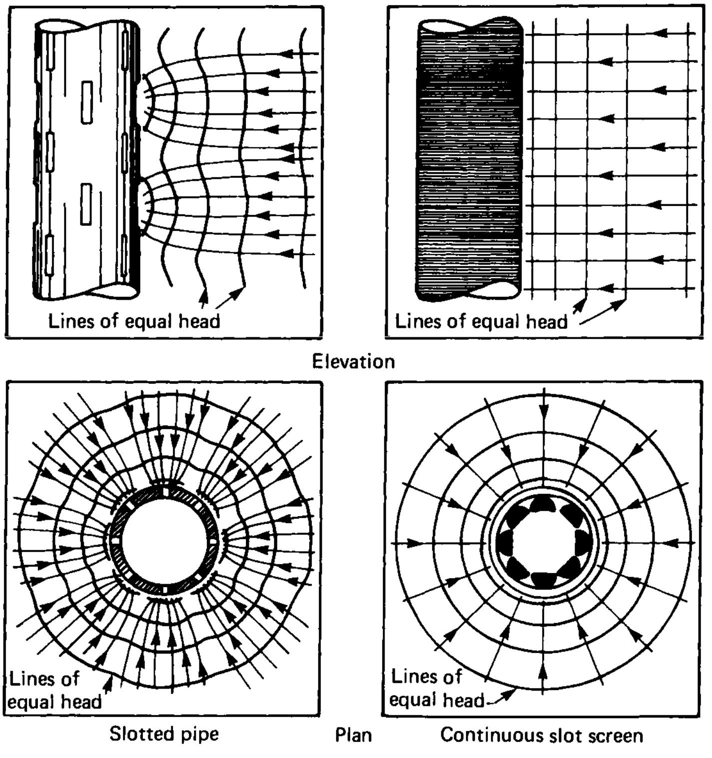
usually drilled oversize to ensure that the screen can be installed without mishap and to allow some slight movement of the screen relative to the hole to make certain that the axis of the screen assembly coincides with that of the pump casing. If the screen is installed crooked in the hole and subjected to bending stresses, this may alter the slot width or cause collapse.

There is no fixed amount by which the hole should exceed the screen or pump casing diameter, since every situation is different and must be decided from a consideration of the strata drilled through, the experience of the driller and the depth of the well. Obviously the amount should be large enough for safety, but not excessive, otherwise penalties may be incurred in the form of increased costs and constructional difficulties. An acceptable compromise may be to make the diameter of the drilled hole 100 mm larger than the well screen[9]. This leaves a 50 mm annular space around the screen and it is advisable to fill this with a formation stabilizer to prevent material from above the aquifer caving around the screen. It also assists the development process. If the space is less than 50 mm a formation stabilizer may not be necessary.

A formation stabilizer acts in a similar way to a gravel pack. The material of which it is composed has the same, or a slightly coarser, grading to the aquifer. The stabilizer simply replaces material removed during construction. The screen slot width should be chosen to suit the aquifer and not the formation stabilizer, the intention being to remove the fine material from both during well development.



(a)



(b)

Figure 6.13 Effect of screen openings on flow patterns around a well (after Anon[9]). (a) Comparison of slotted pipe and well screen, both of stainless steel and with same width of openings, shows that the continuous slot screen has 10 times as much open area per metre length or per square metre of screen surface. (b) Flow nets around two types of screen. Water approaches openings along line indicated by arrows. Flow lines for slotted pipe converge to individual slots; flow lines for well screen are less distorted

When casing or screen assemblies over 12 m in length are to be installed in holes that have a nominal diameter 50 mm or more larger than the outside diameter of the casing, centring guides should be used to hold the casing in the centre of the hole. These guides should be attached to blank sections of pipe located at suitable intervals in the screen and pump casing assemblies. To prevent galvanic action the guides may be made of wood or the same metal as the casing and screen assembly to which they are attached. Metal centring guides are often made out of strips of metal about 30 mm wide bent out into the shape of a bow, the high point of which presses against the side of the hole and thus centres the casing[2]. To be effective the guides must be equally spaced around the casing, either at 90° or 120° intervals, and should be between twice and four times the diameter of the casing in length. Care should be taken to ensure that the guides are located vertically above each other, otherwise they may impede the insertion of a tremie pipe and complicate the placement of the formation stabilizer or gravel pack.

6.7 Pump selection and depth of setting

There are many different types of pump that can be used to lift water from a hole, but the most commonly used types are the vertical shaft turbine pump and the submersible centrifugal pump. Detailed information regarding particular types of pump and their characteristics is readily available from the manufacturers. Consequently pumping equipment will not be reviewed in detail below.

The components of a typical pump consist of an intake section containing the impellers, a motor, a drive shaft connecting the motor and impellers and a water delivery pipe. With a vertical shaft turbine pump the motor is located at the ground surface and connected to the impellers, which are immersed below the pumping water level, by a long vertical drive shaft housed within the central water delivery pipe (*Figure 6.14*). This arrangement requires a straight well because the alignment of the drive shaft is critical to the smooth operation of the pump. It is also important that the drive shaft is adequately lubricated. In a submersible pump the motor and the impellers are close coupled with the electric motor being located below the impellers (*Figure 6.15*). This eliminates the problems associated with a long drive shaft, but means that the pump unit must be pulled from the well when repairs or maintenance are required. Submersible pumps are used for lifts of up to 400 m, while shaft turbine pumps are used for lifts of up to around 250 m[9].

Selection of a suitable pump for a particular location will be based upon performance characteristics published by the pump manufacturers in the form of a series of graphs. These performance curves show the relationship between pump discharge, head or lift, power requirement and pump efficiency for a particular combination of pump and impellers. A typical set of performance curves are shown in *Figure 6.16*. Such data can be used to pick a pump that can discharge the required amount of water against the total head at near maximum efficiency with the lowest possible power requirement. The efficiency and power requirement of the pump are important because this will influence the running cost of the well over a considerable period of time.

Although it is quite easy to select a suitable pump from the manufacturers' data when the precise relationship between head and discharge is known, it is much more difficult to determine what this relationship will be at the well site. For a start, the lift required of the pump is not simply the distance from the pumping water level to the ground surface because there are other factors which must be taken into consideration. The total head

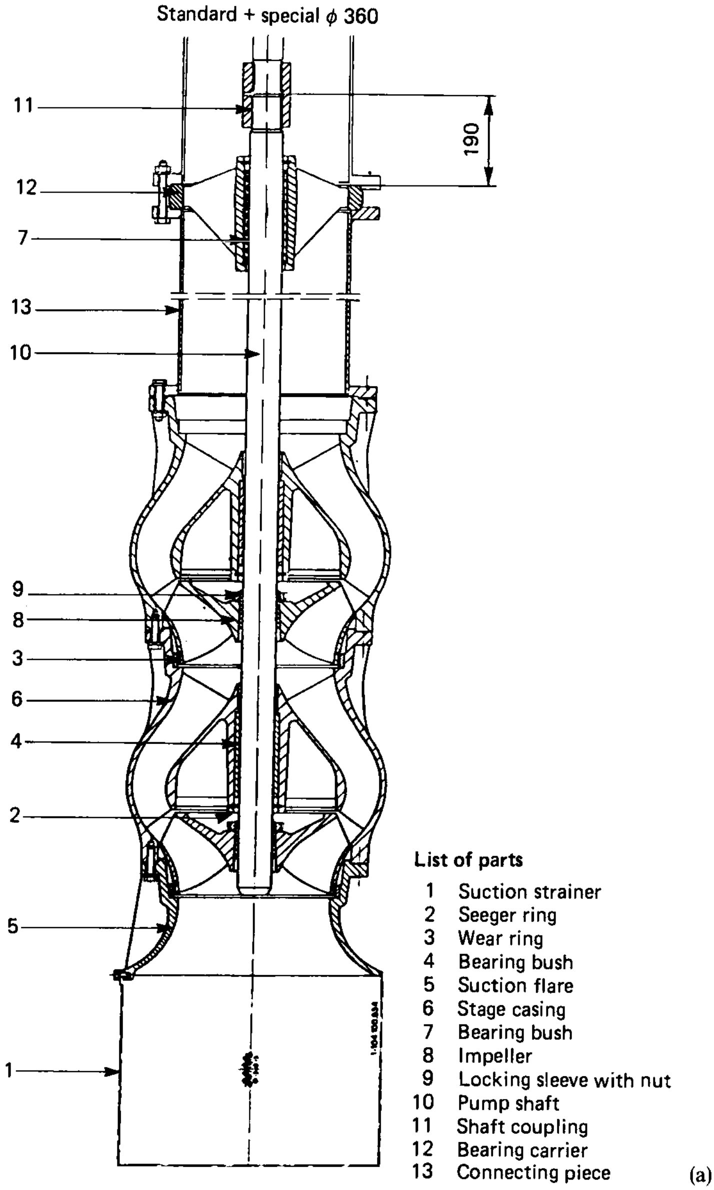
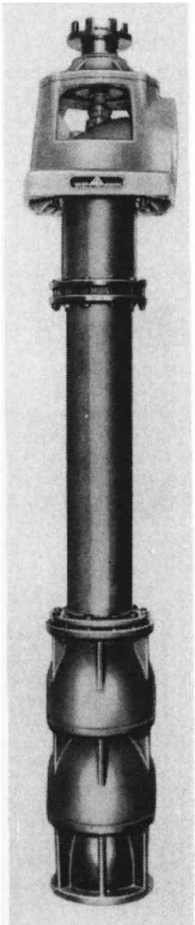


Figure 6.14 A shaft driven multi-stage borehole pump (a).

therefore is expressed by the following formula

$$h_t = h_L + h_f + h_v + h_s \tag{6.3}$$

where h_t is the total head (m), h_L is the total vertical lift (m). It should be remembered that the pump will have to lift the water from the pumping level to some point *above* the ground surface and not to the ground surface. Usually the pump will have to deliver the



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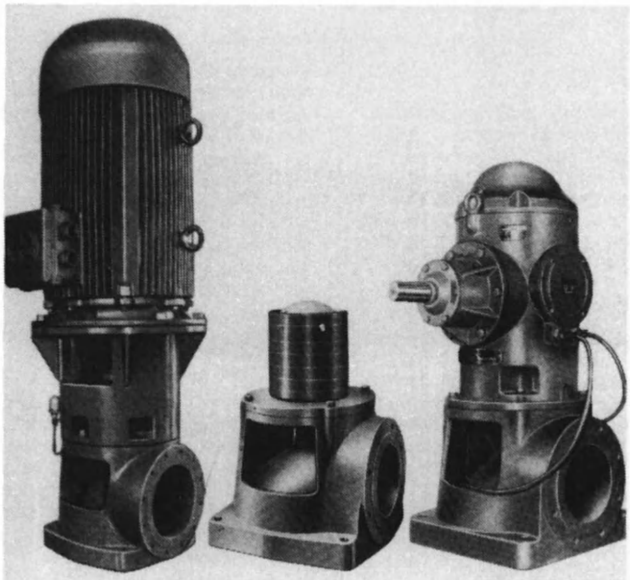


(c)

Figure 6.14 (*continued*) The pump delivers water via a rising main, which requires special head gear to accept the drive shaft (b). The pump is driven by a motor mounted above the head gear (c). (Courtesy Sulzer Brothers Ltd)

water to a storage reservoir at the surface, or discharge through a pipeline (*Figure 6.17*). Allowance must be made for this. h_f is the total friction losses (m) in the well and the delivery pipe, h_v is the velocity head (m) at discharge point and h_s is the submergence head (m) as shown in *Figure 6.17*.

The total vertical lift required of the pump will not be constant, because the water level in the aquifer will fluctuate seasonally and over a number of years according to aquifer recharge and pumping rates. In addition, the yield required of the well may not be constant, since this depends upon demand and the availability of water from other sources. Possibly the highest demand on the well is during a drought when the water table is at a relatively low level, so the pumping plant must have the flexibility to operate reasonably efficiently over the range of anticipated head and discharge. Similarly, it is always good practice to install a pump that can supply more than the design discharge to allow for exceptional demands and the occasional period of over pumping, perhaps while a neighbouring well is undergoing repair. Therefore, it is

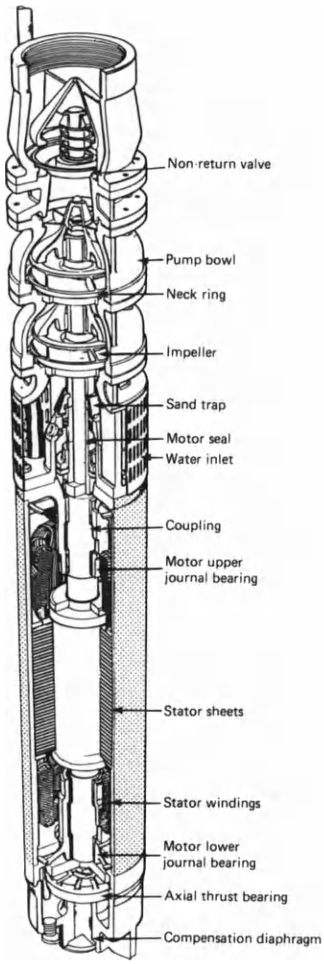


Figure 6.15 A modern multi-stage submersible borehole pump. The 400 V 3-phase electric motor works completely immersed in water. (Courtesy EMU Pumps Ltd)

important that the pump is set at a level that will allow increased drawdown either as a result of increased discharge or decreasing well efficiency due to fouling of the screen. This is another situation where compromise is necessary because if the pump is set too high it may be above the desired pumping water level during drought or periods of increased demand, but if it is set too low the pump will have to operate against a higher than necessary head which will increase the power requirement and hence the running costs.

The depth of setting of a pump, and thus the depth of the pump chamber casing, should be determined from consideration of aquifer water levels, estimated pumping levels, and future trends and requirements. According to Anon[2] this involves an appreciation of the following factors

1. Present static water levels.
2. Minimum static water level recorded in the area.
3. Water level trends in the area and estimates of the minimum water level likely to be associated with given return periods or frequency of occurrence, such as a 1 in 50 year drought.

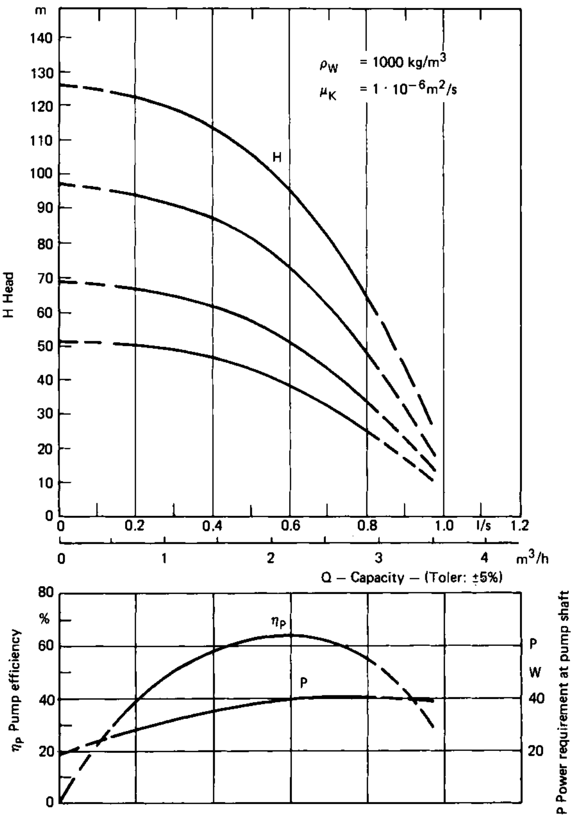


Figure 6.16 Typical head-discharge, efficiency and power requirement curves for a range of multi-stage submersible borehole pumps of 97 mm diameter. (Courtesy EMU Pumps Ltd)

4. Estimated drawdown at the desired well yield.
5. Possible interference by other wells in the area.
6. Required pump submergence.
7. Future operating requirements.
8. The rate of decline of well efficiency.
9. Well construction features, such as the presence of any telescoping overlap.

Admittedly not all this information may be available, so a decision must be taken using whatever data are at hand. It is important that the pump is set at a suitable depth, otherwise considerable operational problems may result and a well may only be able to offer a reduced yield when it is most needed. Once the decision has been made and the pump installed it may be difficult or impossible to alter the setting should it become apparent that the original depth is unsuitable.

If the well design calls for a reduction in diameter below the pump chamber casing, then this should occur at a distance of about ten pipe diameters below the intake or bottom part of the pump.

6.8 Details of well construction

The objective of this section is to describe briefly the methods that may be used to locate and set well screens and complete the construction of the well[2,9,15]. The use of screens is

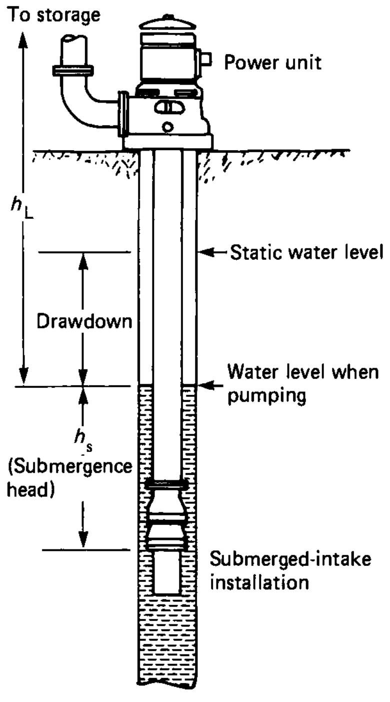


Figure 6.17 Lift required of a pump. A water well pump works against a total head which includes vertical lift, h_L, submergence lead, h_s, plus friction losses plus velocity head

most common in unconsolidated deposits that are not stable, so the problem is basically how to drill a well and set the casing and screen without allowing the hole to collapse.

6.8.1 Methods of setting well screens

The simplest type of well design is that using a single continuous string construction (*Figure 6.18*) in which the screen is welded to the casing. Such wells are easiest to construct if the well is drilled by a rotary method so that the hole is self-supporting. This allows the string to be dropped into the completed hole either as one unit or a piece at a time, depending upon length.

The standard method of constructing wells of telescoping design is the pullback method (*Figure 6.19*). Casing is sunk to the full depth of the well, either as drilling progresses or afterwards, and the screen is lowered through the casing. Many commercially available types of screen are designed so that they will pass through pipe of the same nominal diameter[16]. The casing is then pulled back to expose the screen to the water-bearing formation. The joint between the screen and the casing is sealed using either a self-sealing or lead packer. A lead packer must be expanded by a tool called a swedge block to form an effective seal (*Figure 6.20*).

The standard method of construction is suitable when the hole has been drilled with a percussion cable-tool rig and can also be used when the hole has been constructed using a rotary technique. However, with rotary drilled holes an alternative method of construction is to drill the hole to full depth, then install the solid casing in the top part of the well. The screen is lowered through the casing, the bottom plugged and the top sealed by a packer. As a variation on this method, the hole may be drilled only as far as

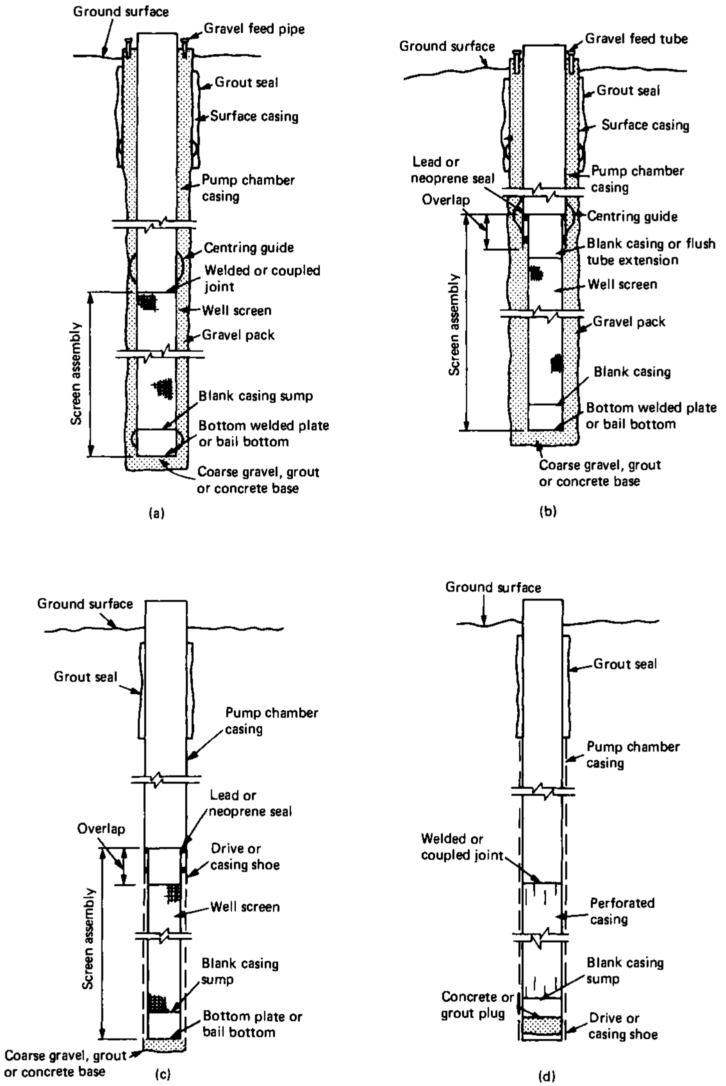


Figure 6.18 Standard well designs used by the American Water Works Association (after Anon[2]). (a) Gravel packed, rotary drilled well for single string construction. (b) Gravel packed, rotary drilled well for telescoping construction. (c) Straight wall, cable tool drilled well for pull-back construction. (d) Straight wall cable tool drilled well for single string construction.

the depth at which the casing is to be set permanently. The casing is then positioned in the hole and grouted as required. The rest of the hole is drilled using a bit just large enough to go through the casing. A telescope size screen is then lowered through the casing into the open hole below (*Figure 6.21*), the drilling mud is removed and the lead packer expanded.

Sometimes the pull-back method proves difficult to use as a result of the grip exerted by subsurface materials on the casing. In such situations the bail down method of setting the well screen may be used (*Figure 6.22*). The first step is to drill and install

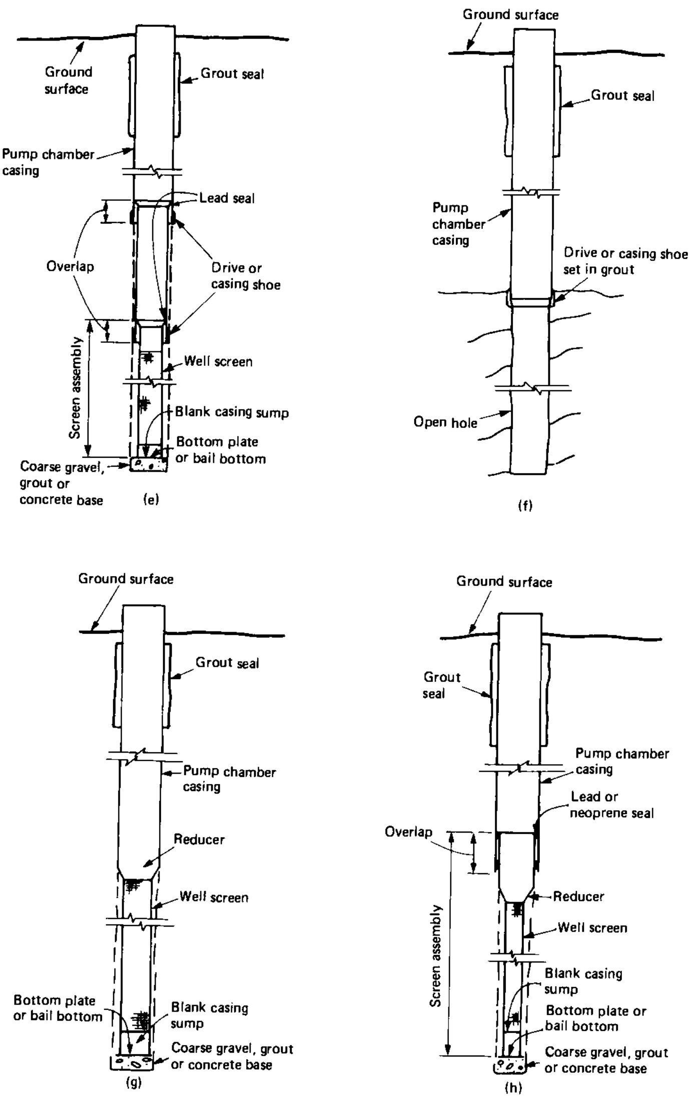


Figure 6.18 (e) Straight wall, cable tool drilled well for multiple telescoping construction. (f) Straight wall, cable tool drilled well in consolidated material (rock). (g) Straight wall, rotary drilled well for single string construction using a reducer. (h) Straight wall, rotary drilled well for telescoping construction using a reducer

casing to a depth such that the pipe is in its permanent position. The screen is then telescoped down through the casing until it rests on the bottom of the hole, after which drilling is restarted. The screen is sunk into the formation below the well casing by operating the bailer and drilling tools through the screen. As the material is removed from below the screen it gradually sinks through the formation. When it has reached the desired depth, the bottom of the screen is plugged and the joint between the screen and casing sealed with a packer. There are other methods of setting screens, but most of them bear some similarity to those described above.

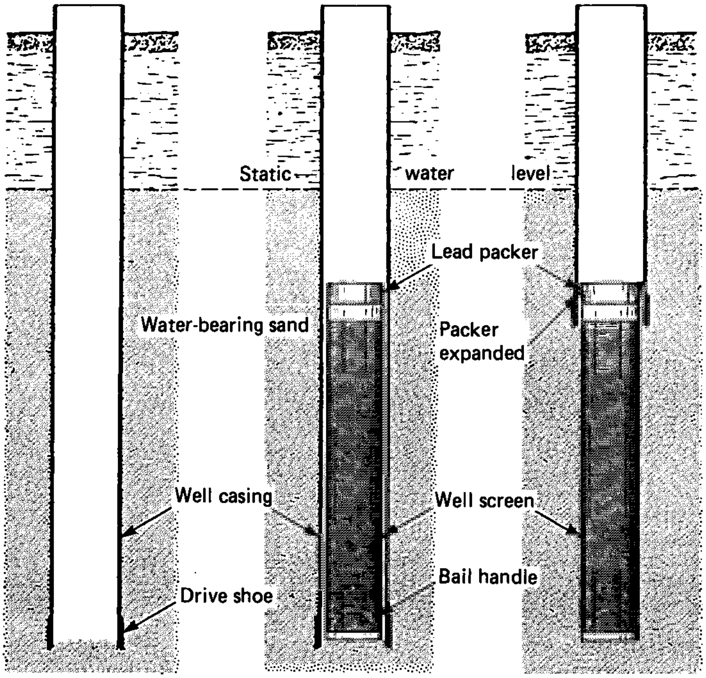


Figure 6.19 The pull-back method of setting well screens (after Anon[9])

6.8.2 Overlaps

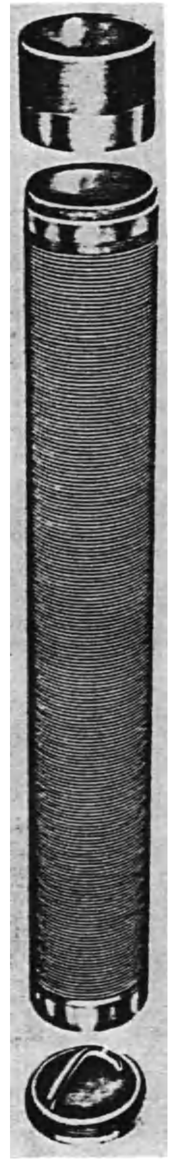
The length allowed between casing and screen assembly is usually about 1 to 2 m. The overlap may be increased in deep wells or those constructed in areas where the possibility of settlement exists. The overlap should be sealed using a packer, as described above and shown in *Figure 6.20*.

6.8.3 Seals around the surface casing

As explained above, the size of the drilled hole is larger than the outside diameter of the well casing and screen assembly. Within the aquifer this annular space is filled with a formation stabilizer, while in the other strata it is usually filled by a grout seal. This seal serves a similar function to the formation stabilizer in that it supports the side of the hole and prevents caving around the casing, but its primary purpose is to form a sanitary seal between the ground surface and the aquifer[2]. Consequently, the sanitary seal should be of sufficient thickness (generally greater than 40 mm), depth and imperviousness to prevent any polluted surface water from entering the aquifer. This is usually achieved by using a cement-based grout mixed with bentonite or aluminium powder over the entire length of the casing. The grout may be placed using a grout pipe or by displacement[9]. If the overlapping casings of a well of a telescopic design have a difference in diameter of about 25 mm or more, it is good practice to fill the annular space with a neat cement–bentonite grout seal.

Regardless of the seal placed around the casing it is important that the area around the well at the ground surface should be finished with a good strong concrete apron.

(c)



(a) (b)

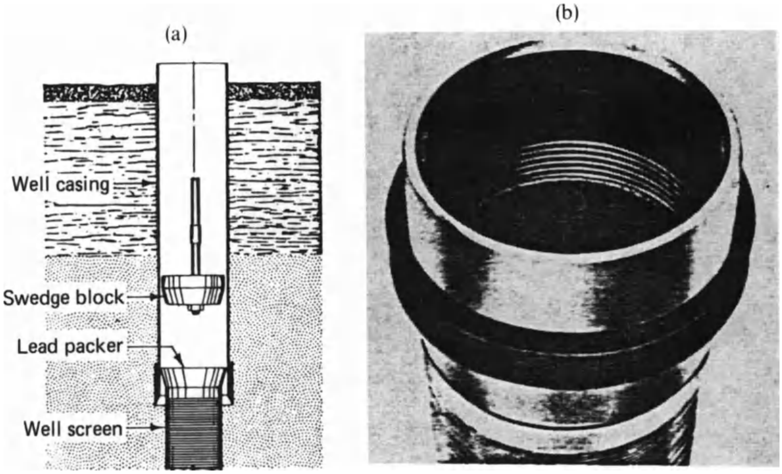


Figure 6.20 Packers and pipe end fittings (after Anon[16]). (a) Lead packer on telescope-size screen expanded to seal inside well casing. Swedge block (shown above screen) is tool used for expanding lead packer. (b) A self-sealing neoprene packer is frequently used instead of a lead packer. It requires no swedging operation. (c) Well screen showing standard end fittings consisting of lead packer at top and closed bottom with bail

This will reduce the effect of settlement around the hole and should be designed so that water will run away from the well.

If the well penetrates more than one aquifer, special consideration must be given to the provision of seals. It is usually desirable to isolate the aquifers to prevent flow from one to the other, particularly in the event that one should become polluted. If one of the aquifers contains water of an inferior quality, then it must be sealed off to safeguard the productive formation (see Chapter 9).

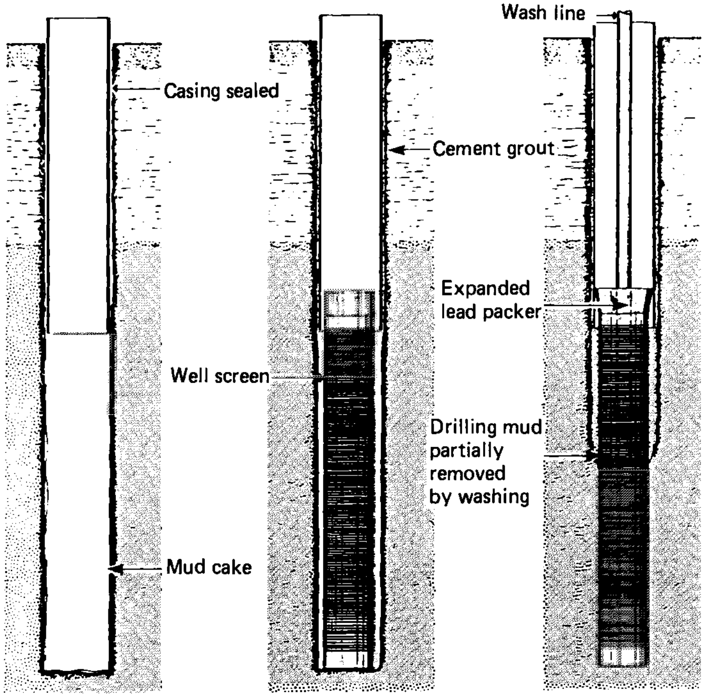


Figure 6.21 Alternative to the pull-back method of setting well screens. The screen is set in open hole, drilled below the well casing by rotary method after casing is cemented in its permanent position (after Anon[10])

6.8.4 Provision of a well base

If the bottom of the hole is in a soft material such as sand or clay, the well should be provided with an artificial base for the casing and screen[2]. This can be achieved by over-drilling the hole by about 1 to 2 m and filling the extra depth with concrete or gravel. This may help eliminate the possibility of subsidence of the well structure.

6.8.5 Placement of the gravel pack

The gravel pack must be placed only against the aquifer. It should never extend to the ground surface as it would then provide a highly permeable conduit that would allow free passage of pollutants from the surface strata to the aquifer.

The gravel pack material is usually placed through two tremie pipes on opposite sides of the hole. The size of tremie that can be used depends upon the size of the annular space, which should be at least sufficient to allow the insertion of a pipe with an internal diameter of 50 mm. The tremie pipe should be lowered to within 2 m of the bottom of the hole and slowly withdrawn as the gravel is placed. The gravel should not be allowed to fall more than 2 m below the bottom of the tremie as this would cause segregation.

A 50 mm tremie pipe is about the smallest that can be used, and under some circumstances a pipe of larger diameter may be required[2]. As a rule, the inside diameter of the tremie pipe should be at least 12 times the diameter of the coarsest fraction of the

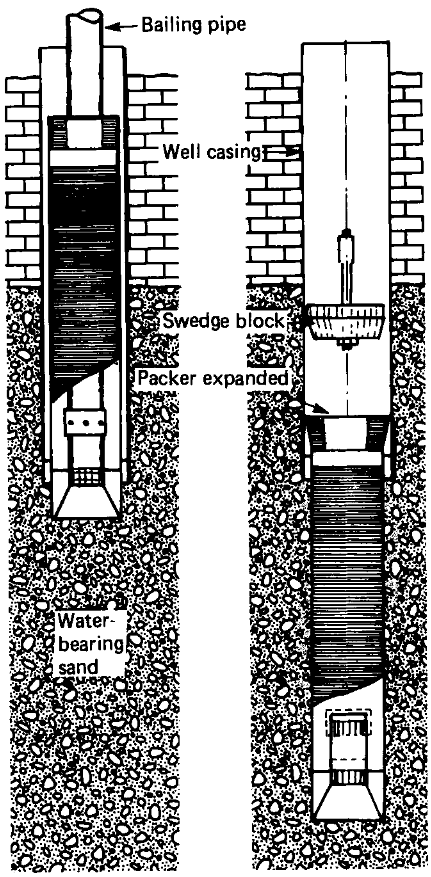


Figure 6.22 The bail down method of setting well screens. Assembly of well screen and bail down fittings at start of operation (left) and final step in completing the screen installation (right) (after Anon[9])

pack material to allow effective placement. This must be kept in mind when deciding whether to specify a gravel pack and when calculating the size of the hole and casing.

6.9 Well development or stimulation

Well development or stimulation has been defined by Koenig[17] as 'treatment of a well by mechanical, chemical, or other means for the purpose of removing an underground resistance to flow'. The term 'well development' is somewhat ambiguous and care should be taken not to confuse it with groundwater or aquifer development—the general process of drilling a well to make an aquifer available for water supply.

Development usually takes place as soon as the construction of the well is complete and before it is put into supply, the general objectives being to repair any damage to the aquifer caused by the construction process and to obtain maximum production efficiency. If, however, the performance of the well deteriorates over a period of time, the development process may be repeated, in which case the procedure is usually termed redevelopment, rejuvenation, rehabilitation or reconditioning.

Development is generally very cost effective and should therefore be regarded as an essential part of the well construction process. As mentioned above, the purpose of development is to optimize the performance of the well[4,9,17,18], or more specifically to

1. Correct any damage to the aquifer that has occurred as a result of the construction process. For example, the cable-tool method of drilling may cause a compacted annulus around the well, while rotary methods may mud-off the aquifer. This is sometimes referred to as the 'skin effect', since the construction process results in a 'skin' of denser than normal material being formed at the hole-aquifer interface that partially seals off the well from the aquifer. One aim of development is to eliminate the skin effect and allow the free flow of water into the well.
2. Remove the finer material from the aquifer, thus enlarging the flow passages in the formation and increasing its porosity so that water can enter the well more freely. The development of an aquifer by removing the fine material is sometimes called natural development, as opposed to artificial development which tries to increase flow to a well by providing an artificial gravel pack around the hole (see Section 6.10).
3. Stabilize the aquifer around the screens. Material which is brought to stability under high development velocities will remain stable under the velocities experienced during normal pumping operations, so the well should yield water that is free from sediment.
4. Obtain the maximum discharge from the well for a given drawdown, that is, to obtain the maximum specific capacity of the well.

Some form of development work always follows the construction of a well in unconsolidated deposits, even if it is just overpumping, but the application of development techniques to wells in hard rock is not so widely practiced. Nevertheless, development of hard rock wells should also be regarded as essential, since the construction process tends to force a rock paste mixture into the fractures and pores of the formation and cause a reduction in permeability. The application of development techniques will remove the rock paste and re-open fissures in the aquifer and improve yields. Any of the techniques that can be applied to unconsolidated aquifers can also be applied to hard rock aquifers, but there are a few additional techniques (blasting, acidizing and hydraulic fracturing) that are applied only to rock formations. Campbell and Lehr[4] listed five major categories of development technique, which in order of decreasing familiarity are: surging (any method that forces water back and forth through the formation), jetting, blasting, acidizing, and hydraulic fracturing (see *Table 6.7*). While the application of these techniques can produce very beneficial results, it should be pointed out that over-zealous application can damage the well or destabilize the aquifer. Consequently the development process should start gently and gradually increase in intensity.

6.9.1 Development techniques for unconsolidated formations

There are a variety of techniques that can be applied to unconsolidated deposits which come under the general headings of surging and jetting. The usual aim is to cause a reversal of flow through the well screen and surrounding formation that will break down sand bridging. Sand bridging is the name given to the situation where particles are packed together and held in place by the constant pulling action of the pump. Reversal of flow destabilizes the sand bridge and aids the development of the aquifer. The methods that are commonly used are as follows

1. *Overpumping* Overpumping consists of pumping the well at a discharge above that for which it was designed. This produces higher than normal velocities which remove

TABLE 6.7. Comparison of methods used for the development of wells

Method	*Most suitable for*	*Effective range*	*Advantages*	*Disadvantages*
Overpumping	Unconsolidated formations. Thin, relatively uniform, permeable aquifers	Most effective over upper 25–50 per cent of the screen length	Simple, cheap, easy	Limited effective range. Does not overcome bridging by itself. Requires a pump with a capacity greater than the design discharge Sand pumping may damage the pump
Rawhiding	Unconsolidated formations. Suitable for most situations. Often finish off with this technique when other methods used	Upper portion of screen	Simple, cheap, easy. More effective than straight-forward overpumping	Limited effective range. May not completely eliminate bridging. Pump must allow reverse flow. Also as above
Surge block or plunger	Unconsolidated formations. Suitable for most situations, but not effective with gravel-packed wells. Most often used with cable tool drilling rigs. Widely used	Whole length of screen depending upon technique adopted	Simple, cheap, relatively easy. A surge block can be made out of simple materials. Very effective	May not be easy to operate with a rotary rig. may be undesirable if the aquifer has a clay fraction (see text)
Compressed air	Unconsolidated formations. Suitable for most situations	Whole length of screen	Quite effective. Relatively cheap	Require skill and experience. May be difficult in deep wells with a considerable depth of water depending upon available compressor capacity
Hydraulic jetting	Any formation. Suitable for most situations including open rock holes and wells with gravel packs	Whole length of screen	Relatively simple, cheap, easy and effective	Not effective with screens that have a small open area
Dispersant chemicals	Any formation. Wells drilled by rotary methods or aquifers having a clay fraction	Generally used in conjunction with jetting or backwashing	Easy to use	Not generally used by themselves
Blasting or shooting	Consolidated formations. Commonly sandstones, because acidizing not effective in sandstone. Also limestones, etc	Effective range depends upon positioning of explosive. Can be the entire hole	Simple in theory. Can be very effective. Also cleans well face	Requires skilled, experienced personnel. Outcome may be unpredictable
Acidizing	Consolidated formations. Usually limestones because they are most susceptible to acid attack, particularly if fractured	Effective over submerged part of hole and surrounding aquifer	Can be very effective. Also cleans well face or screens	Requires skilled, experienced personnel. Dangerous. Outcome may be disappointing. Can be very expensive. Time-consuming
Hydraulic fracturing	Any consolidated formation	Entire hole length, and surrounding aquifer	Can be very effective	Requires skilled, experienced personnel. Can be expensive and time-consuming

material that would otherwise stay in place, but it does not eliminate sand bridging. However, bridging may be overcome if the pump is switched off periodically and the water is allowed to run back into the aquifer.

2. *'Rawhiding' or backwashing* Rawhiding refers to the process of pumping the well at rates that increase progressively from about one-quarter to twice the design capacity, with the pump being switched off at the end of each step. This method is similar, but superior, to overpumping. Rawhiding is often used to finish of development work after some other technique has been used.

3. *Surge block* The surge block or plunger is one of the oldest and most effective methods of well development and is usually used with a cable-tool rig. In a simple form a surge block consists of two or more rubber or leather rings of the same diameter as the well casing sandwiched between slightly smaller wooden rings (*Figure 6.23*). The plunger is moved rapidly up and down above the screen perforations, and as it rises it draws water into the well, while lowering it forces water into the aquifer. This fairly rapid flow reversal overcomes sand bridging and brings sediment into the well. The depth of sediment accumulated at the bottom of the screens can be measured periodically to give an indication of the effectiveness of the surging action, after which it should be removed by bailing. The surge block may incorporate a simple valve to allow water to pass through it on the downstroke so that the action is gentler and dirty water is gradually removed as surging continues.

The surge block may also be used inside the screen itself if the solid block is made about 50 mm smaller in diameter than the screen while the rubber or leather discs are about the same diameter as the screen. The block need not be a tight fit in the screen and in fact it may be advantageous to ensure that it is not to avoid the possibility of the block becoming sand locked or causing damage to the screen. Using this technique the entire screen length can be developed.

Surging is a very effective method, particularly if the block is weighted (often with a

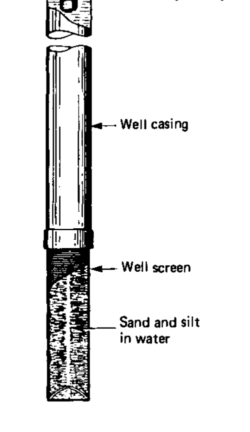


Figure 6.23 Well development with a surge block or plunger. The surge plunger is an effective tool for well development. It is particularly suitable for use with cable-tool equipment—downstroke forces water outwards into granular formation; upstroke pulls in water, silt and fines through the well screen openings (after Anon[9,18])

drill stem) to give extra force to the downstroke. However, if the aquifer contains clay particles some drillers feel that the use of a surge block may smear clay over the screens and cause partial blockage, high differential pressures and possibly collapse of the hole.
4. *Air* Compressed air at about 700 to 1000 kPa may be used as an effective development tool, but this requires considerable skill and experience. The method consists of installing an air pipe inside an eductor or delivery pipe inside the well. For the best results about 60 per cent of the total length of the air pipe should be below water at any particular time when the well is being pumped. To start the development process the air pipe would be positioned near the bottom of the well and below the delivery pipe (see *Figure 6.24*). The basic idea is to achieve a sudden release of air into the well that will force its way into the surrounding aquifer and loosen grains. This will then be followed by a rush of water into the well as the air bubble contracts, creating a surging action. At this stage, the air pipe is pulled back inside the delivery pipe and operates as a conventional air lift until the water discharged is relatively sediment free. The air lift assembly is then raised by about 1 m and the procedure repeated until the whole length of the screen has been treated.
5. *Hydraulic jetting* Hydraulic jetting is a very effective method provided that the well construction enables the jet to be played over a large percentage area of the hole-screen interface. Thus, jetting is effective with continuous slot and some louvre type screens (or open holes in rock) but not effective with perforated pipe where the openings are only a small percentage of the total surface area (see *Figure 6.25*). The effectiveness of the technique stems from the fact that the energy is concentrated over a relatively small area and can be applied selectively until the whole length of the screen has been covered.

The equipment consists of a relatively simple jetting tool and piping, together with a high pressure pump capable of developing about 700 to 1000 kPa. The nozzles on the

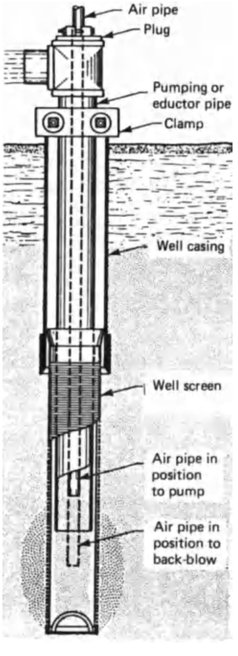


Figure 6.24 Well development using compressed air. A surging action can be produced with compressed air using modified air-lift (after Anon[10,18])

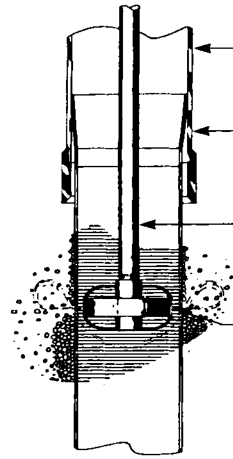


Figure 6.25 Well development using hydraulic jetting. The diagram shows the operating principle of the high velocity water jet for vigorous agitation to remove drilling mud, silt and fine sand in developing a well (after Anon[9,18])

jetting head should be arranged uniformly around the tool and have a diameter of between 5 and 13 mm, in which case the jet will have a nozzle velocity of about 50 m/s or more. The jetting tool should be rotated at approximately 1 rev/min and held at a particular depth for about 2 min, then raised by half the screen diameter and the process repeated until all the open hole has been covered. Because the jet picks up fine sediment and has a high velocity, it has an abrasive action that could cut or damage the screen should it be directed against one small area for too long.

The washing action of the high velocity water jet operating through the screen openings agitates and re-arranges the sand and gravel particles that surround the screen. This successfully removes any drilling mud caked on the walls of the hole, while fines are washed out of the formation. The turbulence caused by the jet tends to re-circulate the fines back into the well, so the best results are obtained if the turbid water is pumped or air lifted from the well at a higher rate than water is being jetted into the formation. This causes a net movement of water out of the well and also allows the effectiveness of the development process to be assessed. This technique is also effective with gravel packed wells (see below).

6. *Dispersion agents* Numerous chemicals are used to act as deflocculants and dispersants of clay and silt. Their function is to help break up mud and cake on the walls of a hole and to aid the removal of the clay fraction from an aquifer. These chemicals are sometimes added to the water used for jetting or backwashing, or added to water that is allowed to stand in the well over a period of time. The chemicals most commonly used are the polyphosphates such as sodium tripolyphosphate ($Na_5P_3O_{10}$), sodium pyrophosphate (NaP_2O_7), tetrasodium pyrophosphate ($Na_4P_2O_7$) and sodium hexametaphosphate $(NaPO_2)_6$.

6.9.2 Development of gravel-packed wells in unconsolidated formations

If a well has a gravel pack, the natural development process is considerably hindered. The reason for this is that the permeability of the gravel is many times higher than that of the aquifer, so instead of development forcing water into and out of the aquifer, it merely forces water to move vertically up and down in the gravel pack instead. Therefore the effective development of a gravel packed well can be difficult to achieve.

In general, the thicker the pack the more difficult it is to develop. Perhaps the most effective way of developing a gravel-packed well is to use the jetting technique, possibly in conjunction with dispersant chemicals and rawhiding.

6.9.3 Development of wells in hard rock aquifers

Many of the techniques described above can also be applied effectively to rock aquifers, but there are development methods that have been perfected specifically for this purpose. In general these techniques tend to be more complex and more expensive to implement than their counterparts in unconsolidated formations, although they also tend to be more effective[4,19].

1. *Blasting or shooting* Blasting or shooting involves detonating explosives in the well[20]. The size of the charges and pattern of firing may vary depending upon the type of explosive used, the nature of the well, the type of rock and the depth of water in the well. The outcome is often somewhat unpredictable. Nevertheless, the explosive gases, under very high pressure, cause expansion in the pore spaces and discontinuities of the aquifer, which is followed by a subsequent contraction. The expansion and contraction produces a strong surging action. The explosion also causes a vibratory effect.

Blasting may gain some of its effectiveness by increasing the diameter of the well, while deposits on the face of the well are removed. Evidence suggests that the specific capacity of newly constructed wells in sandstones may be increased by about 20 per cent by blasting as a result of removing fine material from the well face. Explosives may also be used to rehabilitate old wells that have become clogged, often as a result of deposits of calcium carbonate on the well face. It is claimed that the yields of many clogged wells have been restored to original values through blasting, while the specific capacities of rehabilitated wells are sometimes three times those before blasting.

2. *Acidizing* The process of acidizing consists of placing a non-toxic acid solution in contact with the water-bearing formation[4,19,21]. The parts of the aquifer which are soluble in acid are dissolved, which results in an increased flow of water to the well. The effectiveness of the method is due to the fact that discontinuities in the rock are enlarged and thereby are able to sustain a higher rate of flow. It is not due to any significant increase in the diameter of the well itself. This technique is most commonly used in carbonate rocks, since these are the most susceptible to acid attack. The acid used is normally muriatic acid, a commercial grade of hydrochloric acid which contains a small percentage of impurities, with an inhibiting agent to delay attack on the metal parts of the well. To develop a well by acidizing, the pump and discharge column are usually removed and between 0.5 and 20 m^3 of acid introduced through a temporary pipe extending to near the bottom of the well. The solution is allowed to stand in the well until the acid is spent, which may take anything between 30 min and four days (but generally averaging about one day) depending upon the size of the well and the amount of acid used. The pump can then be re-installed and the spent acid removed from the well by pumping, typically for between 1 and 8 h. To finish the development process the well should then be treated with polyphosphates accompanied by surging or jetting.

Pressure acidizing[4] is an advanced method of well development borrowed from the crude oil industry. The technique is similar to that described above and again would typically employ a 15 per cent solution of muriatic acid. This time, however, a large volume of acid would be pumped into the aquifer at a high pressure, producing deep radial penetration. By this method it is hoped to treat a much larger volume of the formation and to obtain a greater increase of flow than would be possible using the conventional acidizing technique.

The effectiveness of acidizing in increasing the yields of wells depends upon the type of formation and the nature of the permeability[22,23,24]. In a fairly massive carbonate formation the technique may not be particularly effective, while increases of around 50 per cent may be obtained with wells of initially moderate to high capacity constructed in rock masses which are jointed. According to Hargis and McCauley[22], acidization is beneficial for wells that can produce more than 2700 m^3/day, but is of marginal value for wells producing less than this amount. If a dolostone or limestone aquifer has an extensive discontinuity system that has become clogged, then yields may be increased by acidizing by anything between 50 and 500 per cent, or even more. Campbell and Lehr[4] claimed that acidizing improves the yields of about 80 per cent of the wells treated (including new holes) typically by about 50 to 150 per cent. However, there are instances when acidizing proves to be very ineffective. This can be partially explained by the fact that hydrochloric acids cause certain silicate minerals to expand to five times their original size, which could cause plugging of the aquifer and offset the effects of acidizing. If silicate swelling is likely to be a problem, then silicate-controlling agents, whose function is to minimize formation swelling, should be added to the acid. Acidizing is extremely expensive, indeed it may cost as much as the construction of the well itself.

3. *Hydraulic fracturing* Hydraulic fracturing is the most advanced method of well development[2,4,25]. The technique consists of isolating 2 to 3 m lengths of the hole using inflatable packers and then filling the isolated section with water via a pipe leading to the surface. A pump is then used to pressurize the water with the result that the aquifer is actually fractured. If desired, sand or special beads can then be injected into the cracks to prevent them from closing when the water pressure is removed. This is called 'sand fracing'. Hydraulic fracturing may increase the permeability of the aquifer for perhaps 100 m or so around the hole. Some wells have shown an increase in yield of as much as 200 per cent as a result of fracturing, but in all cases the initial yield was small.

6.9.4 The effectiveness of natural well development

It may sometimes be difficult to assess the effectiveness of development because the wells may not be pump tested before the treatment, so there is nothing to compare the post-treatment yield with. Rather more data tend to exist on the effectiveness of rehabilitation. Nevertheless, it is probably true to say that development of a well immediately after construction will significantly increase its yield, perhaps by as much as 600 per cent or more if the aquifer has been mudded off by the construction process[9]. Koenig[17] analysed about 900 development cases and found that

1. For all types of treatment in all types of formations, the median ratio indicated a 20 per cent improvement over the original production of the well.
2. For all types of treatment in all types of formations, the median ratio indicated a 97 per cent improvement over specific capacity immediately before treatment.
3. Failures to achieve improvement over the original well were 43 per cent.
4. Failure to achieve any improvement over the treated well was 11 per cent.
5. Based on improvement in the well before treatment, consolidated formations showed a median of 141 per cent improvement, whereas unconsolidated formations showed only 45 per cent improvement.
6. The median cost ratios for various methods (expressed as a multiple of the cheapest)

are in order of increasing effectiveness

Surging	1.0
Blasting	4.7
Vibratory explosion	1.7
Pressure acidizing	2.5
Fracturing	3.3

To summarize, development can be very effective, but fracturing and pressure acidizing often show a unit cost as high as that of the original production well.

6.10. Artificially developed wells—gravel packing

The aim of a naturally developed well in an unconsolidated aquifer is to create a cylindrical zone of relatively coarse material around the screen by removing the finer fraction during the development process (*Figure 6.26(a)*). The coarse material has a higher permeability than the rest of the formation and thus flow to the well is made easier in the region adjacent to the screens where the loss of head is greatest. This decreases the head loss (or drawdown) and increases the efficiency of the well. The same effect can be achieved by drilling an oversize hole and filling the annulus between the screens and the well wall with a suitable graded gravel. The only difference between a naturally and artificially developed well, apart from the method used to obtain a permeable envelope around the well, is that with a naturally developed well the particle size around the screens will get gradually smaller with distance into the aquifer. With an artificial pack there will be an abrupt change in particle size between the pack and the natural formation material (*Figure 6.26(b)*). For this reason it is important that the gravel pack has the correct grading and does not allow the migration of fines into the pack. If too coarse a gravel pack is used in a fine sand, grains will move into the pack and fill the voids (*Figure 6.26(c)*). This decreases the permeability of the pack and increases the entrance velocity through the screens. The higher velocities mean that more fines are drawn into the well, the end result being either clogging or sand pumping[18].

To be effective the gravel pack mixture must be size graded so that it will stabilize the formation and also permit effective development of the formation–gravel pack interface. One suggested rule[13] is to make the grading of the 50 per cent particle size of the gravel pack 4 or 5 times the 50 per cent size of the formation material. If the ratio is above 10, the aquifer material migrates into the pack, but if it is below 4, the well may not be fully developed. Usually the gravel pack material has a fairly uniform grading in the range between coarse sand and 5 mm rounded (pea) gravel. To make effective development possible, the gravel pack should not be thicker than 200 mm and not thinner than about 35 to 70 mm, while the screens should be of the continuous slot type, if possible, since these are more conducive to the development process[2,9,11]. The screen slot size should be chosen so that it will hold back not less than 85 per cent of the gravel pack mixture.

Gravel packed wells are most effective in fine-sand aquifers where the use of an artificial pack around the screen stabilizes the formation and increases the permeability near the intake. This permits the use of a larger size of screen slot opening. Gravel packs are also used in some rock aquifers. This may be necessary if the aquifer consists of a weakly cemented sedimentary rock that may be disaggregated by the flow of water to the well, or if the rock mass is highly fragmented. However, the use of a gravel pack can

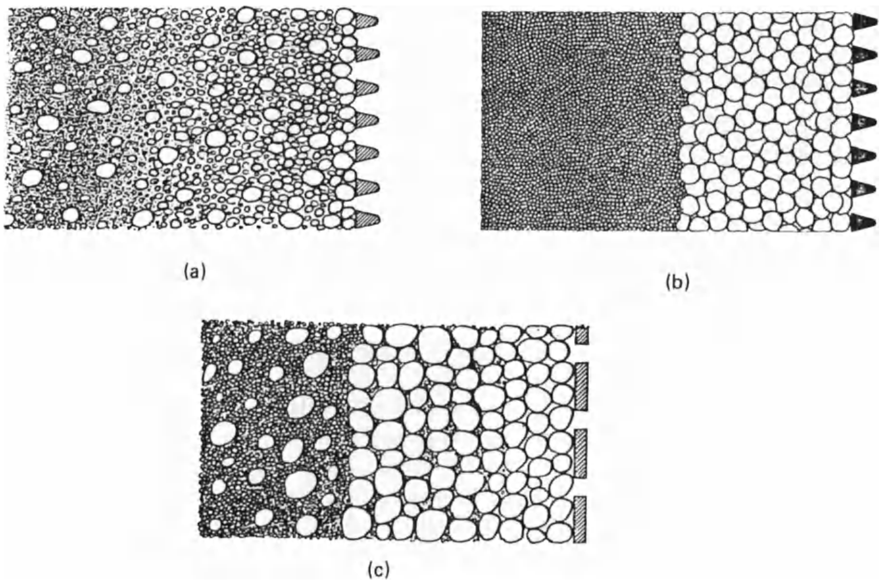


Figure 6.26 The basic difference between the arrangement of the sand and gravel in natural and artificial gravel packed wells. (a) The principle of the natural or 'developed' well with each zone correctly graded to the next so that the whole pack is stabilized. (b) An artificial gravel packed well in which the correct size relationship is established between the size and thickness of the gravel pack material and the screen slot width. Such a well can be effectively developed and will be efficient and stable. (c) Undesirable result of using gravel that is too coarse. The aquifer sand is not stabilized and will eventually migrate into the well. This unstable condition will persist regardless of how thick the gravel pack may be, thus causing a continued threat of sand pumping (after Anon[18])

increase construction and maintenance costs, in addition to complicating the effective development of the well (see Section 6.9.2). Consequently gravel packs should be used only when there are good reasons for doing so, and not indiscriminately.

6.11 Well sterilization

It is good practice to periodically disinfect a well during the drilling operation, while any water that is introduced into the well during drilling or well development should be of drinking water quality[2,4]. Similarly, gravel pack material should be disinfected before being placed in the well. A well provides a direct connection between the ground surface and the groundwater resource and care must be taken to ensure that pathogenic bacteria are not allowed to enter the aquifer during the construction process. The final step in well completion is to remove as thoroughly as possible any foreign substances such as soil, grease or oil and then to disinfect. Of course, many of the organisms that may be found in the well are not harmful, but they can accelerate and aggravate corrosion and encrustation problems thereby reducing the life of a well. Although disinfection may not totally eliminate these problems, it is an inexpensive precaution that is likely to be cost effective.

Disinfection or sterilization is usually accomplished by introducing chlorine into the water in the well and the adjoining aquifer. One practical method of chlorination is to place dry calcium hypochlorite ($Ca(ClO)_2$) in a container made from a short length of perforated tubing capped at both ends and suspended by a cable. By moving the container up and down through the full column of water in the well, the chemical is evenly dispersed.

All new wells, or existing wells that have been subjected to repair or development, should be sterilized before being put into supply.

6.12 Other types of well construction and practice

Although the techniques described earlier in this Chapter can be used to construct a water well under almost any conditions, there are situations where an alternative method (with a more limited range of application) may be more suitable. A few such methods will be considered briefly in this section.

6.12.1 Bored wells

Where a water table exists in an aquifer of unconsolicated material, it may be possible to construct a well quite cheaply by boring a hole with an auger[12]. An auger can range in size from a small hand-powered tool of about 150 mm diameter capable of reaching depths of up to 15 m, to large wagon or crane mounted power driven augers of up to 2.0 m in diameter which can go as deep as 40 m or more (*Figure 6.27*). The auger is screwed into the ground until the bucket or thread is full, then it is brought to the surface and emptied. This can be achieved with power driven tools simply by rotating the bit. Augers are ideal for boring large diameter holes through clays and therefore may be used in conjunction with another drilling method such as the percussion or reverse rotary. If a hole has to be bored through ground which is not self-supporting or below the water table, it will probably be necessary to insert a casing to the bottom of the hole and complete the bore by drilling through the casing.

6.12.2 Wellpoints

If the water table is within 3 to 5 m of the ground surface, it may be possible to construct a small inexpensive well using a wellpoint technique[12,16]. A wellpoint consists of a section of screen, very like or the same as the screens used in larger wells, with a point on the end (*Figure 6.28*). The screen length can be extended by the use of an extension wellpoint, or simply connected to the surface with lengths of blank pipe. Wellpoints usually have diameters of around 50 mm and are sunk to depths of up to 15 m, but occasionally may be set below 30 m. Because of their small diameter a surface-located suction pump must be used to extract water from the well. Typically, wellpoints may yield 120 to 400 m^3/day, which makes them suitable for domestic use or for dewatering construction sites[26].

One advantage of the wellpoint is the speed and relative ease with which they can be constructed. There are two methods that are frequently used to emplace wellpoints. The first consists of driving the wellpoint down with any suitable type of hammer, which can range from a sledgehammer to the type used for driving piles and adding lengths of blank pipe until the desired depth is reached. The second method consists of hydraulic jetting, in which the point of the screen section contains a floating ball valve that allows water to be discharged out of the drive point, but closes when the flow is reversed. The flow of water out of the wellpoint fluidizes the formation so that the well is

Figure 6.27 Crane mounted earth auger capable of boring a 2 m diameter hole to about 44 m depth. Larger and deeper holes can be obtained by modifying the rig. Similar rigs can be wagon mounted. (Courtesy BSP International Foundations Ltd)

slowly jetted or washed into the required position. If the wellpoint meets resistance, such as a dense stratum, it may have to be driven.

6.12.3 Adits

An adit is a tunnel about 1.4 m wide and 2.0 m high located below the water table. The purpose of an adit is to intersect as many discontinuities as possible, thereby increasing the yield of a well[13,14] (see *Figure 6.29*). The construction of an adit, even in self-

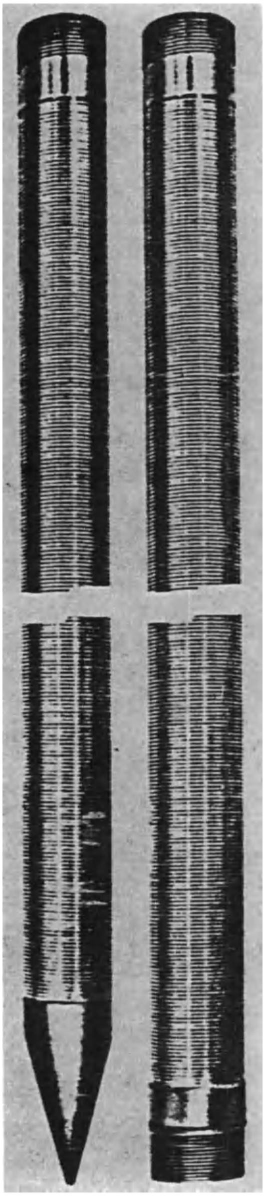


Figure 6.28 Drive wellpoint in stainless steel (after Anon[16])

supporting strata, is a costly and difficult undertaking because the work must take place below the water table. Adits are quite common in the Chalk of southeast England. Experience indicates that it is difficult to forecast reliably the yield of an adit from a consideration of its length.

Adits are of value in situations where the drawdown at a well is limited, perhaps because the aquifer is very thin, the water level in the lower part of the formation is of unacceptable quality, or because the well is near the coast and increased drawdown would cause saline intrusion.

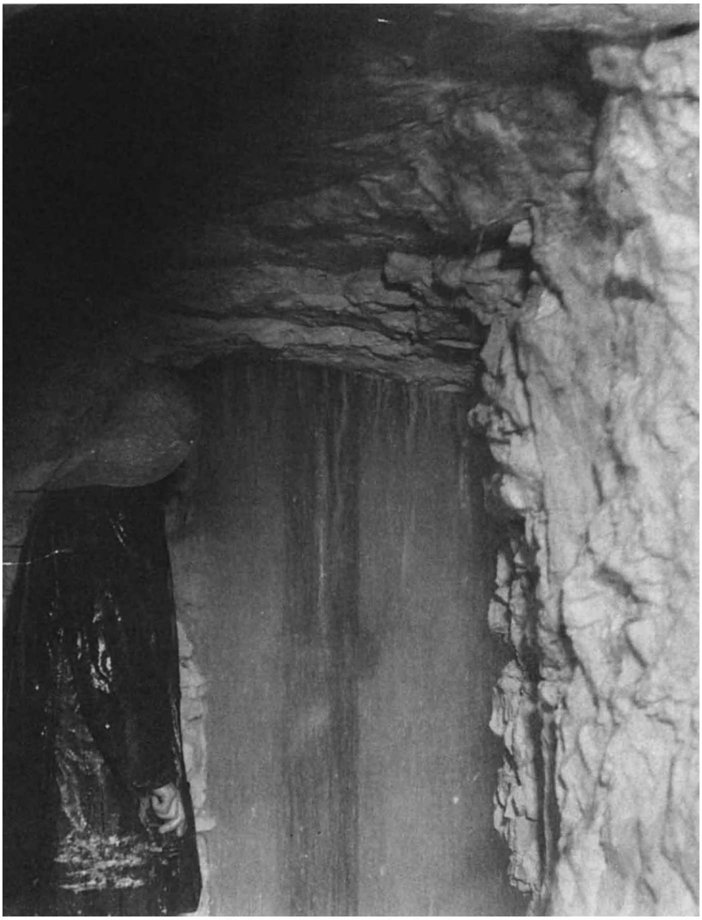


Figure 6.29 Water from a fissure entering an adit in the Chalk of southeast England. (Courtesy G. Stow & Co. Ltd, Henley on Thames,UK)

6.12.4 Radial collector wells

If the water table is lowered beneath a body of surface water, the rate of recharge is increased as a result of induced infiltration[27,28,29,30]. This principle has been used in Europe, America and other parts of the world where sources of water are scarce, to increase the yield of wells or infiltration galleries. The collector well[4,5,12,29], introduced by Ranney in 1933, is a more modern form of infiltration gallery designed specifically to exploit recharge from a surface source (*Figure 6.30*). Such wells consist of a central caisson of 4 m or more internal diameter typically sunk to a depth of between 20 and 40 m. A series of radiating screen pipes or 'laterals' of between 0.2 and 0.6 m in diameter are jacked out from near the bottom of the caisson, generally towards the source of surface recharge. Depending upon well design and aquifer characteristics, the yield of a Ranney collector well may range between 3800 and 114 500 m^3/day, with an average of approximately 27 250 m^3/day. The yield of collector wells removed from a source of surface recharge average around 15 260 m^3/day which is still quite high. The

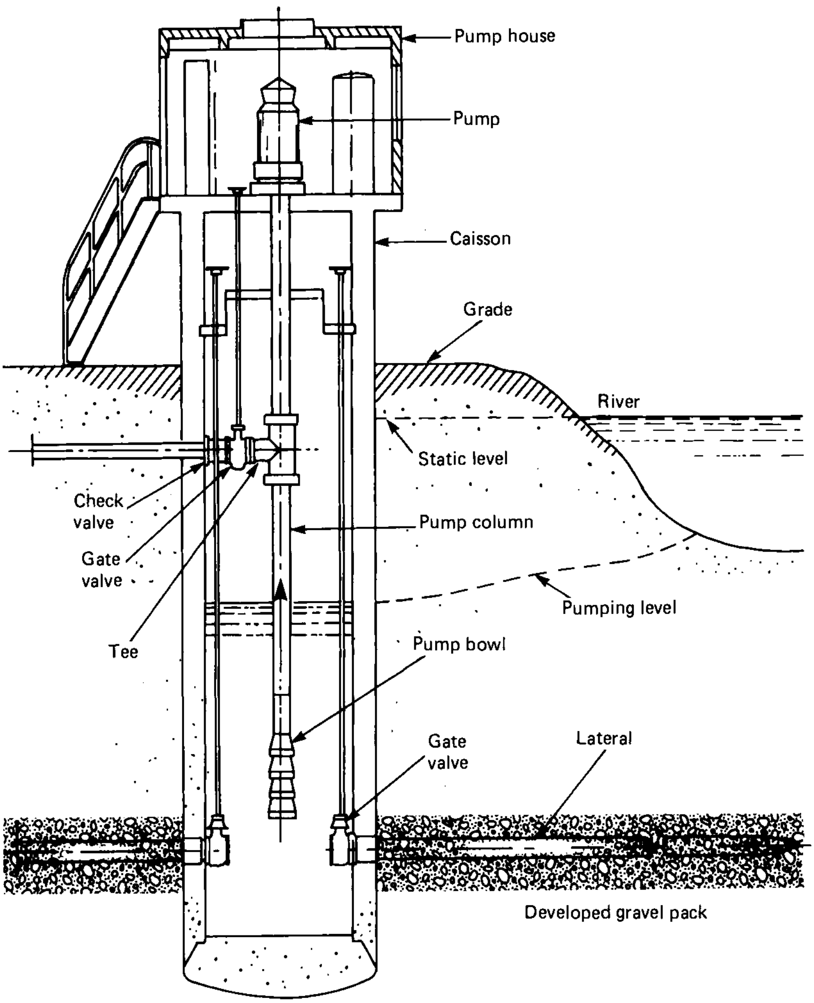


Figure 6.30 Section through the Ranney Radial Collector Well (after Campbell and Lehr[4])

cost of constructing a radial collector well is greater than that of a normal hole, but the cost of water produced may compare favourably with a comparable well field. Hundreds of collector wells have been built in many different countries and it is claimed that they offer many advantages such as increased pump efficiency, low maintenance costs, ease of operation and large yield. However, they also have some disadvantages including difficulty in well development and increased potential for problems connected with corrosion or plugging of laterals.

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Chapter 7

Aquifer hydraulics and pumping tests

7.1 Introduction

When considering a groundwater development project, one of the biggest problems to be encountered is often the lack of data upon which to base an assessment of the viability of the aquifer. A common problem, according to Rushton[1], is the scarcity of data relating to the variations in the value of the coefficients of transmissivity and storage. Knowledge relating to the position and nature of the boundaries and recharge–discharge mechanisms of the aquifer may also be inadequate. For example, an aquifer may (or may not) be hydraulically connected to a surface water resource such as a river. If it is, the yield of the aquifer may be determined not by the amount of water that can be extracted perennially, but by the maximum abstraction that will not reduce the flow in the river below a minimum acceptable value as determined by fishing or amenity interests. Thus, information regarding the recharge and discharge mechanisms may be essential to the efficient management of the aquifer and of the water resources of the region in general.

Although it is possible to obtain some approximate idea of the perennial yield of an aquifer from a desk study employing the methods described in Chapter 4, detailed information can be obtained only from a field pumping test. A pumping test, in essence, involves abstracting water from the well at a known rate and then observing the decline in water level in the aquifer in the vicinity of the well. These data can then be analysed to determine the hydraulic characteristics of the aquifer (see Section 7.4). However, it should be appreciated from the beginning that such tests are both capital and labour intensive, requiring several boreholes, many operatives, and a considerable amount of equipment. Consequently, a pumping test should only be undertaken towards the final stages of a groundwater investigation when the chances are that this investment will not be wasted.

As an illustration of the sort of commitment that a pumping test involves, some 'short' tests described in the literature lasted for three weeks but still failed to produce the desired information regarding the effects of abstraction on the aquifer. Some pilot schemes lasted several months, while other investigations have incorporated pumping programmes spread over a period of many years[2]. The intention of such prolonged tests is to evaluate thoroughly the consequences of abstracting water from an aquifer and to simulate operational conditions, but this does not imply that all pumping tests are between 3 weeks and 3 years in duration; some useful information can be obtained from tests lasting less than 3 h. The length of test required depends upon the nature of the aquifer and the specific objectives of the test.

After a water supply well has been completed it is normal practice to conduct some form of pumping test, to accomplish one or more of the objectives listed in Section 7.5.1. In order to carry out a successful test it is necessary to have some knowledge of aquifer and well hydraulics and in particular how the drawdown varies with the duration of pumping and distance from the pumped well. Without an appreciation of these relationships and the factors which affect them, it may prove difficult (or impossible) to design a suitable observation hole network and to conduct a meaningful test.

The term 'observation hole' is used here to represent, without distinction, any installation such as a cased/screened well, a drive point well screen, a piezometer, or an uncased or open hole, that is used solely to observe water levels and which is non-productive during the test. The term 'well' can be taken to represent the discharging well that is under test.

7.2 Aquifer hydraulics

Aquifers have two important functions, namely the transmission and the storage of water (described quantitatively by the coefficients of transmissivity and storage, T and S, respectively). The yield of a well and the shape and size of the cone of depression, is largely determined by the magnitude of these two coefficients.

When water is first withdrawn from a well a small cone of depression is formed, which quickly increases in size as more and more of the aquifer is desaturated to satisfy the demand of the pump. During the early stages of pumping, most of the water that is abstracted is obtained from storage, the greater the storativity of the aquifer the greater the amount of water obtained per cubic metre of desaturated material. Thus, high storativity results in a relatively small cone of depression, while low storativity results in a larger cone. As the cone spreads and deepens a greater volume of water is released for each unit increase in diameter, so the rate of growth decreases with time. The depth of the cone may be particularly slow to increase, as illustrated by the example below[3].

Duration of pumping (h)	*Volume of cone of depression* (V)	*Radius of cone depression* (m)	*Drawdown at the well* (m)
1	1	122	1.83
2	2	174	1.92
3	3	213	1.98

The relatively small rate of increase in the depth of the cone can cause inexperienced observers to conclude mistakenly that a constant drawdown has been achieved, when in fact, the cone is still slowly expanding. In the latter stages of a pumping test, small increases in drawdown can be obscured quite easily by fluctuations in water level caused by other factors (see Section 7.7).

As the cone of depression increases in size it will intercept progressively more and more of the flow through the aquifer, with only the remaining fraction of the required discharge being obtained from storage. Eventually, the cone will reach such a size that the quantity of flow intercepted equals the discharge, so there will be no further depletion of storage. However, it should be remembered that the water intercepted by the cone may be derived not only from natural aquifer discharge but also from a surface water resource, or vertical recharge as a result of precipitation or leakage through underlying or overlying formations. When the cone of depression has stabilized, a condition of equilibrium is said to exist. The period of pumping required to achieve equilibrium varies from hours to years according to the nature and type of the aquifer. Consequently it is important that the type of aquifer under investigation is identified before a pumping test begins, since the various types respond differently to pumping.

7.2.1 Variation of drawdown with time and type of aquifer

The two main types of aquifer, unconfined and confined, were described in Chapter 2. However, there are several intermediate categories that may be distinguished when conducting a pumping test, and it is essential that all of these can be recognized. The main types are

Unconfined aquifers

An unconfined aquifer is a permeable bed only part filled with water and overlying an impermeable layer (*Figure 7.1*). In fine-grained unconfined aquifers, gravity drainage of the pores is often not instantaneous, water is released only some time after the lowering of the water level. This condition is usually called an unconfined aquifer with delayed yield (for this reason, and because some water is retained, it can be misleading to say that the area of the aquifer within the cone of depression has been dewatered—desaturated may be a better term). Because unconfined aquifers generally have a high coefficient of storage, a long period of pumping may be required before the drawdown stabilizes. In some aquifers equilibrium is never achieved.

Semi-unconfined aquifers

If a layer of permeable material is covered by a fine-grained layer of partly saturated material which is relatively impermeable compared with the aquifer itself, but which has a significant permeability that is too large to be ignored, then such an aquifer is often referred to as semi-unconfined. These aquifers are intermediate between the unconfined and the semi-confined conditions. The delayed yield of the overlying layer can result in a two-phase time-drawdown curve, as shown in *Figure 7.1(b)*.

Semi-confined aquifers

A semi-confined or leaky aquifer is a completely saturated formation that is overlain by a partly saturated semi-permeable layer with either an impermeable or semi-permeable formation below it. The vague term 'semi-permeable' is used to describe a formation which has a low, but measurable, permeability. Lowering of the piezometric head in a semi-confined aquifer as a result of pumping will result in the vertical flow of water from the semi-permeable layer into the aquifer. This causes the cone of depression to stabilize quicker than would otherwise be the case. Horizontal flow in the confining layer can usually be neglected.

A semi-confined aquifer may exhibit both a piezometric and a phreatic surface, since the overlying material is not entirely impermeable. In such aquifers, when conducting a pumping test, it is advantageous to install piezometers in all the semi-permeable layers to detect the movement of water. In general the drawdown of the phreatic level in the

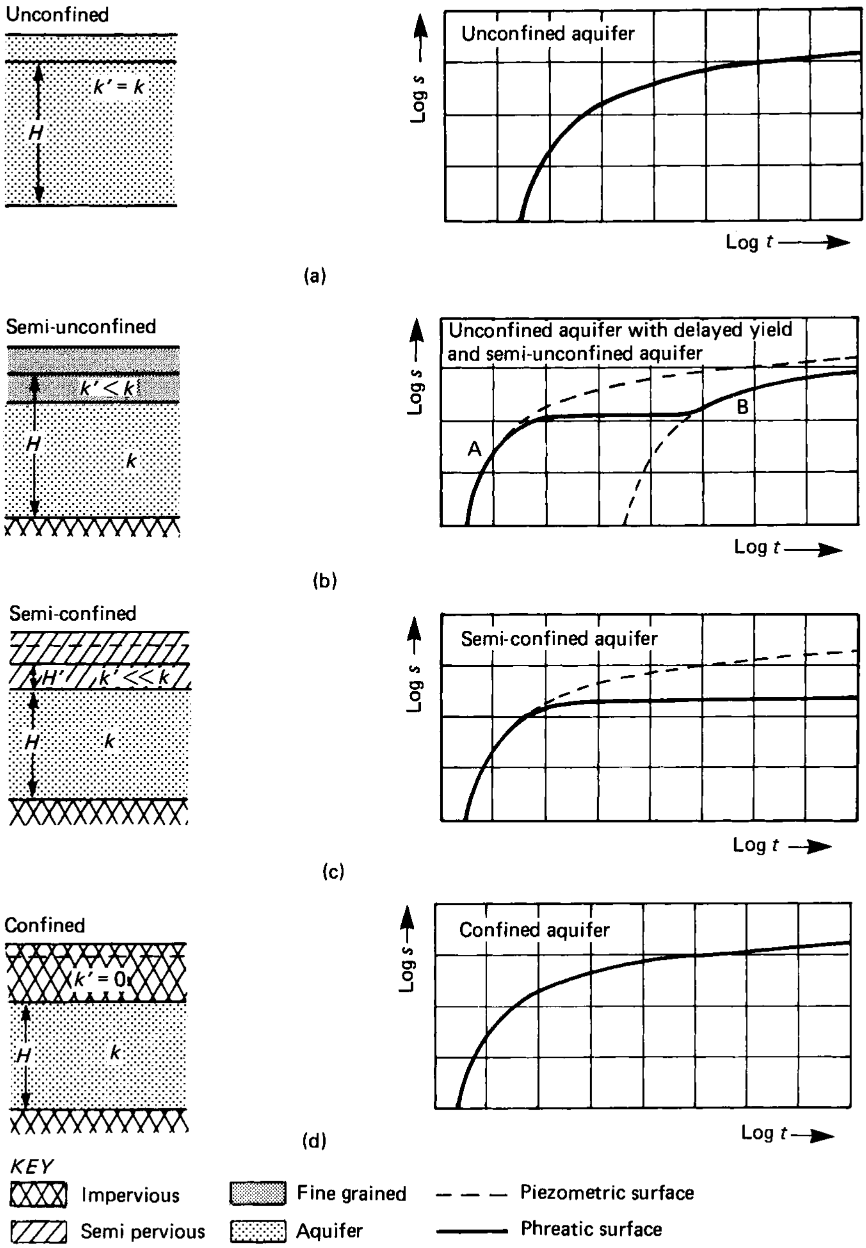

Figure 7.1 Relation between aquifer permeability, k, the permeability of the overlying formation, k', and the double-log type curve of drawdowns, s, against time, t (after Kruseman and de Ridder[4]).
(a) Unconfined aquifer and type curve. Unconfined aquifers generally respond relatively slowly to pumping and it may take months or years to achieve a constant drawdown. (b) Semi-unconfined aquifer and type curve. Curve A represents the type curve for the initial confined condition, curve B is the type curve for the later unconfined state and the line joining A to B is the result of delayed yield. (c) Semi-confined aquifer and type curve. The dotted line represents the type curve of a confined aquifer. Vertical recharge through the formation overlying a semi-confined aquifer results in the drawdown stabilizing earlier than would otherwise be the case. (d) Confined aquifer and type curve. Confined aquifers generally respond relatively quickly to pumping

semi-permeable layer is small compared with the lowering of the piezometric level of the aquifer (since the semi-permeable layer takes some considerable time to drain).

Confined aquifers

A confined aquifer is a completely saturated formation sandwiched between two impermeable layers. Truly confined aquifers are rare, since totally impermeable formations are uncommon. The water in a confined aquifer is usually under pressure so that the piezometric surface is above the top of the aquifer. A fall in the level of the piezometric surface actually represents a decrease in the pressure of the water in the aquifer. Since pressure changes can be transmitted quite quickly and because confined aquifers generally have relatively small coefficients of storage, they usually respond relatively rapidly to pumping.

The four types of aquifer described above can be summarized as follows[4].

Type of aquifer	*Covering layer*
Unconfined with delayed yield	Same as main part of the aquifer
Semi-unconfined	Less permeable than aquifer, but horizontal flow significant
Semi-confined	Semi-permeable, but horizontal flow negligible
Confined	Impermeable

When the time-drawdown data for each type of aquifer are plotted on double-log paper, curves such as those shown in *Figure 7.1* are obtained. These curves, which characterize and identify the major aquifer groups, are known as type curves. Most type curves are based on the assumption that the aquifer is of infinite areal extent and that the overlying and underlying confining beds are impermeable. Deviations from these assumptions result in departures from the theoretical time–drawdown curves in *Figure 7.1*. This is illustrated by *Figure 7.2*, which shows the effect of an impermeable or barrier boundary and a recharge boundary on the observed time-drawdown data.

If the time-drawdown data are plotted on semi-log paper, a linear relationship is obtained, provided that the well is remote from any boundaries (*Figure 7.3*). A deviation from a straight line or a change in gradient may indicate that there is an effective boundary nearby (see *Figure 7.3*(*b*)). Drawdown-log time graphs of this form can be extrapolated to provide useful information regarding the drawdown that would result from a period of pumping longer than that of the test.

7.2.2 Variation of drawdown with distance

The coefficient of transmissivity is particularly important with respect to the relation between distance from the pumped well and drawdown. In a formation with a low transmissivity, the cone is deep with steep sides, while it is flat and shallow when the transmissivity is high, as shown in *Figure 7.4*(*a*). The effect of storativity and pumping rate on the shape of the cone of depression is also illustrated diagrammatically in *Figure 7.4*. It is apparent from the figure that the gradient of the cone of depression increases towards the well. This can be explained by the fact that as the water converges on the well it is forced to flow through smaller and smaller cylindrical sections of the aquifer

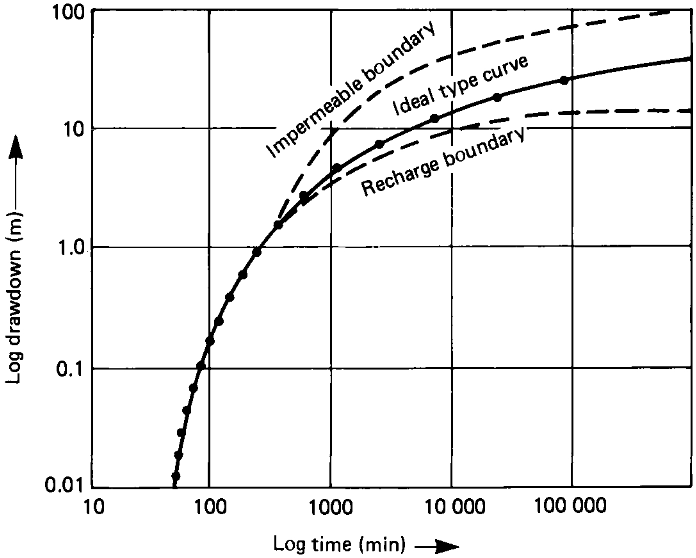

Figure 7.2 The effect of aquifer boundaries on double-log plots of drawdown against time. An impermeable boundary results in an increased drawdown, as shown by the departure of the data plot from the ideal type curve. A recharge boundary, such as leakage through a semi-permeable confining bed, decreases the drawdown that would otherwise be expected so that the data plot below the ideal type curve. In both cases the effect of the boundary becomes apparent after approximately 400 mins of pumping

(*Figure 7.5*), while in unconfined aquifers the saturated thickness also decreases towards the well (see *Figure 6.8*(*a*)). As a result the cross sectional area of flow progressively diminishes, requiring a corresponding increase in velocity so that continuity of flow can be maintained. By Darcy's law, the hydraulic gradient varies directly with the velocity, hence the cone of depression has a small slope at the extreme radius of influence and increases towards the well. When plotted on semi-log paper a linear relationship between distance and drawdown generally is apparent (*Figure 7.6*). By extrapolating the distance–drawdown relationship obtained from the observation holes, it is possible to determine the maximum extent of the cone of depression, otherwise called the radius of influence. Near the well itself, high velocities may result in turbulent flow and additional head loss or drawdown. This is a significant factor when analysing drawdown data from the well and when deciding the location of the observation holes and will be considered in more detail in later sections.

In a uniform, isotropic, homogeneous aquifer (if such an aquifer exists) the cone of depression is symmetrical and the distance–drawdown curve is the same in all directions. In practice, all aquifers are anisotropic to some extent so the distance–drawdown curve may be valid only for one particular section through the cone of depression. For this reason, when performing a pumping test it is advantageous to have at least one observation hole located perpendicularly to a line passing through the other observation holes (see Section 7.5.4) so that variations in the distance–drawdown graph can be detected.

Another factor which may affect the shape of the cone of depression and the flow to the well is the initial slope of the water table or piezometric surface. Usually these slopes are relatively small and of little significance. However, a steep slope would result in an

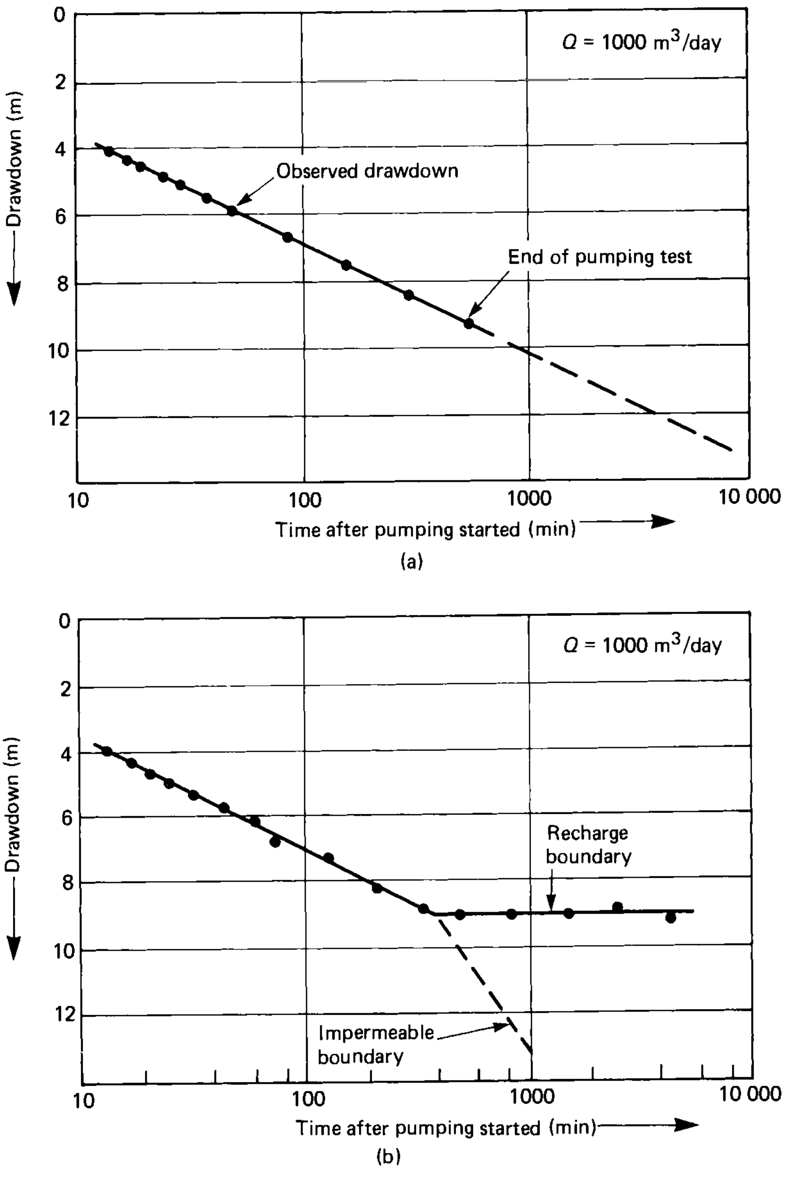

Figure 7.3 Pumping test data plotted on semi-log paper when (a) the aquifer is infinite and (b) recharge and impermeable boundaries are located within the radius of influence of the well. (a) When plotted on semi-log paper (drawdown against log t) a linear relationship exists between drawdown and the duration of pumping. This provides a convenient method of assessing the effect of a period of abstraction longer than that used during the pumping test itself, and simply involves extrapolating the line. However, this assumes that there are no boundaries in the aquifer. (b) When recharge to the aquifer occurs within the radius of influence of the well, the slope of the drawdown-log time curve becomes flatter. The horizontal line indicates that after 400 min of pumping, recharge equalled the well discharge and equilibrium was established. If an impermeable boundary had been encountered by the expanding cone of depression, this would have been indicated by an increase in the slope of the drawdown-log time graph, shown dashed

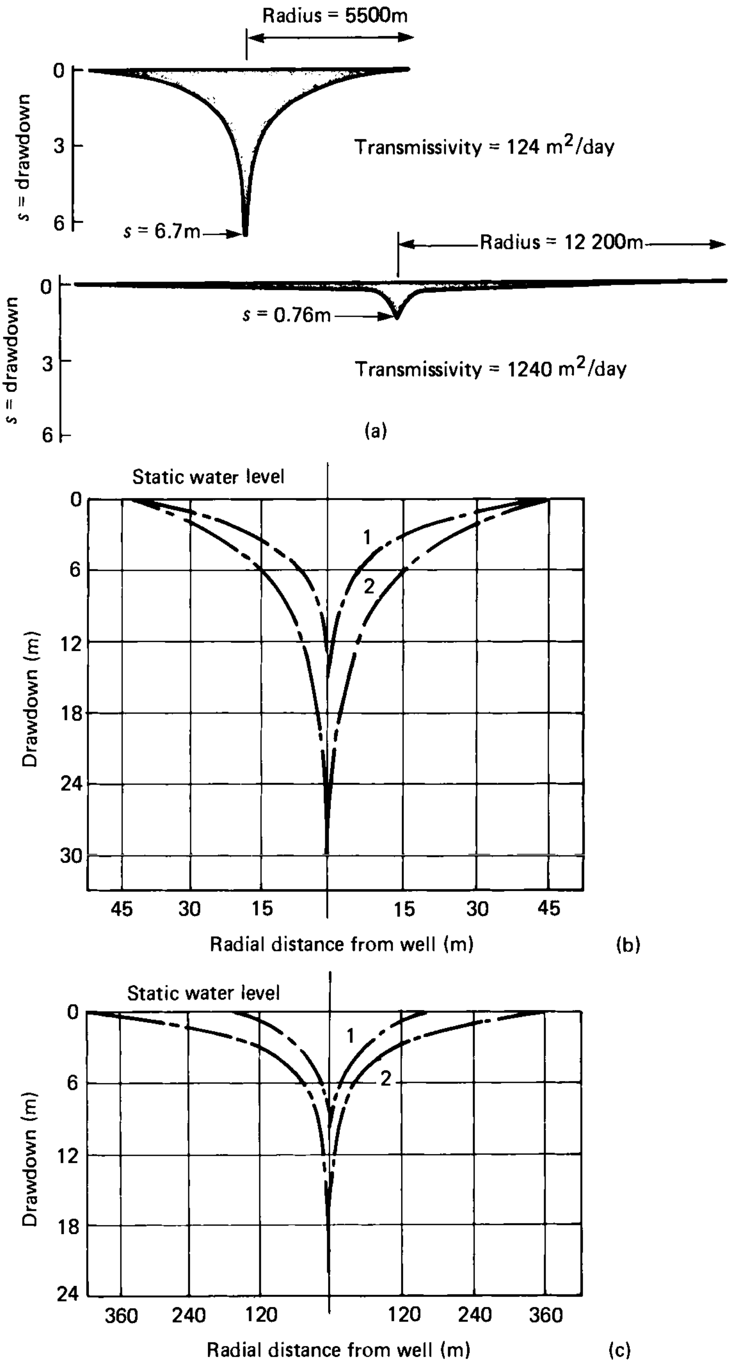

Figure 7.4 The effect of transmissivity, storativity and well discharge upon the size and shape of the cone of depression. (a) Effect of differing coefficients of transmissivity upon the shape and extent of the cone of depression when the pumping rate and the other factors are the same in both cases (after Anon[3]). (b) Influence of storativity on the drawdown in a well when the other factors are the same in both cases (after Anon[5]), S is storativity and S_1 is $50S_2$. (c) Influence of the rate of discharge on the drawdown in a well when the other factors are the same in both cases (after Anon[5]), Q is the rate of discharge and Q_2 is $2Q_1$

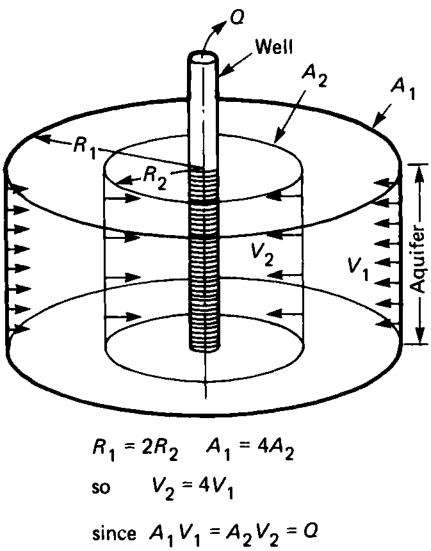

Figure 7.5 As flow converges on a well it passes through imaginary cylindrical surfaces that become progressively smaller necessitating an increase in velocity

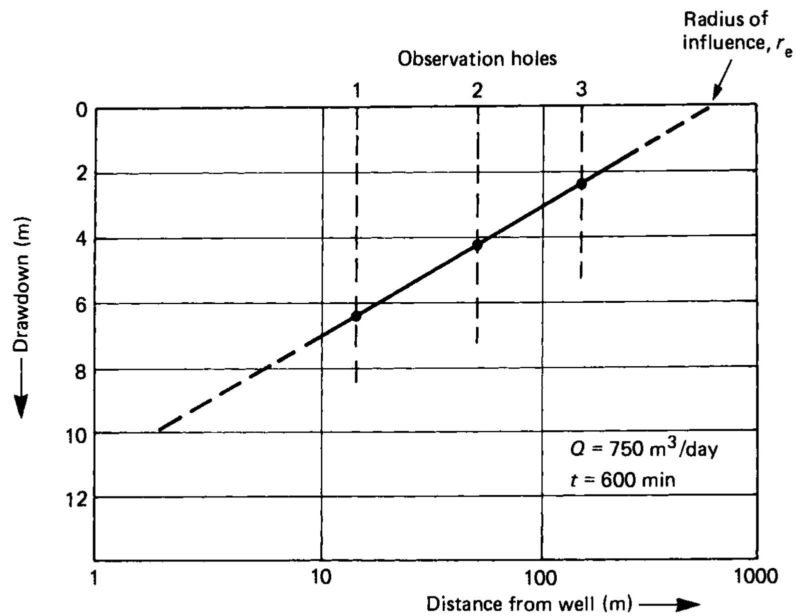

Figure 7.6 When plotted on semi-logarithmic graph paper a linear relationship between drawdown and distance from the pumped well is apparent. The line representing the cone of depression can be extrapolated easily to determine the radius of influence of the test. In this example, the drawdown at three observation holes has been plotted 600 min after pumping started

elliptical, rather than circular, cone of depression being formed since more of the discharge would originate from the upstream side of the well. In effect the area of diversion to the well (see Section 4.5.6) is increased on the upstream side where the natural slope of the water table or piezometric surface complements the slope induced by pumping. The reverse is true on the downstream side of the well where the natural slope of the water table or piezometric surface away from the well is in opposition to the slope towards the well induced by pumping. Consequently, the area of diversion to the well is effectively reduced on the downstream side. This should be remembered when constructing a distance–drawdown graph.

7.3 Well hydraulics

Some of the methods available for analysing pumping test data are based only on the information obtained from the observation holes, while others utilize the drawdown observed in the pumped well itself. Consequently, it is important to have a knowledge of well hydraulics and the implications of these techniques.

The drawdown in a pumped well is really made up of two components. The first is the aquifer loss, which is the drawdown caused by resistance to laminar flow within the aquifer. The second is the well loss, which is the head or drawdown required to overcome the resistance to turbulent flow in the vicinity of the well, through the screen and up to the well (*Figure* 7.7). Thus, a pumping test is really an aquifer test and a well test.

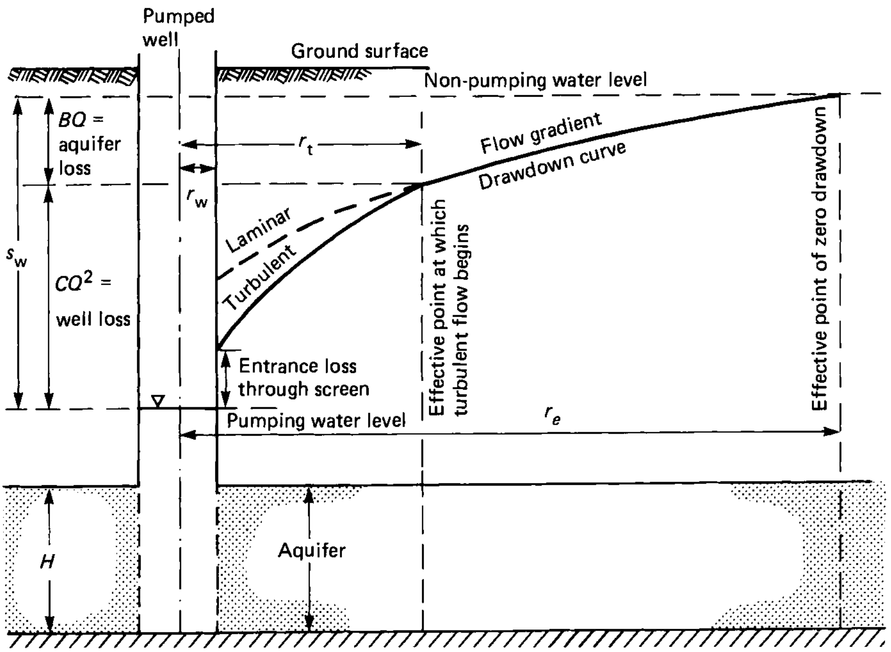

Figure 7.7 Diagrammatic cross section of a well showing the types of flow involved in well hydraulics and the relation of well loss, CQ^2, to aquifer loss, BQ, and total drawdown, s_w (after Bruin and Hudson[6]). Note that the diagram is not drawn to scale and that generally $BQ \gg CQ^2$

The total drawdown in a well is often represented by an equation of the form

$$s_w = BQ + CQ^2 \tag{7.1}$$

where s_w is the drawdown in the well, Q is the rate of discharge from the well, BQ is the aquifer loss (laminar flow, head loss is proportional to velocity, v), CQ^2 is the well loss (turbulent flow, head loss is proportional to v^2) and B, C are coefficients[7,8,9].

There has been considerable debate as to whether or not the square in Equation (7.1) should, in fact, be a higher power[10,11,12]. However, Eden and Hazel[13] suggested that any deviation from the relationship that the well loss varies with the square of the discharge rate should be viewed with suspicion[9].

The evaluation of well losses is most important when observations of the drawdown in the pumped well are to be used in the pumping test analysis. If these readings are not corrected to the theoretical drawdown by deducting the well loss, these data will be incompatible with that obtained from the observation holes (see Section 7.10.5). Additionally, if uncorrected readings are used in, say, Equation (7.9), then this will lead to an underestimation of transmissivity.

The evaluation of the well loss also enables the efficiency of the well to be calculated. Well efficiency, which is the ratio of aquifer loss to the total drawdown in the pumped well, can be expressed as follows

$$\text{Well efficiency} = \frac{BQ}{(BQ + CQ^2)} \times 100 \text{ per cent} \tag{7.2}$$

It should be noted that good design can minimize well losses in a given situation but never eliminate them and that the comparison of well efficiencies is not really valid unless the wells are virtually identical.

Jacob[7] devised a step drawdown test (see Sections 7.6.5 and 7.10.5) that allowed the well loss, well efficiency and the effective radius of the pumped well to be determined. More recently, Clark[8] summarized methods of analysing the data from step drawdown tests and calculating the value of the coefficients B and C. The significance of the well loss coefficient, C, is illustrated by *Table 7.1*.

One reason for conducting a step drawdown test is to enable the well loss to be evaluated when there are no observation holes and the data from the pumped well have to be analysed. Knowledge of the well loss component of drawdown allows the correction of the observed pumped water levels in the well, so giving the theoretical drawdown which can be used in the well discharge equations (see Section 7.4). If there are several observation holes in addition to the well, then a quicker, if less rigorous, method of estimating the well efficiency is to plot a log distance–drawdown graph, as shown in *Figure 7.8*. If the line joining the drawdown recorded in the observation holes is extrapolated until it meets the vertical line which represents the radius or the effective radius of the well, then this represents the aquifer loss or theoretical drawdown that

TABLE 7.1. Relation of well loss coefficient, C, to well condition (after Walton[14])

Well loss coefficient C ($\mathrm{min^2/m^5}$)	*Well condition*
<0.5	Properly designed and developed
0.5 to 1.0	Mild deterioration due to clogging
1.0 to 4.0	Severe deterioration or clogging
>4.0	Difficult to restore well to original capacity

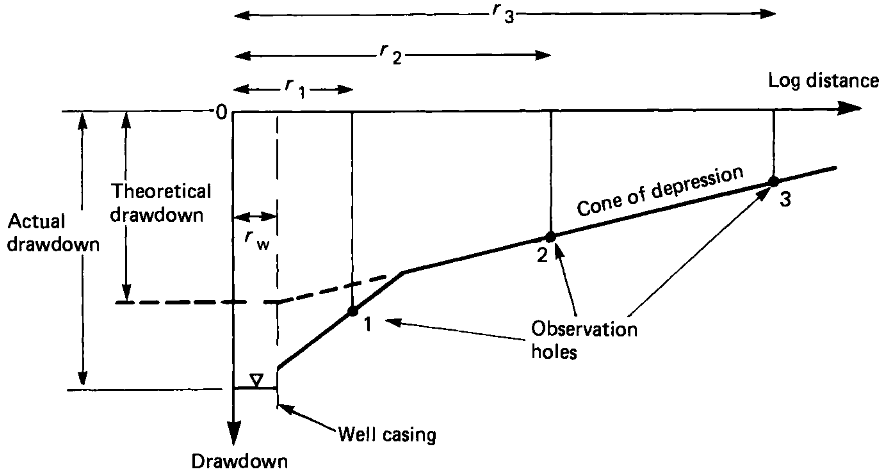

Figure 7.8 Use of a (log distance)-drawdown plot to estimate well efficiency from the ratio of theoretical to actual drawdown in the pumped well. This is a quick technique, although not very rigorous

would be observed if there was no turbulence and well loss. The actual drawdown in the well represents the total loss. Hence the efficiency of the well is given by

$$\text{Well efficiency} = \frac{\text{Theoretical drawdown}}{\text{Actual drawdown}} \times 100 \text{ per cent} \tag{7.3}$$

Of course, if there are no observation holes, or if the aquifer is significantly anisotropic and heterogeneous, this simple technique will not be applicable.

Equation (7.1) can also be written in the form

$$s_w/Q = B + CQ \tag{7.4}$$

where s_w/Q is the specific drawdown. The specific drawdown, or drawdown per unit discharge, is the reciprocal of specific capacity, which is the discharge per unit of drawdown. It can be seen that both the specific capacity and the specific discharge of a well are a function of discharge and the coefficients B and C and consequently values for different wells may not be directly comparable.

7.4 Well discharge equations and the prediction of the response of an aquifer to pumping

The data obtained from pumping tests can be analysed using two types of formulae, namely those applicable to equilibrium and non-equilibrium conditions. The type of test and analysis employed will depend upon the reasons for conducting the test (see Section 7.5.1). However, the equilibrium equations can be used to predict the response of the aquifer to pumping, regardless of which technique is eventually used to analyse the test data.

7.4.1 The equilibrium equations

The Dupuit–Thiem equations[15,16] are valid only when the well is pumped at a constant rate and the cone of depression has achieved a steady state and when the following limiting assumptions are satisfied.

1. The aquifer contains no boundaries in the area around the well, which means that it is effectively infinite in areal extent. This may or may not be true in a given location, but often one reason for conducting a pumping test is to evaluate any hydrological boundaries in the area of influence of the test.
2. The aquifer is of uniform thickness or saturated thickness. Provided that the drawdown is not too large this is generally an acceptable assumption (see Section 6.3.1), but it is usual practice to allow for the 'dewatering' of unconfined aquifers by replacing the drawdown, s, by the corrected drawdown, $s_c=(s-s^2/2H)$ when the dewatering adjustment is greater than 5 per cent of the drawdown, where H is the saturated thickness of the aquifer. For further details see Kruseman and de Ridder[4], Jacob[17] and Brown *et al.*[18]
3. The aquifer is homogeneous and isotropic. Few aquifers in nature satisfy these conditions, but the assumption is reasonable unless the pumping test is on a small scale. Even in heterogeneous aquifers, such as limestone, this assumption is acceptable provided that a sufficiently large volume of the aquifer is affected by the test.
4. The slope of the water table or piezometric surface is negligible before pumping starts. Most water table and piezometric surfaces have small gradients and a slope of a few degrees will not significantly affect the results.
5. The pumped well completely penetrates the aquifer (or at least 85 per cent of it). If a partially penetrating well is adopted, some of the observed data may require the use of a correction factor to allow for the additional drawdown that will result (see Section 6.3).

If these conditions are satisfied the discharge from a well in either a confined or unconfined aquifer can be calculated from a consideration of the cross sectional area of flow and the corresponding velocity, obtained from Darcy's law, so that

$$Q=Av=2\pi rHk(\mathrm{d}h/\mathrm{d}r) \tag{7.5}$$

where Q is the well discharge (m^3/day), H is the saturated thickness of the aquifer (m), k is the coefficient of permeability or hydraulic conductivity (m/day), and $\mathrm{d}h/\mathrm{d}r$ is the gradient of the water table or piezometric surface (*Figure 7.9*). If this equation is integrated between limits representing the conditions at two observation holes at distances r_1 and r_2 from the pumped well where the drawdown is s_1 and s_2 respectively, then it is possible to obtain an equation of the form[19]

$$Q=\frac{2\pi kH(s_1-s_2)}{\ln(r_2/r_1)} \tag{7.6}$$

In the case of an unconfined aquifer, s should be replaced by $(s-s^2/2H)$ as explained in (2) above. The equation can be applied to non-equilibrium conditions provided that the value of (s_1-s_2) is virtually constant, which can be the case even though both s_1 and s_2 are still increasing slowly. This means that a pumping test of a few days may suffice, whereas it may take months or even years for true equilibrium to be established.

If drawdown data are available from only the pumped well and one observation hole, Equation (7.6) can be modified to

$$Q=\frac{2\pi kH(s_w-s_1)}{\ln(r_1/r_w)} \tag{7.7}$$

where s_w is the drawdown in a pumped well of radius r_w. This value of drawdown should

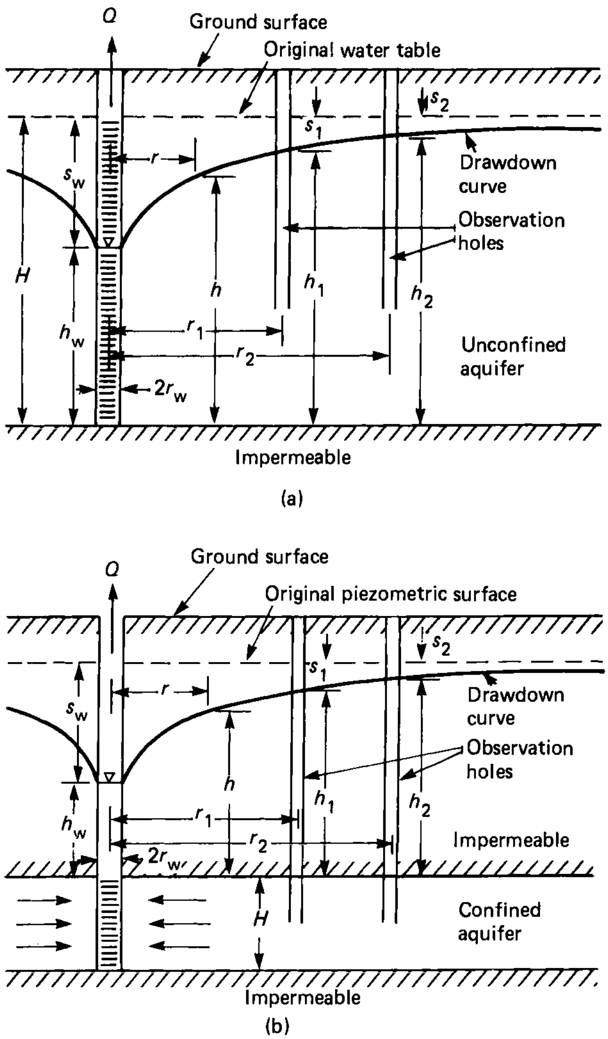

Figure 7.9 Schematic cross section of (a) Radial flow to a well penetrating an unconfined aquifer. (b) Radial flow to a well penetrating an extensive confined aquifer

be corrected to take into consideration well losses and so on. Equations (7.6) and (7.7) can, of course, be used to calculate the transmissivity, T, of the aquifer ($=kH$) from the pumping rate and observations of the drawdown. If drawdown data are available for only the pumped well, the following approximate relation[18] is useful for obtaining a preliminary estimate of the coefficient of transmissivity

$$T=\frac{Q}{2\pi s_w}\ln(r_e/r_w) \tag{7.8}$$

where r_e is the radius of influence, that is the maximum extent of the cone of depression. This equation is based on the assumption that r_2 in Equation (7.6) is equal to the radius of influence, so that s_2 is approximately equal to zero. For conditions commonly encountered in unconfined aquifers, r_e is generally about 300 m, so if r_w is taken as

0.3 m, Equation (7.8) becomes

$$T = 1.2Q/s_w \tag{7.9}$$

The ratio Q/s_w is the specific capacity of the well. For a confined aquifer the value of r_e may be as large as 3000 m, so that for a well of 0.3 m radius Equation (7.9) becomes

$$T = 1.6Q/s_w \tag{7.10}$$

From a comparison of Equations (7.9) and (7.10), it is apparent that the numerical constant does not vary greatly as a result of changing r_e. This, fortunately, means that the value of T is not significantly affected by poor estimates of the radius of influence, so the values in *Table 7.2* can be adopted if a more specific figure is not available. However, it should be appreciated that the value of T obtained applies only to that portion of the aquifer adjacent to the well, which is affected by drilling practices, well radius, heterogeneity, and other variables[18].

TABLE 7.2. Approximate values of the radius of influence, r_e (after Brown *et al.*[18])

Type of formation	*Groundwater condition*	*Extent of cone of depression* (r_e) (m)
1. Fine- and medium-grained sands	Confined	250 to 500
	Unconfined	100 to 200
2. Coarse-grained sands and gravel-pebble beds	Confined	750 to 1500
	Unconfined	300 to 500
3. Fissured rocks	Confined	1000 to 1500
	Unconfined	500 to 1000

The equilibrium equations can be very useful for obtaining preliminary values of T from the results of small-scale pumping tests conducted on a pilot hole or on the main production well prior to the design of the observation hole network and the full-scale test. Alternatively, if an approximate value of T has been obtained using any of the methods described in Chapter 3, the drawdown may be calculated at various points in the aquifer and the most suitable spacing for the observation holes determined.

Limitations of the equilibrium technique include the need to pump the well until the cone of depression has stabilized, or at least until $(s_1 - s_2)$ has a constant value. Unfortunately it is often quite difficult to tell whether or not true equilibrium has been achieved, since hydrogeological boundaries or short period fluctuations in level may mask the effects of pumping (see Section 7.7). An additional disadvantage is that the equilibrium technique must, for reliable results, be applied to two observation holes, whereas the non-equilibrium analysis needs only one. Since equilibrium tests also require a long period of pumping, they tend to be expensive to implement. Because equilibrium analysis is applied to a steady state condition it cannot yield a value for the coefficient of storage, whereas non-equilibrium tests can. Finally, non-equilibrium techniques can be applied to the groundwater recovery data recorded after pumping has stopped, so giving a further inexpensive means of estimating T and S.

7.4.2 The Theis non-equilibrium equations

The five limiting assumptions listed in Section 7.4.1 also apply to non-equilibrium tests, but it is now assumed that the cone of depression has *not* stabilized and that the water level in the aquifer is still falling. Two additional limiting assumptions are that

1. Water storage within the pumped well is negligible, which is generally true if the diameter of the well is relatively small.
2. Water pumped from storage is discharged instantaneously with the fall in head. This assumption is acceptable in most cases, while in others either a type curve which allows for delayed yield or a correction factor can be employed to compensate for this[4].

The development of the non-equilibrium technique represented a major advance in aquifer evaluation and the analysis of pumping test data, since it enabled much shorter tests to be conducted, required only one observation hole and was capable of yielding the value of both the formation constants, S and T. The basis of the method is that the rate of decline of head multiplied by the coefficient of storage and summed over the area of influence equals the discharge, so

$$\frac{\partial^2 h}{\partial r^2}+\frac{1}{r}\frac{\partial h}{\partial r}=\frac{S}{T}\frac{\partial h}{\partial T} \qquad (7.11)$$

This equation was first adapted for use by hydrogeologists by Theis[20] as a result of an analogy between confined groundwater flow and heat conduction, so that

$$s=\frac{Q}{4\pi T}\int_u^\infty \frac{e^{-u}}{u}\,\mathrm{d}u \qquad (7.12)$$

or

$$s=\frac{Q}{4\pi T}W(u) \qquad (7.13)$$

where

$$u=\frac{r^2S}{4Tt} \qquad (7.14)$$

and $W(u)$ is the well function. Equation (7.12) can be expressed as a convergent series of the form

$$s=\frac{Q}{4\pi T}\left[-0.5772-\ln u+u-\frac{u^2}{2!\,2}+\frac{u^3}{3!\,3}-\frac{u^4}{4!\,4}+\cdots\right] \qquad (7.15)$$

Equation (7.15) is known as the non-equilibrium or Theis equation. An explicit solution for this equation cannot be obtained, so the analysis of non-equilibrium pumping test data is accomplished using graphical techniques. These are described in Section 7.10. The method devised by Theis incorporates the use of a Theis type curve, which is a double-log graph of $W(u)$ against u (*Figure 7.10*). However, it is frequently more convenient to use a reversed type curve which is $W(u)$ against $1/u$ (*Figure 7.10*). If the reversed Theis type curve is superimposed on the curve resulting from the drawdown in an observation hole, plotted as s against t/r^2 to the same double-log scale as the type curve, an arbitrary match point on the overlapping part of the graphs can be selected (*Figure 7.11*) which enables the values of S and T to be determined from:

$$T=\frac{Q}{4\pi s}W(u) \qquad (7.16)$$

and

$$S=\frac{4Ttu}{r^2} \qquad (7.17)$$

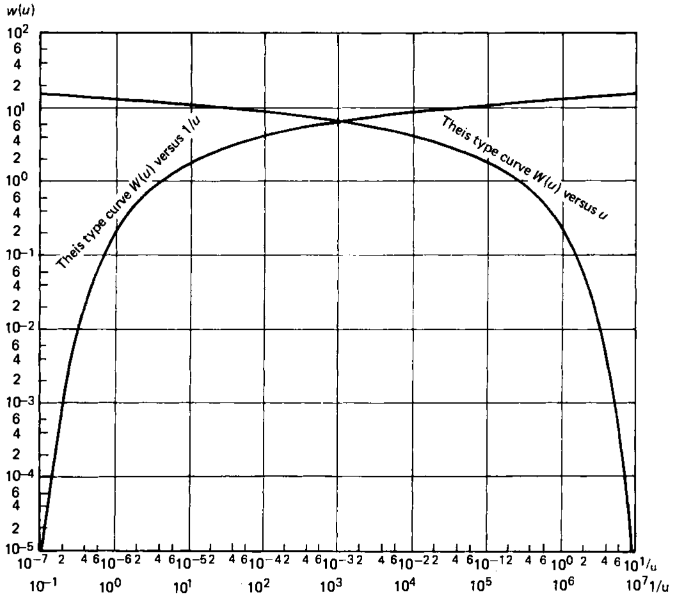

Figure 7.10 Normal Theis type curve of $W(u)$ against u, and reversed Theis type curve of $W(u)$ against $1/u$

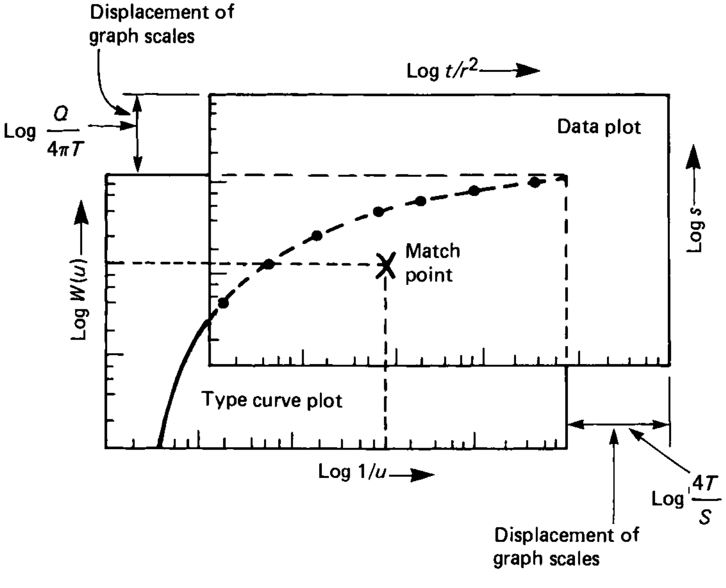

Figure 7.11 Theis method of analysing pumping test data. The reversed Theis type curve of $W(u)$ against $1/u$ is drawn on double-logarithmic tracing paper. The observed test data, also plotted on double-logarithmic paper as s against t/r^2, is then superimposed over the type curve. Keeping the axes of the graphs parallel, an arbitrary match point is chosen either on the curve or on any overlapping part of the graphs (it is often convenient to chose a match point whose coordinates on the type curve graph are either 1 or multiples of 10). The matching coordinate values of $W(u)$ and s are then substituted into Equation (7.16) to calculate T, while the corresponding values of t/r^2 and $1/u$ (as u) substituted into Equation (7.17) gives the value of S. Data from more than one observation hole can be plotted on the same graph, if desired

where equivalent values of t/r^2 and $1/u$, and s and $W(u)$ are obtained from the graphs. An example of this procedure can be found in Section 7.10.1, which also has a table relating $W(u)$, u and $1/u$.

If a normal type curve is used in the analysis, then the drawdown should be plotted against r^2/t and not t/r^2, as described above. The Theis equations can be applied to both confined and unconfined aquifers, but one refinement to take into consideration, the dewatering of unconfined aquifers, is to replace the drawdown s by $(s-s^2/2H)$ when the dewatering adjustment is greater than 5 per cent of the drawdown. This is the same correction that was used with the equilibrium equations[4,17].

Although the above equations are not very useful for predicting the response of an aquifer to pumping, if a preliminary test has been conducted on a pilot hole (see Chapter 3) it may be possible to identify the kind of type curve to be used for the data analysis. It is important to realize that the Theis type curve in *Figure 7.10* represents only one curve applicable to a particular variety of aquifer. Many other families of curves applicable to other kinds of aquifer have been determined (see *Figure 7.1*) and to be proficient in pumping test analysis it is necessary to be aware of all the various type curves available and their limitations. Useful compendiums are provided by Ferris *et al.*[21], Hantush[22] and Reed[23]. If the site conditions revealed by the pilot test do not conform to any of the known type curve formulations, then Brown *et al.*[18] recommended that it should be abandoned.

The type curve of observed data obtained from a preliminary test on the production well can be used to decide the optimum distance for additional observation holes. These should be located so that data are obtained on both the vertical and horizontal part of the curve. If this is not achieved an error may be incurred in the calculated value of the coefficient of storage. Additionally, the observation holes should be located so that no data are recorded during the first few minutes of the test. During this period the rapid change in head makes the pump discharge difficult to control and time lag effects may also be significant.

7.4.3 The Jacob non-equilibrium approximation technique

The seven limiting assumptions listed in Sections 7.4.1 and 7.4.2 also apply to the Jacob technique[24], but it is now assumed that

1. The value of r is small and/or the value of t is large.
2. The value of $u=r^2S/4Tt<0.01$. In practice this means that the data obtained in the first hour of a test on a confined aquifer, and the first 12 h with an unconfined aquifer, will not be valid and should be ignored in the subsequent analysis, although it can be plotted along with the rest of the data. The early data will probably form a curve, whereas a linear relationship should be obtained.

If the above conditions are satisfied, then only the first two terms of the convergent series expressed in Equation (7.15) are significant. Consequently the drawdown can be expressed by the asymptote

$$s=\frac{Q}{4\pi T}\left(-0.5772-\ln\frac{r^2S}{4Tt}\right) \tag{7.18}$$

which can be rewritten as

$$s=\frac{Q}{4\pi T}\left(\ln\frac{4Tt}{r^2S}-0.5772\right) \tag{7.19}$$

which reduces to

$$s=\frac{2.30Q}{4\pi T}\log\frac{2.25Tt}{r^2S} \tag{7.20}$$

Therefore, a drawdown–log time graph will be in the form of a straight line. If this line is extended until it intercepts the time axis at a point, t_0, where the drawdown $s=0$, then substituting these values into Equation (7.20) gives

$$0=\frac{2.30Q}{4\pi T}\log\frac{2.25Tt_0}{r^2S} \tag{7.21}$$

Because $2.30Q/4\pi T\neq 0$ it follows that $2.25Tt_0/r^2S=1$, so that for a time–drawdown analysis

$$S=\frac{2.25Tt_0}{r^2} \tag{7.22}$$

and

$$T=\frac{2.30Q}{4\pi\,\Delta s} \tag{7.23}$$

where Δs is the drawdown per log cycle of time. An example of this technique is given in Section 7.10.2.

If the drawdown in three or more observation holes can be recorded simultaneously at some time, t, after pumping started, then an alternative technique is to plot these data on a drawdown–log distance graph. Again, a straight line should be fitted through the points and extended until it intercepts the distance axis where $s=0$ and $r=r_0$, where r_0 is the radius of influence at the end of the chosen time period, t. Using the same reasoning as above, it can be shown that for a distance–drawdown analysis

$$S=\frac{2.25Tt}{r_0^2} \tag{7.24}$$

and

$$T=\frac{2.30Q}{2\pi\,\Delta s} \tag{7.25}$$

where Δs is the drawdown per log cycle of distance, obtained from the graph. This procedure can be repeated at intervals throughout the duration of the test; the values of S and T should agree quite closely.

It is apparent that the slope of the drawdown–log distance graph is twice that of the drawdown–log time plot. This very convenient relationship exists because t appears, in Equation (7.20), to the first power, whereas the r term is squared. Since $\log r^2$ is equivalent to $2\log r$, it follows that the value of Δs for the drawdown–log distance graph will be twice that for the drawdown–log time graph. Consequently, if one relationship is known, the other can be deduced. For instance, if a single observation hole is used to obtain a drawdown–log time graph, it is also possible to construct a drawdown–log time plot, despite the fact that there is only one observation hole[3]. Thus, if a mini pumping test is conducted on a well with only one pilot or observation hole nearby, the results from this hole can be used to decide the optimum distances for the other holes.

7.4.4 The Chow non-equilibrium technique

Chow[25] devised a method of pumping test analysis that avoided the laborious curve fitting of the Theis method and which does not have the restrictions of the Jacob technique. The seven limiting assumptions listed in connection with the Theis method still apply, since this technique is based directly on the Theis equations. The procedure is to plot the drawdown–log time relationship at one of the observation holes, as for the Jacob method. The result is a gentle curve. An arbitrary point is chosen on the curve and the coordinates of the point, s and t, noted. Next, the tangent to the curve is drawn through the chosen point and the drawdown per log cycle of time determined. The value of $F(u)$ is then calculated from

$$F(u)=\frac{s}{\Delta s} \tag{7.26}$$

Knowing the value of $F(u)$ it is possible to find the corresponding values of $W(u)$ and u from a nomogram (*Figure 7.12*) or Equation (7.33). The values of the aquifer constants can then be determined from

$$T=\frac{Q}{4\pi s}W(u) \tag{7.27}$$

and

$$S=\frac{4uTt}{r^2} \tag{7.28}$$

An example of this technique is given in Section 7.10.3. Although this procedure allows relatively short tests to be conducted and is an efficient way of analysing pumping test data, the method is not very useful for predicting the likely response of an aquifer to pumping, unlike the techniques previously described.

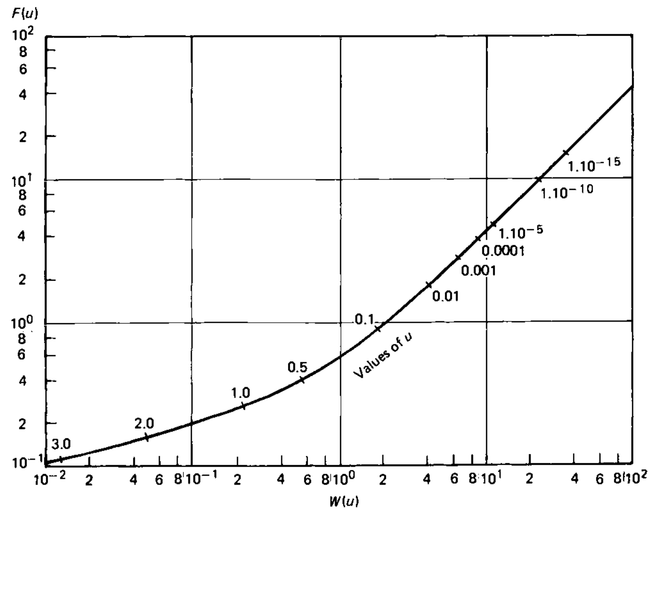

Figure 7.12 Chow nomogram giving the relation between $F(u)$, $W(u)$ and u (after Chow[25])

7.5 Planning a pumping test

Pumping tests are expensive to conduct, typically costing around £20 000 per well in 1985. In Britain wells are pumped for about two weeks (336 h) on average during a test. Anon[26] derived the following formula for estimating the cost of a pumping test

$$\text{Pumping test cost (£)} = 136 \times \text{hours}^{0.66} \text{ in 1976} \qquad (7.29)$$

This model illustrates the relationship between cost and duration of pumping, although the cost must be translated to present-day values (see Section 1.5). Obviously, the expense incurred increases with the length of time spent on site, so tests must be carefully planned to ensure that they are executed swiftly and efficiently. There are numerous other costs such as the time required to assemble the plant and manpower and set up the test, the expense of a temporary pipeline to discharge the water abstracted from the well, plant, equipment and so on. An additional expense is the cost of constructing the observation holes, typically three holes at about £6000 each in 1985 and the well itself. However, the last two may be considered as separate items.

The chances of conducting a successful test may be increased if the following steps are adopted.

1. Decide the objectives of the test, and thus the type of test required.
2. Decide the optimum location of the test/production well.
3. Conduct a preliminary investigation using a pilot/observation hole.
4. Decide the location of the observation holes relative to the test well, and the depth of hole and type of installation required.

These steps make a vital contribution to the satisfactory completion of a test, and for this reason are described in more detail below.

7.5.1 Objectives of a pumping test and type of test required

Pumping tests are not always conducted for the same reasons, although most tests have some common objectives. To a large extent the objectives determine the type of test required. Common reasons for undertaking a pumping test are

1. To determine the hydraulic characteristics of the aquifer and the regional pattern of groundwater flow. This can be achieved by pumping the well at a constant rate approximately equal to its design discharge. The non-equilibrium formula can be used to calculate both T and S so only a short test need be conducted, although it is preferable to continue pumping until a stable drawdown is obtained, followed by a complete recovery of the water level (*Figure 7.13*(*a*)). This enables the equilibrium and recovery data to be analysed as well.
2. To investigate the effects of the design abstraction rate on the water level in the aquifer and on rivers and the environment in general. The effect of abstraction on other wells may be of particular interest since any 'interference' between wells will lead to increased drawdown and reduced yield. A suitable test may involve pumping the well at a constant rate near to the design discharge for an extended period of time. This would be preferable to pumping at a reduced rate and extrapolating the results to the design condition, since some boundaries may only become apparent at higher rates of discharge when the drawdown is larger. Consequently, a two-stage test may be considered, in which abstraction starts at the design discharge but is increased subsequently by some significant amount so as to reveal any boundaries that would be

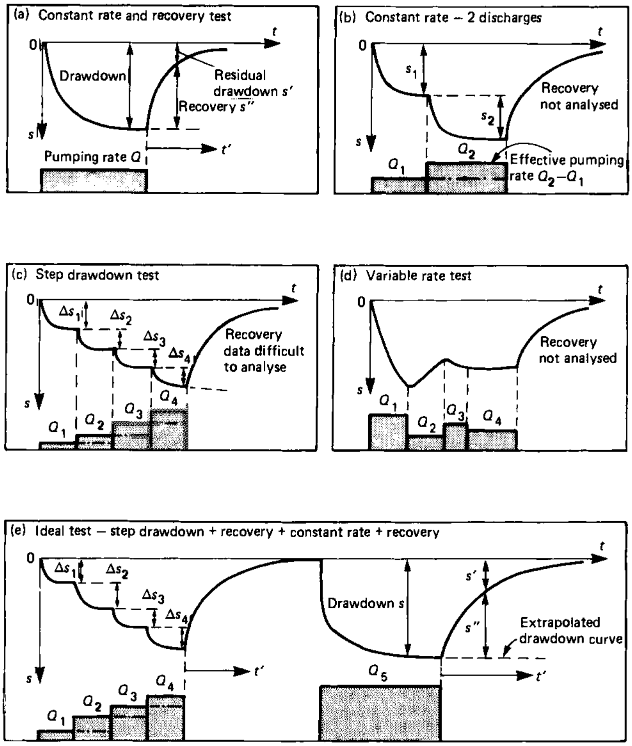

Figure 7.13 Diagrammatic representation of various types of pumping test

apparent during periods of overpumping (*Figure 7.13*(*b*)). The recovery data would not be analysed in this case.

3. To determine the aquifer loss, well loss, and the efficiency of the well. This is best achieved by conducting a step drawdown test, in which the discharge from the well increases at preselected time intervals (*Figure 7.13*(*c*)). Clark[8] recommended at least four steps, each of about 3 h duration. The recovery data following a step drawdown test can be analysed, but the results are not very reliable. A step drawdown test may also indicate the most suitable pumping rate for any later constant rate tests (*Figure 7.13*(*e*)).
4. To determine the perennial yield of the well. This can be achieved by varying the discharge from the well (*Figure 7.13*(*d*)) until the water level becomes steady (see Section 4.5.6). This is only practical if the aquifer has a short response time.
5. To determine the future operational set-up at the well. A test may be conducted to investigate the yield-drawdown (specific capacity) relationship at the well, the optimum size and depth of setting of the pump and its power requirement (see Section 6.7). A short test at or above the design discharge may be adequate for this purpose, provided that any underlying trend in groundwater level is taken into consideration.

Clark[8] recommended that a well testing programme should consist ideally of a 4 or 5 stage step drawdown test followed by complete recovery. This should be followed by a 24 or 72 h constant discharge test, the length depending upon the type of aquifer (see Section 7.6.3), at the production discharge. This test would end with the complete recovery of the water level (*Figure 7.13*(*e*)). A test programme like this allows the characteristics of the aquifer to be evaluated through the analysis of equilibrium, non-equilibrium and recovery data, while the aquifer and well losses can be determined from the step drawdown data. The application of more than one technique to more than one set of data is generally desirable since this improves the reliability of the conclusions drawn from the test. In some aquifers, however, equilibrium conditions may not be attained.

7.5.2 Selection of a suitable site for a pumping test

There are many factors which influence the choice of a site for a pumping test, but one of the most important is that the well is situated in a part of the aquifer capable of giving a high yield. Frequently the test well becomes a production well, so this makes economic sense. Some guidance to the location of wells for maximum yield was given in Section 4.6.

The economic factors governing the construction and operation of the well are important also. Obviously the depth and hardness of the strata to be penetrated influence the cost of the well, while the depth to water determines the future pumping lift and hence, operational costs. The quality of the water at a particular site may be an additional consideration, both from the point of view of its aggressiveness to well fittings and the treatment required before it can be put into supply.

The cost of a pumping test is reduced if existing wells in the vicinity of the test site can be used as observation holes. Alternatively, new observation holes may be constructed anyway, with the additional data provided by the existing wells allowing a more comprehensive analysis of the test results. Nevertheless, if there are already production wells in the area it may be advisable to pick a more remote site in order to avoid interference and derogation between abstraction wells, during both the testing and operational phases (see Section 7.7.3).

The hydraulic considerations which influence the selection of the pumping test site include the limiting assumptions associated with the analysis of the test data (Section 7.4). Although some of these limitations are relatively unimportant, others, such as the assumption that the aquifer is infinite, are significant. Any potential hydraulic boundaries should be carefully evaluated before a major production well is located nearby. Sometimes, of course, one of the objectives of a pumping test is to reveal the presence of these boundaries.

The possibility of surface water–groundwater interconnection should also be kept in mind when planning a pumping test. Ideally, the water abstracted from a well during a pumping test should be discharged outside the area of influence of the test, preferably to a surface water drain. This is a wise precaution even if surface water–groundwater interaction seems unlikely, since the surface deposits may be found subsequently to be more permeable than first thought. It is also good practice to measure streamflows in the vicinity of the well before, during and after the test to determine if they have been affected by abstraction from the aquifer.

One obvious, but easily overlooked, requirement of the test site is that it should be accessible. The construction of a well and the conduct of a pumping test involves considerable manpower and plant, all of which must be able to reach the site easily. However, with a confined aquifer it may not be desirable to locate the well near a railway or highway, since the piezometric level may be sensitive to changes in surface loading. Other potential causes of fluctuations in groundwater level are shown in *Table 7.3*.

Once the optimum area for the test/production well has been identified, which may require some degree of compromise, a small diameter pilot hole should be sunk in the

TABLE 7.3. Summary of mechanisms that lead to fluctuations in groundwater levels (after Freeze and Cherry[27])

	Unconfined	*Confined*	*Natural*	*Man-induced*	*Short-lived*	*Diurnal*	*Seasonal*	*Long-term*	*Climatic influence*
Groundwater recharge (infiltration to the water table)	√		√				√		√
Air entrapment during groundwater recharge	√		√		√				√
Evapotranspiration and phreatophytic consumption	√		√			√			√
Bank-storage effects near streams	√		√				√		√
Tidal effects near oceans	√	√	√			√			
Atmospheric pressure effects	√	√	√			√			√
External loading of confined aquifers		√		√	√				
Earthquakes		√	√		√				
Groundwater pumpage	√	√		√				√	
Deep-well injection		√		√				√	
Artificial recharge; leakage from ponds, lagoons, and landfills	√			√				√	
Agricultural irrigation and drainage	√			√				√	√
Geotechnical drainage of open pit mines, slopes, tunnels, etc.	√			√				√	

locality to confirm the suitability of the site. If the conditions are found to be unfavourable the site can be abandoned without having incurred the expense of constructing a large diameter well, whereas if conditions are as expected the pilot hole can be used as an observation hole. Before sinking any additional holes a preliminary study should be conducted on the pilot hole.

7.5.3 The preliminary study

The preliminary study is important, since the data obtained from it may be used to determine the location of the production well and the observation holes. In particular, it is important that the preliminary study gives an indication of the transmissivity of the aquifer, so enabling the size of the cone of depression during the pumping test to be estimated. The observation holes can then be spaced accordingly. If the observation holes are too near the well where the flow is turbulent, the recorded drawdown data will be inconsistent with that obtained from parts of the aquifer where the flow is laminar. If the observation holes are too far away, there may not be a significant reduction in water level. Similarly, if the holes are too deep they will be unnecessarily expensive, but if they are too shallow they may be ineffective. The affect of any boundaries on drawdown must be taken into consideration at this stage.

During the preliminary study three main types of data should be collected. The first type is geological data relating to the lithology and character of the aquifer and the overlying and underlying strata and to the location of boundaries. The second is hydrogeological data, such as the direction of groundwater flow, the water table or piezometric gradient, the trend of water levels and so on. If these data are not available from existing wells in the region, it is advisable to complete one or more pilot holes in the area some time before pumping is due to commence in order to obtain such information. Thirdly, and less obviously, meteorological/hydrological data should be collected over a considerable period before the start of the pumping test. Typically this includes rainfall, evapotranspiration, atmospheric pressure and precipitation measurements. Without this background information it is impossible to evaluate the effect of abstraction on the environment, or to correct the pumping test data for meteorological changes which occur during the test (see Section 7.7).

The techniques that can be employed during a preliminary study to obtain these sorts of data have been described in Chapter 3.

7.5.4 The location and design of the observation holes

The observation holes should be placed at carefully selected locations so as to ensure that the most comprehensive and meaningful data are available for analysis. This implies that the observation holes should be neither too close nor too remote from the pumped well and that the spatial distribution should be chosen to suit the topography of the test site, bearing in mind that the number of holes that can be used may be restricted by financial considerations. If an inappropriate observation hole network is adopted, a very expensive pumping test may be wasted with delays and further expense being incurred while it is improved. Thus, it can be appreciated that the design of the observation holes is as important as the design of the production well itself. The factors which influence the location and design of the observation holes are discussed below.

Distance to the observation holes

This is, to some extent, dependent upon the type and character of the aquifer to be tested. In unconfined aquifers the spread of the cone of depression is rather slow because the aquifer is being desaturated, so unless the pumping test lasts for several days or more, only areas within about 100 m of the well will experience a significant drawdown. Confined aquifers behave rather differently since the decline in the piezometric surface is really due to a pressure effect. Confined aquifers also have relatively small coefficients of storage. As a result, the cone of depression may spread quite rapidly over distances of several hundred metres.

Other factors which affect the size of the cone of depression are the transmissivity of the aquifer and the pumping rate. This is relatively obvious from Equations (7.6) to (7.10), which indicate that the drawdown is proportional to the pumping rate. Consequently the observation holes should be more widespread if a large pumping rate is to be used and closer to the well if a small discharge is envisaged. If the transmissivity of the aquifer is low, the cone of depression will be deep but relatively small. If the transmissivity is high, the cone will be large but shallow (see *Figure 7.4*). Therefore, observation holes can be placed further away from the pumped well when the transmissivity of the aquifer is large.

Stratification of the aquifer is yet another factor that must be considered when deciding the location of the observation holes. In general the horizontal permeability of an aquifer tends to be greater than the vertical permeability. This means that the effects of pumping are transmitted more quickly in the horizontal than the vertical direction. As a result the drawdown observed at any particular distance from the pumped well may be different at various depths within the aquifer. These differences become less as the duration of pumping and the distance from the pumped well increase. For practical purposes the effects of stratification may become negligible at a distance of about three to five times the aquifer thickness from the well.

The pattern of flow to fully and partially penetrating wells in confined and unconfined aquifers was considered in Section 6.3. In essence the effect of partial penetration (that is less than about an 85 per cent open hole) is to increase the vertical flow of water near the pumped well so that the drawdown is enlarged. It should also be remembered that the water has to flow through concentrically smaller and smaller sections of the aquifer as it approaches the well (*Figure 7.5*) and that the saturated thickness of an unconfined aquifer decreases towards a pumping well. The resulting increase in the water velocity causes turbulent flow and increased drawdown in the area surrounding the well. This problem, and that due to partial penetration, can be avoided if the nearest observation hole is located a suitable distance away from the well (see *Table 7.4*). The drawdown in the pumped well itself, however, is still affected and is

TABLE 7.4. Suggested spacing of the observation holes

Aquifer condition	*Minimum distance from pumped well to nearest observation hole*	*General distance within which observation holes should be located*
Fully penetrating well in unstratified confined or unconfined aquifer—most cases	1 to $1\frac{1}{2}$ times aquifer thickness	20 to 200 m confined aquifer 20 to 100 m unconfined aquifer
Fully penetrating well in a thick or stratified confined or unconfined aquifer	3 to 5 times aquifer thickness	100 to 300 m confined aquifer 50 to 100 m unconfined aquifer
Partially penetrating well (<85 per cent open hole) in a confined or unconfined aquifer	$1\frac{1}{2}$ to 2 times aquifer thickness	35 to 200 m confined aquifer 35 to 100 m unconfined aquifer

incompatible with that recorded in the observation holes. For this reason analysis involving the drawdown in the pumped well should be avoided if possible. An additional consideration when deciding the location of the nearest well is that the drawdown observed in the first minute of the test is likely to be unreliable due to surging of the pump and time lag effects. Consequently the nearest well should be sufficiently remote from the pumped well that it does not respond until at least one minute of the test has elapsed. This is particularly important with non-equilibrium tests.

Another point to remember when deciding the final spacing of the observation holes is that the results of a pumping test are usually plotted to a log scale. Thus, to obtain a reasonable distribution of points on a drawdown–log distance graph (*Figure 7.6*), the observation holes should be located at distances such as 20, 50, 100, 200 and 500 m from the pumped well. It is also extremely useful to have one observation hole outside the radius of influence of the test (see *Table 7.2*) to record natural changes in groundwater level.

The number and spatial distribution of the observation holes

The data obtained from a single observation hole can be used to calculate the average value of the coefficients of transmissivity and storage, but if two or more observation holes are positioned at different distances from the well, both time–drawdown and distance–drawdown analyses are possible (see Sections 7.4.3 and 7.9). This provides a cross check on results and allows more reliable conclusions to be drawn. .Kruseman and de Ridder[4] recommended that at least three observation holes should be adopted. Other investigators recommended four observation holes; three on a line through the test well and one on a line perpendicular to this, also passing through the test well[5]. The first three holes provide suitable data for the distance–drawdown plot, while the fourth gives an indication of whether or not the cone of depression is asymmetric, which would be the case if the aquifer was significantly anisotropic and heterogeneous. In addition to these four boreholes, other observation holes may be necessary in order to evaluate known or suspected boundaries. The general rule here is to place the observation hole far enough away from the boundary for the pumping effects to be clearly established before the influence of the boundary becomes apparent. A possible exception to this rule is when there may be a hydraulic connection between the aquifer and a surface water resource such as a river. In this case it may be desirable to have a borehole situated very close to the river on the side of the well. An observation hole on the far side of the river can also provide useful data as to whether or not the cone of depression stops at the river as a result of recharge, or continues past this potential boundary. At least three holes are needed to locate less obvious hydrogeological boundaries using image well techniques (see Section 7.8).

In some situations there may be one or more aquifers, separated by relatively impermeable formations, lying above the aquifer to be tested. It is advisable to record the water levels in these aquifers so that vertical leakage and delayed yield can be evaluated. Similarly, if a confined or semi-confined aquifer is covered by a layer of low permeability material that is partly saturated, the effect of the pumping test on the water level in this material should be monitored.

It can be seen that a large number of observation holes are required if a rigorous pumping test analysis is to be conducted. This is obviously an expensive proposition, although the cost can be reduced if one or more of the observation holes can be used to test more than one well. Further savings can be made by using holes of small diameter and designing them to be as shallow as possible without jeopardizing their effectiveness.

The diameter of the observation holes

One way of reducing the cost of the observation holes is to use the smallest diameter that is compatible with the design requirements. Obviously, the narrower the hole the cheaper it is to construct. The inside diameter of observation holes commonly range from 50 to 150 mm. When deciding the diameter of hole required the following points should be considered

1. Is the observation hole going to be used as a pilot hole as well? If so, it must be large enough to accept all the equipment and instruments described in Chapters 2 and 3.
2. Is the observation hole to be equipped with an autographic water level recorder either during or after the test? Autographic recorders frequently use floats of about 100 mm diameter operating inside the well casing, but float sizes vary from around 40 to 200 mm. The basic rule is the larger the float, the greater the recording accuracy. The

casing should have a diameter at least 50 mm larger than the float, but additional clearance should be allowed if the well is more than 30 m deep[28]. This is necessary to allow the free operation of the float and cable. Of course, a pressure transducer and chart recorder or some other device could be used to monitor water levels, although these are not always quite so accurate[28,29].

3. What is the effective design life and function of the observation hole? Would a small diameter (say 50 mm) hole fitted with a piezometer suffice? (See Section 2.5.3.) In unconfined aquifers, particularly near a discharging well, there may be a distinct advantage in using a piezometer rather than an open borehole[30]. Under these conditions an open hole may provide a route for vertical flow between areas with different groundwater heads. On the other hand, if the hole is to be used regularly throughout the life of the production well then it should be of a suitably robust construction. It should have a sediment sump and be of such a diameter as to allow the periodic removal of the sediment from the hole (*Figure 7.14*).

4. Is it is necessary to monitor the water level in more than one aquifer, or in an aquifer and the underlying and overlying strata? If so, the hole must be large enough to allow for a multiple installation such as that shown in *Figure 7.15*.

5. Is the response time of the observation hole important? If a fast response is required, as may be the case when the hole is near to the test well and a non-equilibrium analysis is to be undertaken, then a hole with a small diameter should be used. Large wells can cause a lag in the observation of drawdown.

6. Are water quality samples to be taken from the hole? If so, the diameter and construction of the hole must be compatible with the use of a water sampler.

7. Would any other type of installation be less expensive but just as effective? In some locations drive point well screens (see Section 6.12.2) may make an adequate

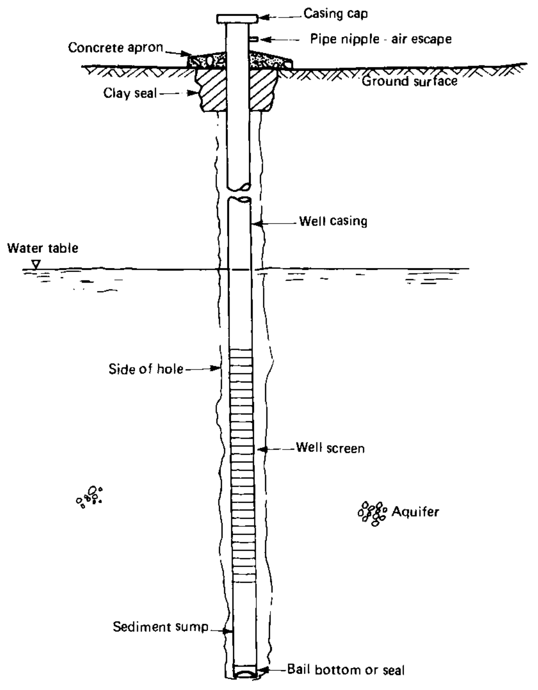

Figure 7.14 Features of a typical cased screened observation hole

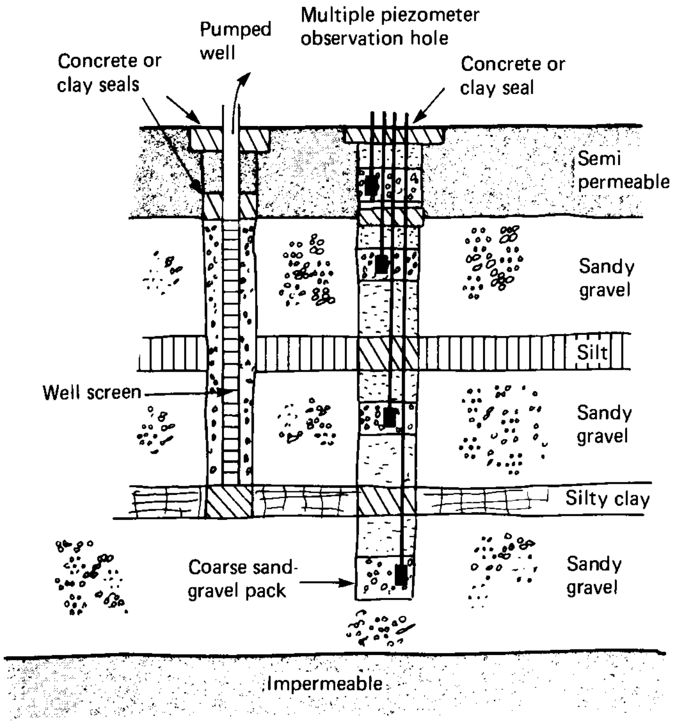

Figure 7.15 Schematic section of a multiple piezometer observation hole

observation hole. Sometimes an uncased hole can be used for the duration of a pumping test if the formations penetrated are competent and there appears to be little risk of collapse.

The depth of the observation holes

One of the most important factors in observation hole design is the depth. Obviously, the hole should be as shallow as possible on the grounds of economics, but it must be deep enough to ensure that the groundwater level never falls below the bottom of the screen. An observation hole that is too shallow and which is left high and dry in the cone of depression is of no use whatsoever. This problem is most acute with unconfined aquifers; it is unlikely that a confined aquifer would be operated in such a way as to desaturate the aquifer (see Section 6.3.1). For this reason every effort should be made to predict the range of drawdown likely to be experienced at the observation hole during both the pumping test and the operational life of the well. As described previously, the effect of hydrological boundaries on drawdown must be taken into consideration.

A commonly observed rule[3,4] for determining the depth of an observation hole is that the hole should be constructed to about the same depth as the middle of the screened section of the pumped well (*Figure 7.15*). The observation hole or piezometer should be equipped with a screened section about 1 to 2 m long. It is usually not necessary to have a screened section longer than this.

In complex situations where there is more than one aquifer separated by layers of relatively impermeable material, the piezometers or screens should be installed in the centre of each saturated strata. This also applies to the semi-permeable deposits overlying confined or semi-confined aquifers.

Construction of the observation holes

The construction of an observation hole is very similar to that for a well, as described in Section 6.8. The bored hole must be larger than the diameter of the casing to eliminate any problems due to lack of verticality or straightness. The annular space between the sides of the hole and the screen section should be filled with a uniform coarse sand. To prevent vertical leakage from one formation to another, a clay or cement seal should be placed adjacent to any clay layers. Kruseman and de Ridder[4] claimed that a plug of fine sand is almost as effective for this purpose. It is also essential that there is no seepage from the ground surface down the sides of the well casing. A surface seal or cap should therefore be provided. Brown *et al.*[18] suggested that the casing should extend about 0.7 to 1.2 m above the ground (*Figure 7.14*). A cased/screened hole should also be provided with a screen section about 1 to 2 m long and a sediment sump to prevent the screen from becoming blocked.

The casing and screen may be fabricated from various types of material. In some respects the type of material used depends upon the method of construction to be adopted, the geological conditions, the design life and the aggressiveness of the groundwater at the site. A plastic casing and screen would be adequate for many situations. An alternative type of installation may employ a piezometer (see *Figure 2.13*). This is cheaper than the type of construction described above, although not quite so versatile.

7.6 The conduct of a pumping test

7.6.1 Preparation

Before the test begins there are certain data that should be collected. These are listed below, with an indication of the accuracy required where appropriate[18]

1. A description of the well and observation hole locations. These data should be plotted on a suitable map.
2. The distance from the pumped well to each observation hole (± 0.5 per cent).
3. Details of the diameter, casing type, screen type, method of construction and any other salient features of all the wells and observation holes used in the test.
4. The total depth of all the wells and observation holes (± 1 per cent).
5. Details of the datum and method used to measure the depth to water in all the wells and observation holes.
6. The elevation of the datum at each well and observation hole (± 3 mm).
7. The standing water level in the wells and observation holes below the chosen datum (± 3 mm) should be recorded over a period of time before the pumping test starts. The length of this period should be twice the duration of the pumping test itself, or longer. This enables the antecedent trend of the water level to be determined, and the drawdown data recorded during the test corrected accordingly (see Section 7.7.1).
8. The barometric pressure before and during the test, as in (7) This enables the barometric efficiency of the aquifer to be assessed. Fluctuations in atmospheric pressure may cause changes in groundwater level, and the drawdown data must be corrected accordingly (see Section 7.7.2).

9. Rainfall data. These data are needed so that any physical recharge of groundwater can be distinguished from increases in water level due to pressure changes (as in (7) and *Table 7.3*) or the effect of recharge boundaries. If possible the test should be conducted when heavy rain is least likely.

While these data are being collected, all the observation holes that are to be used during the pumping test should be inspected and subjected to a short inflow test. The aim of an inflow test is to check the responsiveness of each observation hole and to ensure that it is in effective hydraulic contact with the aquifer and has not become blocked. This is particularly important with old observatiom holes that have not been used for some time (see *Figure 7.16*). The response can be tested by injecting a known quantity of water into the hole and measuring the subsequent decline in water level. After a period of about 3 to 4 h (generally much less) the water level should have fallen back to within about 3 mm of its original level. If it has not, the observation hole will be unable to reflect satisfactorily changes in water level during the pumping test.

While the inflow tests are being conducted on the observation holes, the equipment required for the pumping test should be assembled. This includes the devices to be used to measure the water levels in the wells, such as a pressure transducer and Y/T chart recorder (which is a useful combination for monitoring 'slug tests' as well), autographic recorder, electrical circuit, tape or some other means. These instruments should be checked to make sure that they are in good working order. The personnel who have to operate the equipment should be fully briefed to ensure that they are proficient in its use. Once a test starts it cannot be halted because someone does not know how to operate a piece of equipment.

At the same time as the water level instrumentation is being checked, the pumping plant should be assembled on site. Basically this consists of the pump itself, a gate valve

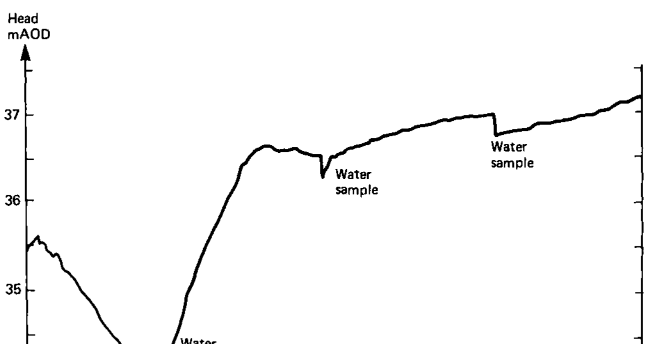

Figure 7.16 Hydrograph of an observation hole that has become blocked above the screen section. During 1974 and most of 1975 the hole appears to have been functioning normally and the hydrograph exhibits a typical seasonal variation in groundwater level. Additionally, the water level recovers relatively quickly following the extraction of a water sample in January 1975. However, after October 1975 the seasonal variation of water level is no longer in evidence and the trend of the hydrograph is progressively upward, due to the fact that the hole had become blocked and rainwater was trickling down the inside of the well casing. Note that it takes a long time for the groundwater level to recover following the removal of a sample in July 1976. This gives a clear indication that the hole is not in hydraulic contact with the aquifer

and some means of measuring the discharge from the pump. The aim during the test is to maintain a reasonably constant discharge with ideally no more than a 5 per cent variation[5]. Electrically powered pumps are to be preferred, since engines (even with automatic speed control) often produce variations in discharge of as much as 25 to 50 per cent and need to be checked more frequently[18]. The gate valve, which should be installed on the pump discharge pipe, is used to control the flow from the pump. This is necessary for two reasons. First, the discharge from the pump will decline as the water level in the well falls and the lift increases. Thus, the valve should be partially closed initially and progressively opened during the test so as to maintain a constant discharge. Secondly, if a step drawdown or variable discharge pumping test is required, the valve initially should be partly closed and opened up incrementally at suitable time intervals. In order to monitor the discharge from the pump some form of flow measuring device is required. This frequently takes the form of an orifice meter, flow meter, or V-notch set into the side of a rectangular tank[28,31]. Whatever the device adopted, it should be capable of measuring the discharge easily and accurately over the range expected. The measuring device should be calibrated prior to the test[3,5,32].

7.6.2 Implementation

If a pumping test is to progress smoothly and efficiently it is necessary to have a clear idea beforehand of the sequence of operations and of the data that must be collected at each stage. It is a good idea to duplicate essential equipment, where practical, in case of breakdown. If the discharge from the well is stopped, for any reason, during the first 24 h of a pumping test, then groundwater levels should be allowed to recover and the test re-started. Once a test has been in progress for 24 h, a break in pumping of up to 1 h may be acceptable, otherwise pumping should be continuous.

Immediately prior to the commencement of the test the static water level in the pumped well and the observation holes should be recorded. The test itself starts when the pump is first switched on, and this should be recorded as the time-zero of the test. To begin with, it is desirable to have at least one person at each observation hole and two at the well. The coordination and smooth running of the test can be improved by issuing the personnel involved with portable two-way radios. For example, if the test fails to start at the appointed time, the observers in the field can be informed of this. Otherwise the drawdown may be recorded at the agreed time intervals, but from an incorrect time-zero.

After the pump has been switched on the gate valve should be adjusted to obtain the desired constant discharge. During the first few minutes the pump discharge may fluctuate quite considerably as the initial drawdown is quite rapid, so this must be monitored carefully. The discharge should then be checked at specified intervals[18,28], such as 5, 10, 20, 30, 60, 120, 240, 480, 720 and 1440 min, and at daily intervals thereafter, or as necessary.

The drawdown in the well and the observation holes should be recorded at intervals such as 1, 1.5, 2, 2.5, 3, 4, 5, 6, 7, 8 and 10 min for the first 10 min, 10, 15, 20, 25, 30, 40, 50, 65, 80 and 100 min for the first 10 to 100 min, and subsequently at 100 min intervals until completion.

The increased frequency of observations near the start of the test is partly required because the drawdown increases most rapidly during this period, but is also necessary in order to achieve a uniform distribution of data points when plotted to a log scale. After the first two hours of the test the number of personnel involved may be reduced, one person then having sufficient time to travel to several observation holes to take readings. The observations need not be recorded simultaneously or at precisely predetermined time intervals, as long as the correct interval from time-zero and the corresponding drawdown are recorded. All the relevant information should be logged on record sheets specifically designed for the purpose[28,33].

7.6.3 Duration of the pumping test

To some extent the duration of a pumping test should be decided from an appraisal of the data collected while it is in progress and it should not be terminated on the basis of a rigid, pre-conceived idea of what the most suitable length would be. The test should be continued until it is concluded that a stable drawdown (or a constant $(s_1 - s_2)$) value has been obtained, or until the objectives of the test have been obtained and there is insufficient reason for continuing. In general the length of test required depends upon

1. The type and nature of the aquifer; for instance, equilibrium will be more quickly established in a confined aquifer than in an unconfined aquifer.
2. The type of analysis to be applied, and whether or not a stable drawdown must be established.
3. The accuracy required. Usually, the longer the test the more reliable are the data obtained.

Additionally it must be remembered that the cost of constructing the observation holes and assembling the plant and manpower for the test generally far outweighs the cost of actually conducting the test. In the latter stages of a test the number of personnel involved may be reduced to as few as two, so the exercise may as well be allowed to continue until it is decided that there is nothing more to be gained. This may mean until the cone of depression has stabilized, since this allows the non-equilibrium and equilibrium analyses to be used, so a cross check on results is possible. The longer period of pumping also facilitates the evaluation of hydrological boundaries and the effect of abstraction on the environment, and generally improves the chances of achieving the objectives listed in Section 7.5.1. Kruseman and de Ridder[4] and Anon[5] suggested that the duration of continuous pumping required, under average conditions, to reach a steady state flow situation is

Confined aquifer	24 h
Semi-confined aquifer	15 to 20 h
Unconfined aquifer	72 h

The length of the test should be extended to 30 h in fine sand and to 170 h in silt and clay (to allow for delayed drainage). However, if the pumping rate is large the duration of the test should be increased to at least 4 days (up to 3000 m^3/day), 7 days (up to 5000 m^3/day), and 10 days (over 5000 m^3/day). According to Anon[26] the average length of a pumping test in Britain is about two weeks (336 h).

It is recommended that a running plot of the test data is maintained as an indication of how long the test should continue. Once the cone of depression has stabilized, the next phase of the test, such as an increase in the pumping rate, or a recovery test, may be implemented.

7.6.4 Conduct of a recovery test

A recovery test involves the analysis of the rise in groundwater level that results when the pump is switched off. The recovery can be imagined to be the consequence of a hypothetical recharge well which is switched on at the same instant that the abstraction well is, in reality, switched off. If the imaginary recharge well and the real abstraction well are operating at the same rate, then by superposition, the two wells cancel each other out, so causing the groundwater recovery.

The advantages of recovery tests lie in the fact that the actual discharge from a well can vary considerably during a pumping test, whereas during the recovery phase it is assumed that the recharge well operates at a constant rate equal to the mean discharge during the drawdown phase of the test. Moreover, a recharge test costs very little to conduct, yet enables estimates of the coefficients of transmissivity and storativity to be obtained. Consequently it is common practice to incorporate a recovery test into the ideal pump test programme (see *Figure 7.13(e)*).

To initiate a recovery test the pump should be switched off (which becomes time zero for the recovery phase) and the subsequent increase in water level in the aquifer recorded. The procedure for this sort of test is exactly the same as for a standard pumping test. It must be remembered that in this case the water level recovery will be most rapid in the period just after the pump is stopped and the frequency of the observations should reflect this. Water level data from the pumped well may not be accurate during the early part of the test as a result of reverse flow down the discharge column when the pump is stopped. The analysis of recovery data is described in Sections 7.10.4.

7.6.5 Conduct of a step drawdown test

A step drawdown test, as the name infers, is a test in which the drawdown at a well is recorded while the discharge is increased incrementally. The change in discharge between steps should be made as quickly as possible, while the discharge should be constant throughout each step. Ideally, a step drawdown test should comprise four or more steps each of about 3 h duration, although shorter steps may be adequate[7,8,9]. The purpose of such tests is generally to evaluate the well loss (see Section 7.3), so that drawdown data from the pumped well can be corrected prior to analysis. The step drawdown test is also of value in forming an assessment of a suitable discharge for a subsequent constant rate pumping test (see *Figure 7.13(e)*) and when deciding the most appropriate distance to the observation holes. The approximate value of the coefficient of transmissivity can be calculated from the results of the test and, using some methods of analysis, an estimate of the coefficient of storage can also be obtained. An example of typical data resulting from a step drawdown test, and its analysis, is given in Section 7.10.5.

7.7 Correction of pumping test data prior to analysis

If a pumping test lasts for several days, it is quite probable that the drawdown data observed during this period will be inconsistent or in some way incorrect. There are various means by which errors and inconsistencies can be introduced and one of the first steps, prior to commencing the actual analysis of the data, is to correct the observations recorded during the test. The factors which must be considered are described below.

7.7.1 Antecedent trend

Observations of the drawdown recorded during a pumping test are generally measured from the rest water level that existed just before the pump was switched on. However, this ignores the fact that the rest water level may have been changing before the test started and that it would not necessarily be constant over the period of the test. Thus, the antecedent trend, or the change in rest water level that would have occurred naturally over the period of the test, must be taken into consideration. An example is shown in *Figure 7.17*, which illustrates a situation where the rest water level is rising prior to the test. The drawdown during the test must, therefore, be measured from the projected rest water level and not just from the level immediately before the pump was switched on.

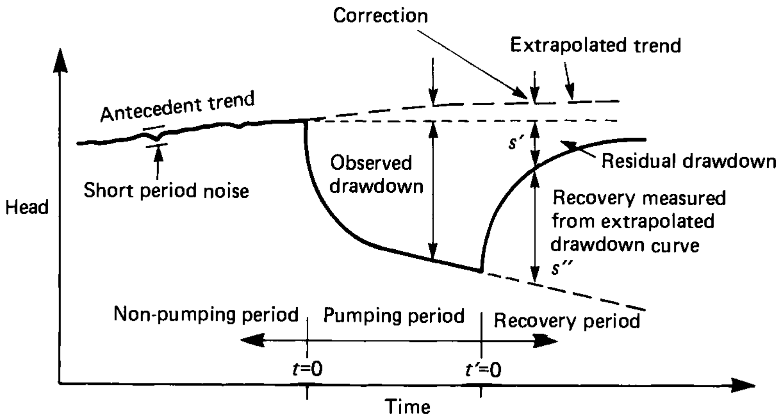

Figure 7.17 Hydrograph from a hypothetical observation hole. The true drawdown during the pumping test is the observed drawdown plus the natural change in water level estimated from the antecedent trend by extrapolation. The antecedent trend may also affect the calculation of the residual drawdown, s', during the recovery test. Additionally, the recovery is measured from the extrapolated drawdown curve recorded before the pump was stopped

When conducting a recovery test the objective is to monitor the calculated recovery, s'', which is the difference between the extrapolated drawdown at the end of the pumping period, and the recovery recorded after the pump was switched off (*Figure 7.17*).

7.7.2 Short period fluctuations in water level (noise)

Slow, long-term changes in groundwater level are often the result of variations in groundwater storage, whereas rapid increases or decreases in level over short periods of time may be due to pumping or pressure effects (see *Table 7.3*). The fluctuations induced by barometric pressure changes may be particularly significant and the drawdown data from pumping tests must be corrected accordingly. Under these circumstances care must be taken to distinguish between changes in groundwater level caused by pressure effects and those caused by hydrogeological boundaries, which can be similar in appearance.

Changes of water level as a result of variations in atmospheric pressure are most apparent with confined aquifers, which act rather like huge barometers. The pressure of the atmosphere is shared between the rock skeleton and the water confined in the aquifer, with the result that the water pressure in the aquifer is less than atmospheric. Consequently if there is an increase in atmospheric pressure the water level in a well tapping a confined aquifer decreases, since the atmospheric pressure on the free water surface in the well is greater than the pressure of water in the aquifer.

The barometric efficiency of an aquifer is the ratio of the change in water level in the well to the inverse of the equivalent change in a water barometer. Some aquifers may have barometric efficiencies as high as 85 per cent, although values between 20 and 75 per cent are more usual. Barometric efficiency may be calculated by selecting a distinctive fluctuation in atmospheric pressure, converting the barometer readings to metres of water (1 mbar = 10.2 mm of water = 0.75 mm of mercury) and then constructing an inverted plot of the data on graph paper (*Figure 7.18*). The water level data for the same period are plotted and the barometric efficiency is calculated by using

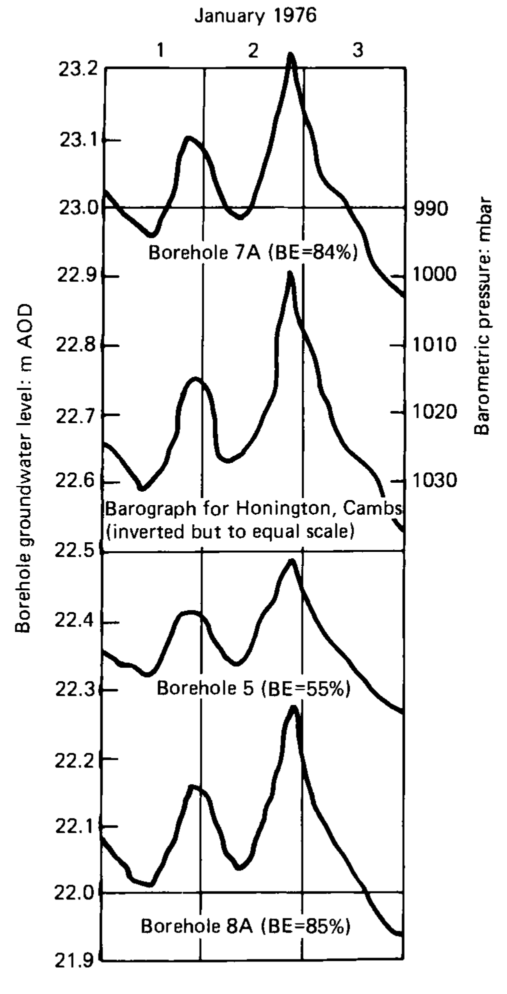

Figure 7.18 Response of selected boreholes in a semi-confined chalk aquifer in East Anglia to changes in atmospheric pressure. The barometric efficiency (BE) of the boreholes in the diagram range from 55 to 85 per cent (after Foster and Robertson[34])

the ratio

$$BE = \frac{\Delta h}{\Delta AP} \times 100 \tag{7.30}$$

where BE is the barometric efficiency (%), Δh is the change in water level (in metres of water) as a result of ΔAP, the change in atmospheric pressure (in metres of water). For further details see Jacob[35] and Walton[14]. Alternatively, the following correction can be applied to water level observations

$$\Delta h = \frac{BE \cdot \Delta AP}{100} \tag{7.31}$$

Changes in atmospheric pressure are the most common cause of short-term fluctuations in water level, but certainly not the only cause (*Table 7.3*). Variations in tidal level, river stage, or any other form of external loading may have a similar effect. Unconfined aquifers may exhibit diurnal fluctions in water level, particularly if the water table is near the ground surface, as a result of evapotranspiration losses (see Todd[19]). The net effect of all these fluctuations is to superimpose short period 'noise', or unwanted complications, on groundwater hydrographs and thus introduce additional uncertainties into the analysis of pumping test data.

7.7.3 Correction for interference from other wells

It is quite possible that some of the observation holes used during a pumping test may lie within the area of influence of other wells and that these wells cannot be shut down prior to and during the test. In such cases any change in the pumping rate of these interfering wells will have an affect on the drawdown recorded in the observation holes, which means that an appropriate correction must be applied.

When two wells are mutually interfering, the general rule is that the total drawdown is the sum of the individual drawdowns (*Figure 7.19*). This is sometimes called the

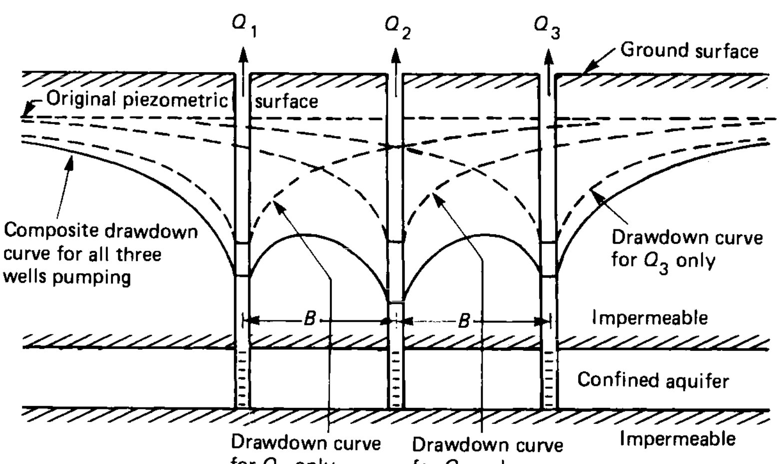

Figure 7.19 Individual and composite drawdown curves for three mutually interfering wells in a line illustrating the principle of superposition (after Todd[19])

principal of superposition. Therefore, if the drawdown equivalent to the pumping rate (or change in the pumping rate) of the interfering well is known, this can be subtracted or added to the drawdown recorded during the test, as appropriate.

7.7.4 Correction for partial penetration and other complications

The effect of partial penetration is to increase the drawdown in the pumped well and any nearby observation hole compared to that which would be recorded with a well that completely penetrates the aquifer. However, if the nearest observation hole is at least twice the thickness of the aquifer from the well, the effects of partial penetration can generally be ignored provided that drawdown data from the pumped well are not used in the analysis. This does assume, of course, that the thickness of the aquifer is known, which may not be the case with a partially penetrating well. If an observation hole is located within the area where the effects of partial penetration are significant, then a correction factor can be applied to the observations of drawdown. Again, the thickness of the aquifer is needed to calculate the correction factor. Methods of correction for partial penetration have been devised by Hantush[22,36], Jacob[37], Huisman[38] and others. For further details see Kruseman and de Ridder[4].

If the data from a pumped well are to be analysed, this must first be corrected for the well loss. This is the loss of head or additional drawdown that is experienced in the vicinity of the well as a result of turbulent flow (see Section 7.3). The well loss can be evaluated using a step drawdown test (see Sections 7.6.5 and 7.10.5).

It is also possible to apply corrections for almost any violation of the limiting assumptions listed in Section 7.4. For instance, corrections may be used to allow for varying aquifer thickness[22], the initial slope of the watertable, variable discharge rate[24,39,40,41], storage in the pumped well[42] and so on. A review of these techniques was provided by Kruseman and de Ridder[4].

7.8 Boundary analysis using image wells

One of the assumptions of the basic method of pumping test analysis is that the aquifer is infinite in areal extent and has no boundaries. This is often unrealistic, but no appreciable error results unless the well is located close to a boundary when the drawdown will be either greater or less than expected, depending upon whether the boundary forms a barrier to flow or provides additional recharge (see *Figure 7.20*). In such instances an aquifer of finite extent can be transformed to one of infinite extent by the use of images.

An image well is an imaginary well introduced to create a hydraulic flow system which will be equivalent to the effects of a known physical boundary on the flow system. For example, *Figure 7.20(a)* shows an imaginary recharge well placed opposite and at the same distance from a stream as the real well. The image well operates simultaneously at the same rate of discharge as the real well, so that the cone of impression from the recharge well and the cone of depression from the discharging well cancel exactly along the line of the stream. This gives a constant head along the stream which is exactly equivalent to the constant elevation of the stream forming the aquifer boundary. The comparable system for a barrier boundary is illustrated in *Figure 7.20(b)*.

The method of images is often applied to the data obtained from pumping tests with

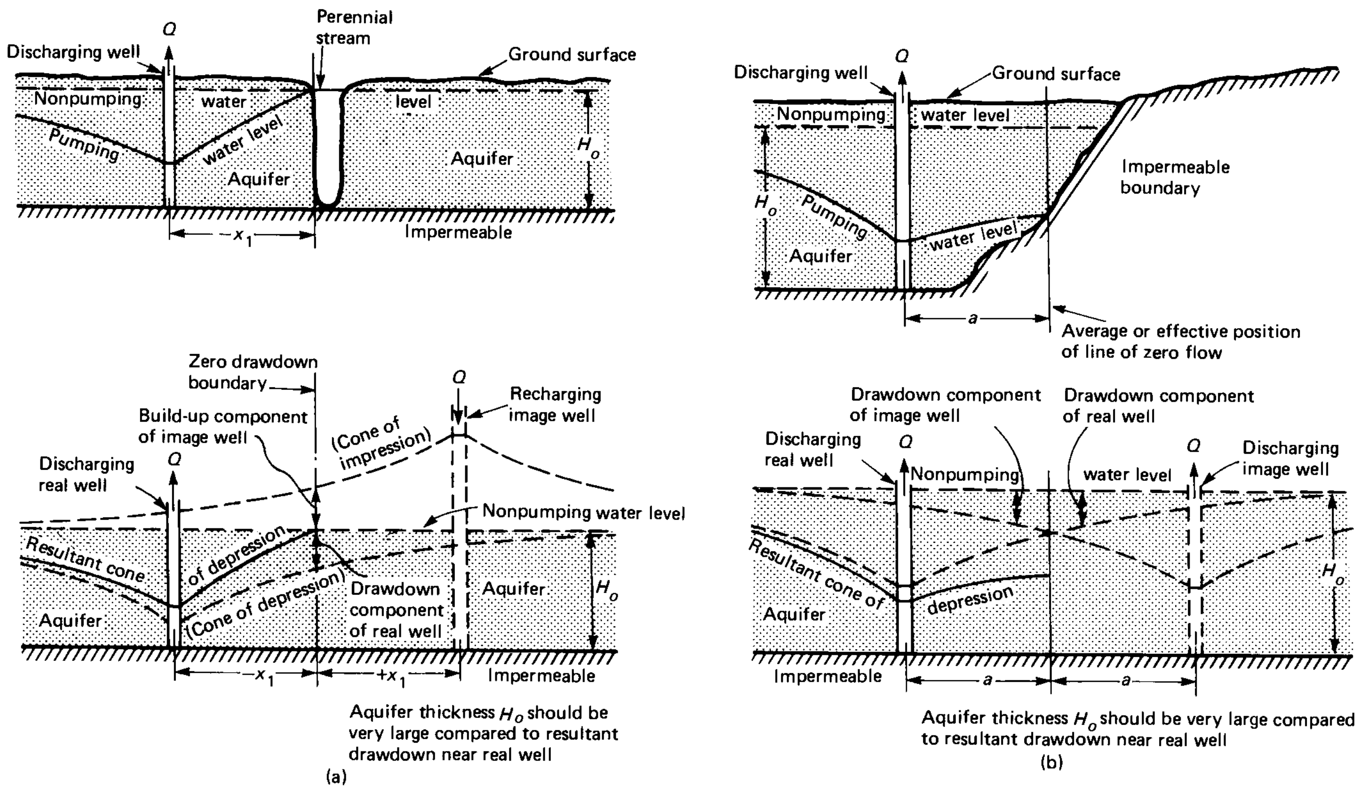

Figure 7.20 Schematic cross section of an aquifer with a straight boundary and the equivalent image well system. (a) Well near a recharge boundary. (b) Well near an impermeable or barrier boundary (after Ferris *et al.*[21])

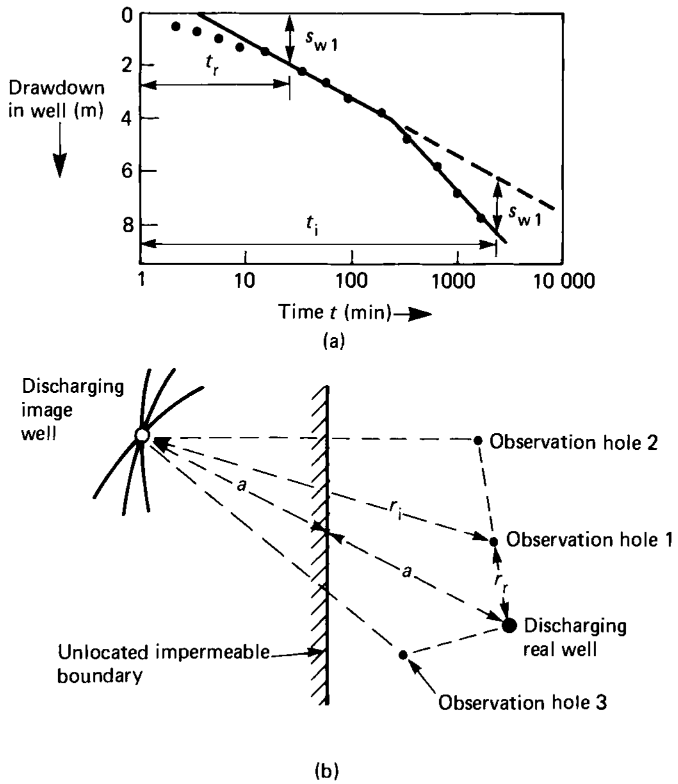

Figure 7.21 Application of the image well technique to an impermeable boundary. (a) Jacob semi-logarithmic plot of the time-drawdown data showing the effect of the boundary. (b) Location of the image well and the boundary

the objective of locating any boundaries within the area of influence[43]. Changes in the slope of the time–drawdown data (*Figure 7.21*(*a*)) indicate the presence of boundaries, which can then be located by applying the following formula to the time–drawdown curves obtained from at least three boreholes. Considering observation hole 1 in *Figure 7.21*(*b*), then

$$r_i = r_r \sqrt{\frac{t_i}{t_r}} \tag{7.32}$$

where r_i is the distance from image well to observation hole 1, r_r is the distance from real pumped well to observation hole 1, t_r is the time after pumping started, before the boundary becomes effective, for a particular drawdown (s_{w1} in *Figure 7.21*(*a*)) to be observed in the real well and t_i is the time after pumping started, after the boundary becomes effective, when the divergence of the time–drawdown curve caused by the image well is equal to s_{w1}.

Having determined the distance from each of the three observation holes to the image well, these distances can be set out on a map using a pair of compasses. The intersection of the three arcs gives the location of the image well. The boundary is equidistant between the image and the real pumped well.

Image well theory is quite versatile and can be applied to complex situations where multiple image wells are used to simulate a number of boundaries; for instance, two impermeable boundaries which are mutually perpendicular, or an impermeable boundary intersected by a recharge boundary such as a stream. For further details see Ferris *et al.*[21], Walton[14] and Chan[44]. The problem of wells near rivers and the proportion of well discharge derived from the surface source will be considered in more detail in Chapter 8.

7.9 Extension of pumping test data

Once a pumping test has been completed, it may be necessary to predict the drawdown that would have resulted from a longer period of pumping or from a larger abstraction rate. In other words, the pumping test data must be extrapolated. This can be done quite easily. Suppose that a drawdown–log time graph at an observation hole has been plotted (*Figure 7.22(a)*) and it is apparent that

$Q = 1\ m^3/min$ for 300 min
$s = 2.3$ m after 300 min of pumping at an observation hole 15 m from the well
$\Delta s = 1.7$ m/log cycle

The drawdown–log distance graph can be obtained by drawing a line with a slope of 3.4 m/log cycle through a point representing a distance of 15 m and a drawdown of 2.3 m (see *Figure 7.22(b)*), since the slope of a semi-log distance–drawdown graph is twice that of the semilog time–drawdown graph (see Section 7.4.3). If the distance–drawdown relationship for some other pumping rate is required, this can be calculated quite easily because the drawdown in a well or observation hole is proportional to the discharge so the effect of any other abstraction rate can be determined pro-rata as follows

Pumping rate (m^3/min)	*Drawdown at 15 m observation hole after 300 min* (m)	Δs m/log cycle
1	2.3	3.4
2	4.6	6.8
5	11.5	17.0

These data can be plotted on a graph and used for various purposes, such as to assess the interference at another well that will result from various rates of abstraction (*Figure 7.22(b)*). If interference data for periods of continuous pumping longer than 300 min in duration are required, then this can be obtained by extrapolating the line on the semi-log time–drawdown graph to find the drawdown at any particular time. For example, if after 1000 min of pumping at 1 m^3/min the drawdown in the 15 m observation hole was 3.2 m, this too can be plotted on the drawdown–log distance graph. The gradient of this line would still be twice that of the drawdown–log time graph, that is 3.4 m/log cycle. Hence the two lines on the semi-log distance–drawdown graph representing the effects of 300 min and 1000 min of pumping would be parallel. If these lines are extrapolated to the axis representing zero drawdown, the radius of influence of the well for differing periods of abstraction can be assessed (*Figure 7.22(c)*).

There are many other ways in which the results of a pumping test can be extended.

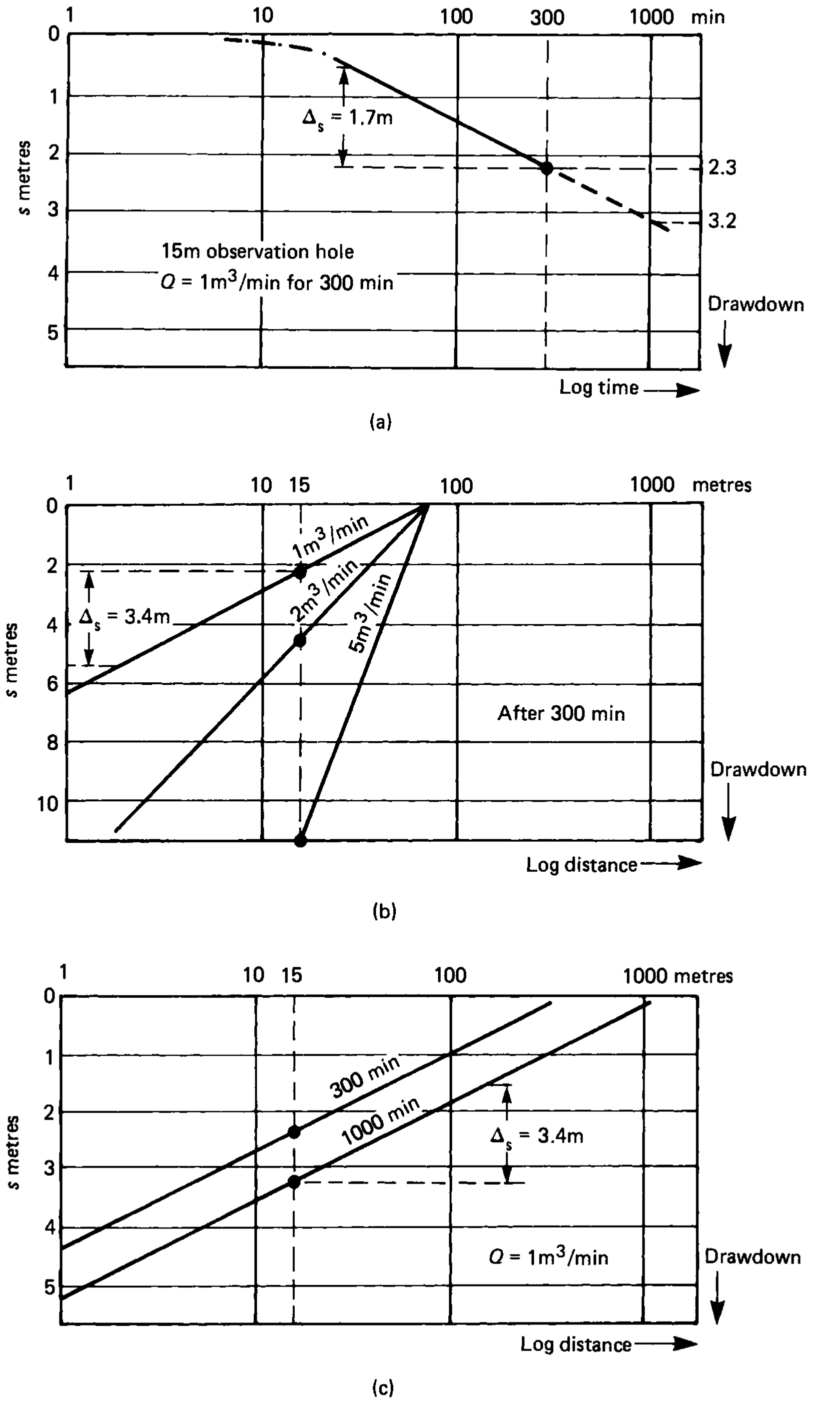

Figure 7.22 Illustration of the extension of pumping test data. (a) Jacob semi-log time-drawdown graph. (b) Semi-log distance-drawdown graph for various discharges. (c) Semi-log distance-drawdown graph for differing periods of abstraction

The well discharge equations (Equation (7.6) onwards) may also be useful for this purpose. However, when extending data in this fashion, the possible complicating effect of boundaries should be remembered. It has been assumed above that the aquifer is infinite, which is generally not the case.

7.10 Examples of pumping test analysis

7.10.1 Theis type curve analysis

Table 7.5 shows the drawdown recorded in a confined aquifer at an observation hole whcih is 100 m from a well that is discharging at a rate of 1500 m^3/day. Column 3 of the tables gives the calculated values of t/r^2. The observed drawdown, s, is plotted against t/r^2 in *Figure 7.23*, which also shows the superimposed type curve plot of $W(u)$ against $1/u$ obtained from *Table 7.6*. Two match points have been selected in *Figure 7.23*, one relating $W(u)$ and s, and the other $1/u$ and t/r^2. Thus the following values are obtained

$$W(u) = 10^0 = 1$$

$$s = 0.690 \text{ m}$$

$$1/u = 10^1 = 10$$

$$t/r^2 = \frac{2.87 \times 10^{-3}}{1440} \text{ days/m}^2$$

TABLE 7.5. Pumping test data for Theis analysis

Time since pumping started t (min)	*Observed drawdown* s (m)	*Calculated* t/r^2 (days/m^2)
2.0	0.100	2.0×10^{-4}
2.5	0.137	2.5×10^{-4}
3	0.183	3.0×10^{-4}
4	0.274	4.0×10^{-4}
5	0.335	5.0×10^{-4}
6	0.396	6.0×10^{-4}
7	0.488	7.0×10^{-4}
8	0.579	8.0×10^{-4}
10	0.625	1.0×10^{-3}
15	0.792	1.5×10^{-3}
20	1.036	2.0×10^{-3}
25	1.189	2.5×10^{-3}
30	1.250	3.0×10^{-3}
40	1.463	4.0×10^{-3}
50	1.646	5.0×10^{-3}
65	1.829	6.5×10^{-3}
80	1.951	8.0×10^{-3}
100	2.103	1.0×10^{-2}
200	2.606	2.0×10^{-2}
300	2.957	3.0×10^{-2}
400	3.109	4.0×10^{-2}
500	3.261	5.0×10^{-2}

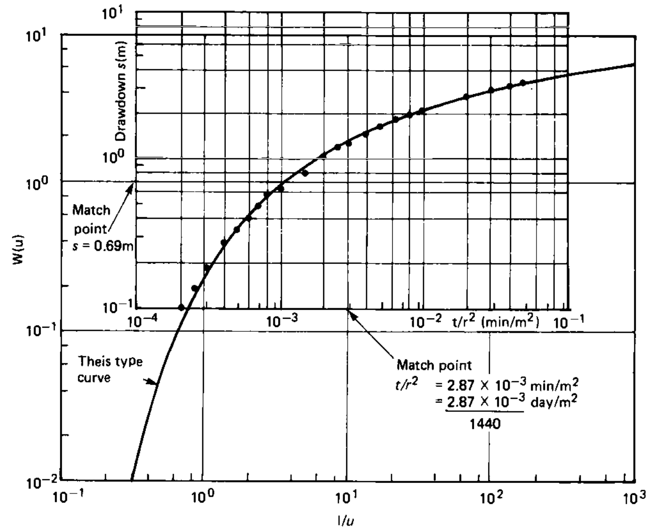

Figure 7.23 Theis analysis

Substituting these values in Equations (7.16) and (7.17) gives

$$T=\frac{Q}{4\pi s}W(u)=\frac{1500}{4\times 3.142\times 0.690}\times 1=173\ \mathrm{m}^2/\mathrm{day}$$

$$S=4T(t/r^2)u=\frac{4\times 173\times 2.87\times 10^{-3}}{1440}\times\frac{1}{10}=0.000138$$

7.10.2 Jacob analysis

Using the same data as in Section 7.10.1, but this time using a semi-logarithmic plot of drawdown against time (*Figure 7.24*) the following values can be obtained from the diagram

$\Delta s = 1.6$ m

$t_0 = 4.9/1440$ days

Substituting these values in Equations (7.22) and (7.23) gives

$$T=\frac{2.30Q}{4\pi\,\Delta s}=\frac{2.30\times 1500}{4\times 3.142\times 1.6}=172\ \mathrm{m}^2/\mathrm{day}$$

$$S=\frac{2.25Tt_0}{r^2}=\frac{2.25\times 172\times 4.9}{100^2\times 1440}=0.000132$$

7.10.3 Chow analysis

The time–drawdown data of *Table 7.5* has been plotted semi-logarithmically in *Figure 7.25*. The data form a curve during the early part of the test, but approximate to a straight line as the test progresses. An arbitrary point has been chosen on the curve and the tangent to the curve at that point constructed. The drawdown per log cycle of time of the tangent line can then be determined. From *Figure 7.25* it is apparent that for the

TABLE 7.6. Values of $W(u)$ for given values of $1/u$ and u

$1/u$		$\times 1$	$\times 10^1$	$\times 10^2$	$\times 10^3$	$\times 10^4$	$\times 10^5$	$\times 10^6$	$\times 10^7$	$\times 10^8$	$\times 10^9$	$\times 10^{10}$
	u	$\times 1$	$\times 10^{-1}$	$\times 10^{-2}$	$\times 10^{-3}$	$\times 10^{-4}$	$\times 10^{-5}$	$\times 10^{-6}$	$\times 10^{-7}$	$\times 10^{-8}$	$\times 10^{-9}$	$\times 10^{-10}$
1.000	1.0	0.2194	1.823	4.038	6.332	8.633	10.94	13.24	15.54	17.84	20.15	22.45
0.833	1.2	0.1584	1.660	3.858	6.149	8.451	10.75	13.06	15.36	17.66	19.96	22.27
0.667	1.5	0.1000	1.465	3.637	5.927	8.228	10.53	12.83	15.14	17.44	19.74	22.04
0.500	2.0	0.04890	1.223	3.355	5.639	7.940	10.24	12.55	14.85	17.15	19.45	21.76
0.400	2.5	0.02491	1.044	3.137	5.417	7.717	10.02	12.32	14.62	16.93	19.23	21.53
0.333	3.0	0.01305	0.9057	2.959	5.235	7.535	9.837	12.14	14.44	16.74	19.05	21.35
0.286	3.5	0.006970	0.7942	2.810	5.081	7.381	9.683	11.99	14.29	16.59	18.89	21.20
0.250	4.0	0.003779	0.7024	2.681	4.948	7.247	9.550	11.85	14.15	16.46	18.76	21.06
0.222	4.5	0.002073	0.6253	2.568	4.831	7.130	9.432	11.73	14.04	16.34	18.64	20.94
0.200	5.0	0.001148	0.5598	2.468	4.726	7.024	9.326	11.63	13.93	16.23	18.54	20.84
0.167	6.0	0.0003601	0.4544	2.295	4.545	6.842	9.144	11.45	13.75	16.05	18.35	20.66
0.143	7.0	0.0001155	0.3738	2.151	4.392	6.688	8.990	11.29	13.60	15.90	18.20	20.50
0.125	8.0	0.00003767	0.3106	2.027	4.259	6.555	8.856	11.16	13.46	15.76	18.07	20.37
0.111	9.0	0.00001245	0.2602	1.919	4.142	6.437	8.739	11.04	13.34	15.65	17.95	20.25

Examples: If $1/u = 0.400 \times 10^2$, then $W(u) = 3.137$

If $u = 5.0 \times 10^{-6}$, then $W(u) = 11.63$

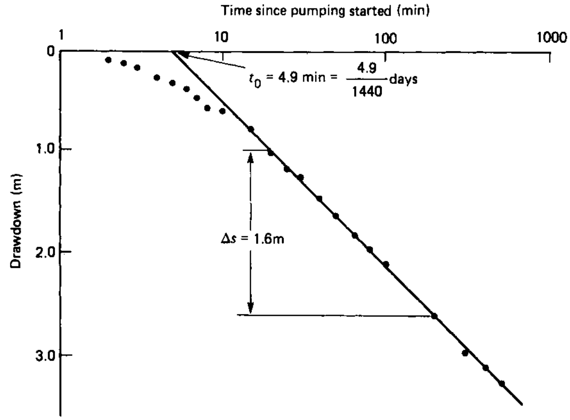

Figure 7.24 Jacob analysis

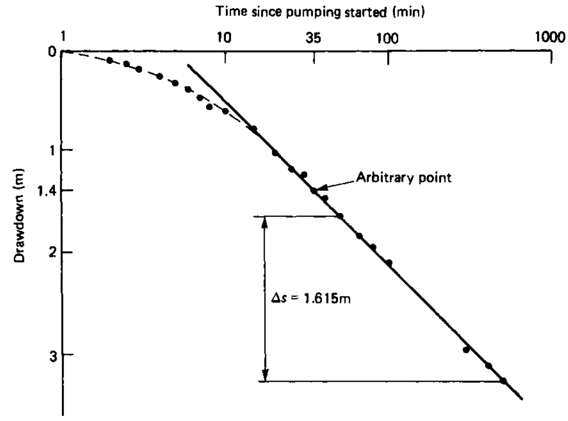

Figure 7.25 Chow analysis

arbitrary point selected

$s = 1.4$ m when $t = 35$ min or 35/1440 days

$\Delta s = 1.615$ m

so

$$F(u) = \frac{s}{\Delta s} = \frac{1.4}{1.615} = 0.867 \tag{7.26}$$

From a nomogram of $F(u)$ against $W(u)$ and u, as shown in *Figure 7.12*, the values of $W(u)$ and u can now be determined. These values can also be calculated from the following equation using *Table 7.6*

$$F(u) = \frac{W(u)e^u}{2.30} \tag{7.33}$$

In either case, the values of the two variables are found to be

$W(u) = 1.79$

$u = 0.11$

With a pumping rate of 1500 m^3/day and an observation hole at a distance of 100 m from the well, as in Section 7.10.1, substitution of the values of s, $W(u)$, u and t into Equations (7.27) and (7.28) enables the coefficients of transmissivity and storage to be calculated

$$T=\frac{Q}{4\pi s}W(u)=\frac{1500}{4\times 3.142\times 1.4}\times 1.79=153\ \mathrm{m^2/day}$$

$$S=\frac{4uTt}{r^2}=\frac{4\times 0.11\times 153\times 35}{100^2\times 1440}=0.000164$$

7.10.4 Recovery analysis

The coefficients of transmissivity and storage can be determined from the analysis of the groundwater recovery curve, which is a graph depicting the increase in water level with time following the cessation of pumping. This, effectively, is the inverse of a time–drawdown curve.

Table 7.7 shows the drawdown and recovery data recorded in a confined aquifer at an observation hole 100 m from a well discharging at a rate of 1500 m^3/day. The analysis commences by obtaining the recovery, s'', measured from the extrapolated drawdown curve as shown in *Figure* 7.26. Thus, the recovery or increase in water level is measured from a line which represents what the water level would have been if the well had continued pumping at the same rate. The recovery is not measured from the lowest point of the drawdown curve, since the antecedent trend before pumping stopped must be taken into consideration (as with a normal pumping test). The calculated recovery at various times after pumping ended is shown in *Table* 7.7. These data can be plotted as a drawdown–log time graph (*Figure* 7.27) and subject to analysis using the Jacob technique (see Section 7.4.3 and 7.10.2). By employing Equations (7.22) and (7.23), the coefficients of transmissivity and storage may be calculated as follows

$$T=\frac{2.30Q}{4\pi\,\Delta s''}=\frac{2.30\times 1500}{4\times 3.142\times 1.6}=172\ \mathrm{m^2/day}$$

$$S=\frac{2.25Tt'_0}{r^2}=\frac{2.25\times 172\times 5.0}{100^2\times 1440}=0.000134$$

The data could also be analysed using the Theis type curve technique (see Sections 7.4.2 and 7.10.1).

7.10.5 Step drawdown test analysis

One approach to the analysis of step drawdown data[8,12] is to evaluate Equation (7.4) which is

$$s_w/Q=B+CQ$$

or

$$s_w=BQ+CQ^2$$

The first stage of the analysis involves plotting the test data as a drawdown–log time graph as shown in *Figure 7.28*. The drawdown during each step of the test is

TABLE 7.7. Drawdown and groundwater recovery data

Time since pumping started, t (min)	*Drawdown s* (m)	*Time since pumping stopped, t′* (min)	*Calculated recovery, s″*	*Comment*
0	0	—	—	Drawdown phase
5	0.290	—	—	
10	0.442	—	—	
15	0.853	—	—	
20	1.030	—	—	
30	1.280	—	—	
40	1.463	—	—	
50	1.615	—	—	
65	1.768	—	—	
80	1.920	—	—	
100	2.057	—	—	
150	2.377	—	—	
200	2.606	—	—	
250	2.804	—	—	
300	2.956	—	—	
350	3.078	—	—	
400	3.170	—	—	
450	3.255	—	—	
500	3.307	—	—	
505	2.972	5	0.375	Recovery phase
510	2.804	10	0.550	
515	2.621	15	0.725	
520	2.454	20	0.900	
530	2.210	30	1.140	
540	1.981	40	1.400	
550	1.829	50	1.550	
565	1.585	65	1.825	
580	1.433	80	1.975	
600	1.274	100	2.150	
650	0.975	150	2.500	
700	0.823	200	2.700	
750	0.716	250	2.825	
800	0.671	300	2.910	
850	0.625	350	2.975	
900	0.610	400	3.050	

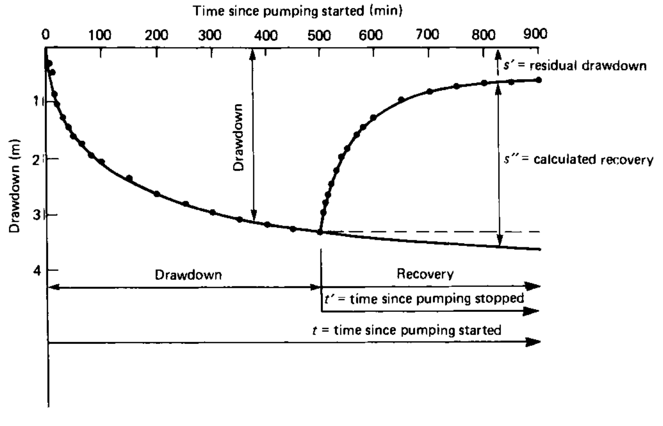

Figure 7.26 Recovery analysis

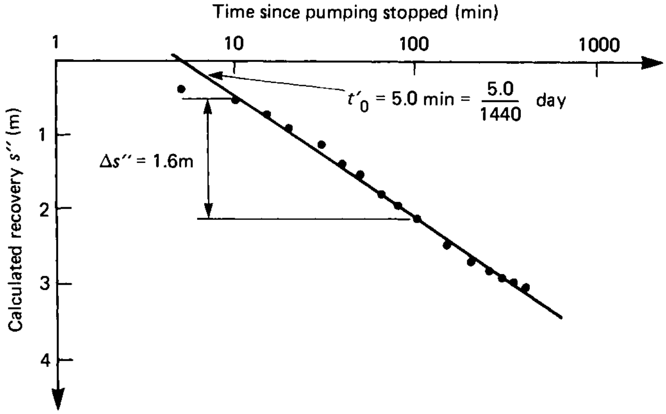

Figure 7.27 Analysis of recovery data using Jacob's technique

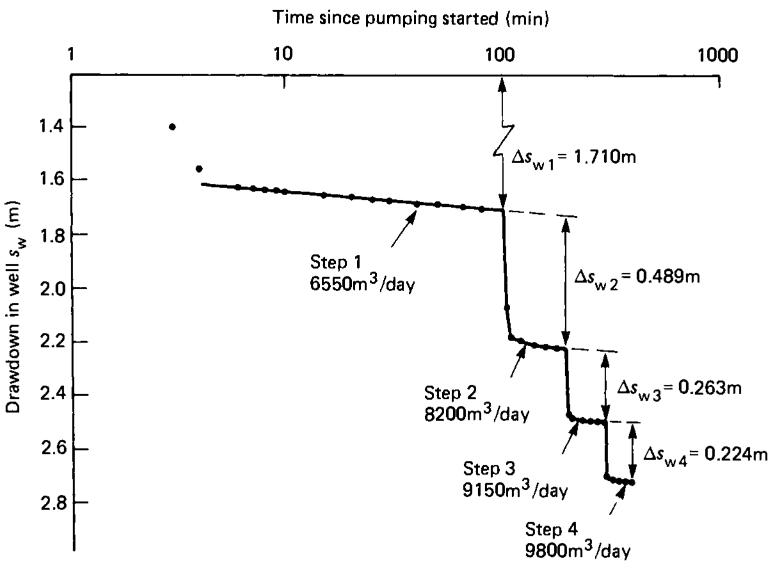

Figure 7.28 Step drawdown test analysis

extrapolated, as for the recovery technique described in Section 7.10.4. The extrapolated line is then used to determine the incremental drawdown, Δs_w, caused by a change in the pumping rate. The length of each step of the test is not important provided that the Δs_w values are calculated for the same elapsed time within each time step. For the data in *Figure 7.28* these have been calculated after 100 min of each step have elapsed and are as shown below (column 3).

1	2	3	4	5
Step	*Discharge rate* (Q m^3/day)	*Incremental drawdown* (Δs_w m)	*Cumulative drawdown* ($s_w = \Sigma \Delta s_w$ m)	s_w/Q (m/m^3/day)
1	6550	1.710	1.710	2.61×10^{-4}
2	8200	0.489	2.199	2.68×10^{-4}
3	9150	0.263	2.462	2.69×10^{-4}
4	9800	0.224	2.686	2.74×10^{-4}

Next, the sum of the incremental drawdowns is obtained for each step of the test (column 4). Finally, the specific drawdown, s_w/Q at each step is calculated (column 5). When the specific drawdown is plotted against the discharge rate, Q, a straight-line graph is generally obtained[9] with a slope C and an intercept B as shown in *Figure 7.29*.

The evaluation of the well loss, CQ^2, enables the corrected drawdown in the pumped well to be estimated (see Section 7.3). The actual drawdown in a pumped well is always more than the theoretical drawdown which is based upon the assumption that all the flow in the aquifer is laminar. As a result of turbulent flow losses near the well, the actual drawdown is greater than would be predicted from an extrapolation of the drawdown recorded at observation holes some distance from the well (see *Figures 7.7* and *7.8*). If there are no observation holes and it is necessary to analyse the data from the pumped well, then the drawdown must be corrected to compensate for the turbulent well loss. This correction can be accomplished by subtracting the well loss, CQ^2, from the total or observed drawdown at the well (see *Figure 7.30*). The drawdown resulting from laminar flow is, therefore, approximated by BQ. The Theis or Thiem equations can then be applied to the data.

The efficiency of a well may be calculated using Equation (7.2). In this example, with a discharge of 5000 m^3/day the efficiency is calculated as follows

$$\text{Well efficiency }(\%) = \frac{BQ}{BQ + CQ^2} \times 100 = \frac{2.385 \times 10^{-4} \times 5000 \times 100}{2.385 \times 10^{-4} \times 5000 + 3.5 \times 10^{-9} + 5000^2}$$

$$= 93 \text{ per cent}$$

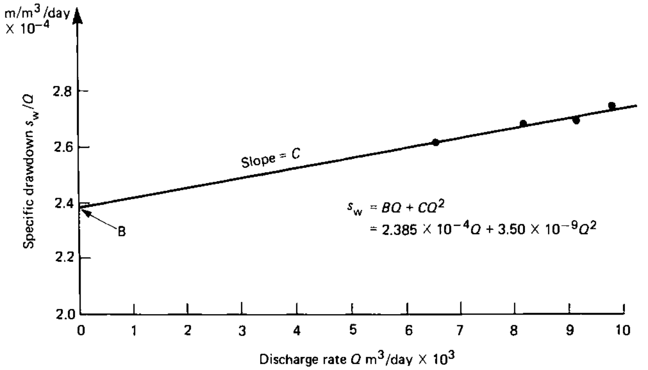

Figure 7.29 Variation in specific drawdown

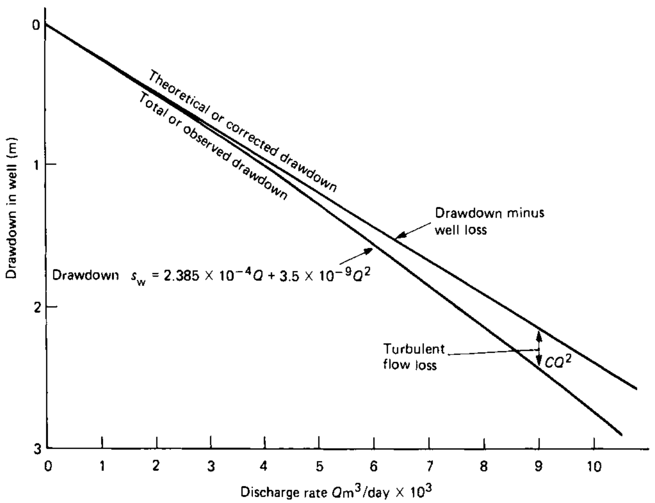

Figure 7.30 Correction for well loss

The value of well efficiency obtained from such a calculation is not particularly significant by itself. No well is 100 per cent efficient. It does, however, provide a useful standard against which the future performance can be compared. For instance, a progressive decline in well efficiency could indicate clogging of the screen and surrounding aquifer. The efficiency could also be used to assess the effectiveness of aquifer development (see Section 6.9).

The transmissivity of the aquifer can be calculated from the first step of the test, after the drawdown data have been corrected for well loss, using either the equilibrium or Theis equations as appropriate. Alternatively, an approximate equilibrium analysis may be conducted using Equation (7.9), which can be re-arranged as follows after subtracting the well loss CQ^2 from Equation (7.1)

$$T = 1.2Q/s_w = 1.2/B$$

where B is the coefficient obtained from the step drawdown analysis. For the above example, the coefficient of transmissivity is given by

$$T = \frac{1.2}{2.385 \times 10^{-4}} = 5031 \text{ m}^2\text{/day}$$

Using the Bierschenk and Wilson[12] technique it is not possible to calculate the coefficient of storage of the aquifer. Clark[8] presented more rigorous methods of analysis which enable this coefficient to be calculated.

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Chapter 8

Groundwater pollution

8.1 Introduction

Once a water well or well field has been successfully put into supply it must be recognized that there exists the strong possibility that groundwater pollution may occur at some time in the future. Certainly groundwater pollution is not an abstract concept. It is of common occurrence, as a survey of recent literature shows. For instance, the problems facing Dade County, Florida, include saltwater intrusion, high nitrate levels and volatile organic pollution[1]. In other parts of the USA and Canada, toxic chemicals have been a cause of concern[2,3,4,5]. In fact, most of man's activities have a direct, and usually adverse, effect upon water quality (see *Figures 8.1* and *8.2*).

Jackson[6] gave a comprehensive list of potential contaminants, the characteristics of the leachate or effluent and the typical rate of production of the effluent or waste. The many diverse sources of pollution and the worldwide scale of the problem are clearly illustrated by the case histories which Jackson presented. These included examples of groundwater contamination arising from arsenic pesticides (Hungary), insecticide disposal (Romania), nitrates (UK and Hungary), lead (Italy), metal plating wastes (USA), hydrocarbons (Czechoslovakia), sugar waste disposal (Cuba), bacterial contamination (Mexico and Italy), road salt (Canada) and oil/gas production (Hungary). The threat of groundwater pollution commonly posed by chemical, biological and radioactive agents was also discussed by Jackson[7].

Brown *et al.*[8] defined 'pollution' comprehensively as the 'addition of chemical, physical or biological substances or of heat which causes deterioration in the natural quality, generally through the action of man or animals or any other kind of activity'. More specifically Walker[9] defined pollution as 'an impairment of water quality by chemicals, heat, or bacteria to a degree that does not necessarily create an actual public health hazard, but that does adversely affect such waters for normal, domestic, farm, municipal or industrial use'. On the other hand, Walker used the term 'contamination' to denote 'impairment of water quality by chemical or bacterial pollution to a degree that creates an actual hazard to public health'. Although this is a useful distinction in many situations, it is not strictly applied below.

The natural quality of underground water often compares favourably with that from surface sources and the groundwater from deep aquifers in particular can be remarkably pure. However, this does not mean that the quality can be relied upon. In fact, groundwater can undergo cyclic changes in quality[10], natural variations in

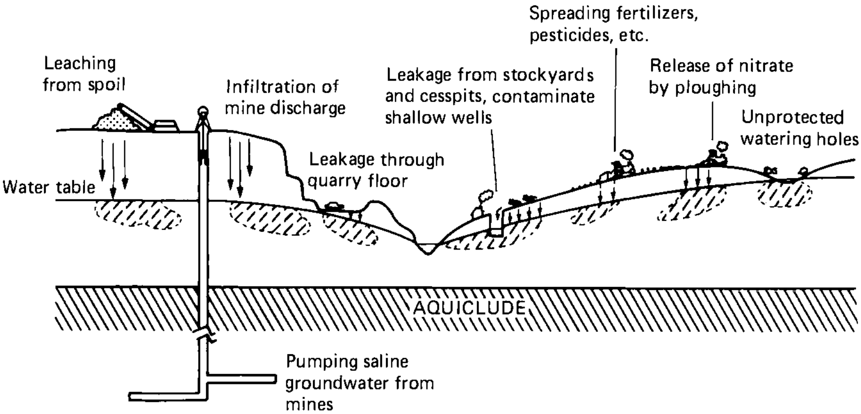


Figure 8.1 Sources of contamination from agriculture and mineral exploitation (after Jackson[7])

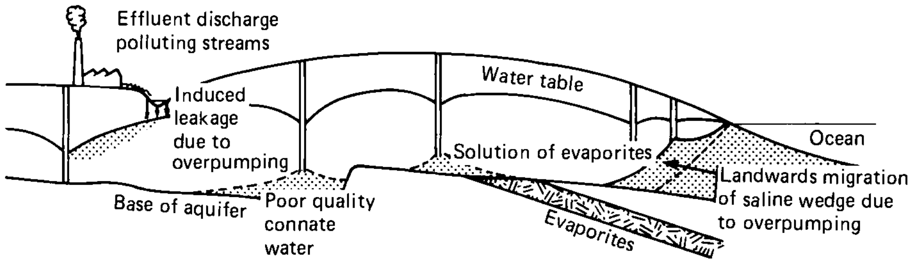


Figure 8.2 Sources of contamination by groundwater abstractions and effluent discharges (after Jackson[6])

groundwater quality occur with depth, rock type and so on as described in Chapter 5 and of course there is always the possibility of accidental spillage[4,11]. Thus, the purity of groundwater should not be taken for granted and untreated water should not be put directly into supply[12]. Some form of chlorination or disinfection, at least, is always advisable if the water is to be used for domestic purposes. This is a sensible precaution that reduces the risks involved should a supply become contaminated, which may happen from time to time.

The most likely source of contamination is from animals, man in particular. For example, in nineteenth century Britain it was common practice to obtain water from shallow wells and to dispose of sewage via earth closets and dunghills. This unhappy practice resulted in sewage contamination of water supplies and outbreaks of cholera in London in 1854 and 1866[13]. Although over 100 years later the use of earth closets and shallow wells has largely been eliminated in Britain, there is no reason for complacency. According to Pearce[14] there was an outbreak of typhoid in Croydon, England, in 1937 after an infected workman defecated into a large borehole while the chlorination equipment was shut down. More recently, in 1980, 1000 people in Wetherby, West Yorkshire, were affected by gastroenteritis as a result of the Bramham borehole becoming contaminated by a combination of a leaking sewer and a polluted surface stream which passed within 8 m of the well[15].

In 1964 some 3 per cent of Britain's homes were still without a piped water supply, while in 1979 less than 94 per cent of the population of the area covered by South West Water were connected to the authority's system[14]. Thus, in remote rural parts of Britain, it is still common for water to be drawn from wells and springs, not all of which have associated treatment plant to disinfect the water before it reaches the consumer. As a result, a significant proportion of the water provided by these sources has, at some time, suffered from sewage contamination.

In the less developed parts of the world the problems are much more acute. It was estimated[16] that in 1964 more than 900 million people were without a public water service of any kind, while in the developing countries as many as 500 million people per year were affected by water-borne or water-related diseases[17,18]. By 1980 the situation had deteriorated. The World Health Organisation then estimated that 1320 million people (57 per cent) of the developing world (excluding China) were without a clean water supply, while 1730 million (75 per cent) were without adequate sanitation. At least 30 000 people per day die in the Third World because they have inadequate water and sanitation facilities[19]. In fact, many Third World countries are now facing the same type of problems that were encountered in Europe at the start of the Industrial Revolution in the eighteenth century. The control of water pollution in developing countries is a necessity; some disastrous environmental results have already occurred through lack of attention to this problem[20]. These facts indicate that groundwater pollution is not just a hydrogeological problem, many political, social, economic and medical factors are also involved[21].

The greatest danger of groundwater pollution is from surface sources such as farm animals, man, sewers, polluted streams, refuse disposal sites and so on. Areas with thin soil cover or where the aquifer is exposed, such as the recharge area, are the most critical from the point of view of pollution potential. Any possible source of contamination in these areas should be carefully evaluated, both before and after well construction, and the viability of groundwater protection measures considered[22]. Changes in land use may pose new threats to water quality. Obvious precautions against pollution are to locate the wells as far from any potential source of contamination as possible and to fence off the tops of wells so that animals cannot defecate adjacent to the well. Good well design and construction is also important. These and other topics are considered in more detail below. The routine analysis of water samples and the continuous monitoring of groundwater quality, which is an essential part of aquifer management, are considered in Chapter 9.

However, it should be appreciated that the slow rate of travel of pollutants in underground strata means that a case of pollution may go undetected for a number of years. During this period a large part of the aquifer may become polluted and cease to have any potential as a source of water[23].

8.2 Rock type, pollution potential and attenuation of a pollutant

The attenuation of a pollutant as it enters and moves through the ground occurs as a result of four major processes[6,8,24].

1. *Biological processes* Soil has an enormous purifying power as a result of the communities of bacteria and fungi which live in the soil. These organisms are capable of attacking pathogenic bacteria and can also react with certain other harmful substances.
2. *Physical processes* As water passes through a relatively fine-grained porous media

such as soil and rock, suspended impurities are removed by filtration.

3. *Chemical processes* Some substances react with minerals in the soil/rock, and some are oxidized and precipitated from solution. Adsorption may also occur in argillaceous or organic material.

4. *Dilution and dispersion* The traditional method of disposing of effluent has generally been to dilute it with a large quantity of relatively pure water so that the concentration of the pollutant eventually becomes negligible at some distance from the source, partly as a result of the above processes. The wisdom of this practice has been questioned in recent years, but it still retains support[25,26].

From a consideration of these processes it is apparent that the self-cleansing capacity of a soil–aquifer system will depend upon three principal factors; the physical and chemical form of the pollutant, the nature of the soil/rock comprising the aquifer and the way in which the pollutant enters the ground.

The form of the pollutant is clearly an important factor with regard to its susceptibility to the various purifying processes. For instance, pollutants which are soluble, such as fertilisers and some industrial wastes, are not affected by filtration. Metal solutions may not be susceptible to biological action. Solids, on the other hand, are amenable to filtration provided that the transmission media is not coarse grained, fractured or cavernous. Karst or cavernous limestone areas pose particular problems in this respect. Non-soluble liquids such as hydrocarbons are generally transmitted through a porous media, although some fraction may be retained in the host material. Usually, however, the most dangerous forms of groundwater pollution are those which are miscible with the water of the aquifer[24].

In general, the concentration of a pollutant decreases as the distance it has travelled through the ground increases. Thus, the greatest pollution potential exists for wells tapping shallow aquifers that intersect or lie near ground level. Aquifers that are exposed or overlain by a relatively thin formation in the recharge area are also at risk, particularly when the overlying material consists of weathered or creviced limestone or dolostone, sandstone, faulted material, sand, gravel or any other unconsolidated material. Conversely, deeply buried aquifers overlain by relatively impermeable shale or clay beds can generally be considered to have a low pollution potential and are less prone to severe contamination[27]. One approach to groundwater quality management is to mark areas with a high pollution potential on a map and to pay particular attention to activities within these vulnerable areas.

When assessing pollution potential, an additional consideration is the way in which the pollutant enters the ground. If it is evenly distributed over a large area its probable effect will be less than the same amount of pollutant concentrated at one point. Concentrated sources are most undesirable because the self-cleansing ability of the soil in that area is likely to be exceeded. As a result the 'raw' pollutant may be able to enter an aquifer and travel some considerable distance from the source before being reduced to a negligible concentration.

A much greater hazard exists when the pollutant is introduced into an aquifer beneath the soil. In this case the powerful purifying processes that take place within the soil are bypassed and attenuation of the pollutant is reduced. This is most critical when the pollutant is added directly to the zone of saturation, because the horizontal component of permeability is usually much greater than the vertical component. For instance, intergranular seepage in the unsaturated zone may have a typical velocity of less than 1 m/year, whereas lateral flow beneath the water table may be as much as 2 m/day under favourable conditions[28]. Consequently, a pollutant can travel a much greater distance before significant attenuation occurs. This type of hazard often arises from poorly maintained domestic septic tanks and soakaways, from the discharge of quarry wastes, farm effluents and sewage to streams and from the disposal of refuse and commercial wastes. Atkinson[29] quoted 11 instances of groundwater contamination within the space of a few years in the Mendip Hills (England) as a result of such activities. Similarly, leaking pipelines and underground storage tanks have resulted in the abandonment of several million domestic and other water wells in the USA[30].

Although biological pollution in the form of micro-organisms, viruses and pathogens is quite common, it is often a subject of which hydrogeologists have little or no understanding. However, when considering biological contamination, the first important point which must be appreciated is that not all bacteria are harmful. On the contrary, many are beneficial and perform valuable functions, such as attacking and biodegrading pollutants as they migrate through the soil.

The bacteria which normally inhabit the soil or aquatic environments thrive at temperatures of around 20°C. Bacteria which are of animal origin prefer temperatures of around 37°C and so generally die quite quickly outside the host body. Consequently, it is sometimes erroneously assumed that pathogenic bacteria cannot survive long underground. However, Brown *et al.*[8] pointed out that under these conditions bacteria may have a life span of up to 4 years. It is generally assumed that bacteria (which are the size of small silt or large clay particles, perhaps 1×10^{-3} mm) move at a maximum rate of about two-thirds of the water velocity. Since most groundwaters only move a few metres per year, the distances travelled are usually quite small and, in general, it is unusual for bacteria to spread more than 33 m from the source of the pollution. Of course, in openwork gravel, cavernous limestone or fissured rock, the bacteria may spread over distances of many kilometres[8,31].

Many outbreaks of water-borne disease have been attributable to viruses originating from human faeces[32]. Viruses are parasites which require a host organisms before they can reproduce, but they are capable of retaining all their characteristics for 50 days or more in other environments[8]. Because of their relatively small size, perhaps 20×10^{-6} mm, viruses are not greatly affected by filtration but are prone to adsorption, particularly when the pH is around 7 (see Jackson[6]). However, Brown *et al.*[8] suggested that viruses are capable of spreading over distances which exceed 250 m, although 20 to 30 m may be a more typical figure[31].

For non-karstic terrains, the minimum recommended safe distance between a domestic well and a source of pollution is as shown in *Table 8.1*. In karst or weathered limestone areas, however, pollutants may be able to travel quickly over large distances.

TABLE 8.1. Recommended safe distance between domestic water wells and sources of pollution (after Romero[31])

Source of pollution	*Distance* (m)
Septic tank	15
Cess pit	45
Sewage farms	30
Infiltration ditches	30
Percolation zones	30
Pipes with watertight joints	3
Other pipes	15
Dry wells	15

Hagerty and Pavoni[33] observed the spread of contaminated groundwater over a distance of 30 km through limestone in approximately 3 months. They also noted that the degradation, dilution and dispersion of harmful constituents was less effective than in surface waters.

The coliform group of bacteria, and *Escherichia coli* (*E. coli*) in particular, are one of the most frequently used indicators of bacterial contamination but are themselves harmless or non-pathogenic. The reason *E. coli* have gained this distinction is because they are easy to detect in the laboratory while being present in large numbers in the intestines and faeces of animals. Since the whole coliform group is foreign to water, a positive *E. coli* test indicates the possibility of bacteriological contamination. If *E. coli* are present then it is possible that the less numerous pathogenic or harmful bacteria, which are much more difficult to detect, are also present. On the other hand, if there are no *E. coli* in a sample of water, then the chances of faecal contamination and of pathogens being present are generally regarded as negligible. However, Morris and Waite[15] pointed out that some viruses are more resistant to disinfection than *E. coli*, so under some circumstances this generalization may not be valid.

8.3 Faulty well design and construction

Perhaps the greatest risk of groundwater contamination lies in the transfer of pollutants from the ground surface to the aquifer. Therefore it is not surprising that one of the most common causes of groundwater contamination is poor borehole design, construction and maintenance. A faulty well can ruin a high-quality groundwater resource.

During the construction of a well there is an open hole which affords a direct route from the surface to the aquifer. Apart from the possibility of surface run-off entering the hole during periods of rainfall, various insanitary materials such as non-potable quality water, drilling fluids, chemicals, casings, screens and so on, are deliberately placed in the hole. This provides an ideal opportunity for chemical and bacteriological pollution to occur[34], but lasting damage can be avoided if the well is completed, disinfected and pumped within a short space of time. Under these circumstances most of the potentially harmful substances are discharged from the well. However, if there is a lengthy delay in completing the well, the possibility of contamination increases. Similarly, if the well is constructed in a cavernous or highly permeable formation, the chances of recovering all the harmful materials introduced during construction are decreased, while the possibility of pollution at nearby wells is increased.

As far as the well structure itself is concerned, Campbell and Lehr[27] described the following means by which pollution can occur

1. Via an opening in the surface cap or seal, through seams or welds in the casing, or between the casing and the base of a surface mounted pump (*Figure 8.3(a)*).
2. As a result of reverse flow through the discharge system (*Figure 8.3(b)*).
3. Through the disturbed zone immediately surrounding the casing (*Figure 8.3(c)*).
4. Via an improperly constructed and sealed gravel pack (*Figure 8.3(d)*).
5. As a result of subsidence due to the inability of the basal formation to support the weight of the well structure, sand pumping, or seepage resulting from reduced effectiveness of the surface cap or seal (*Figure 8.3(e)*).
5. As a result of the grout or cementing material forming the seals failing through cracking, shrinking, etc.

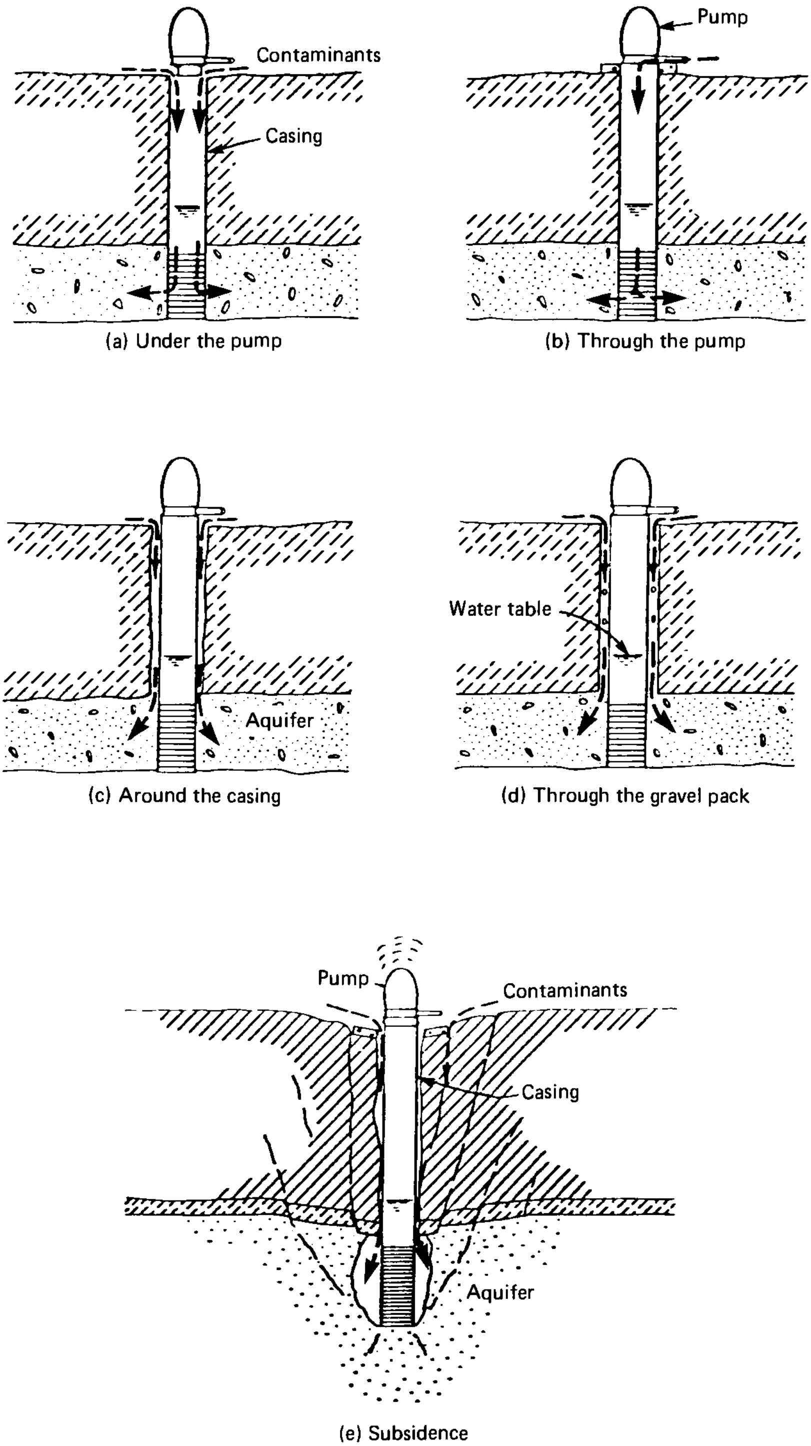
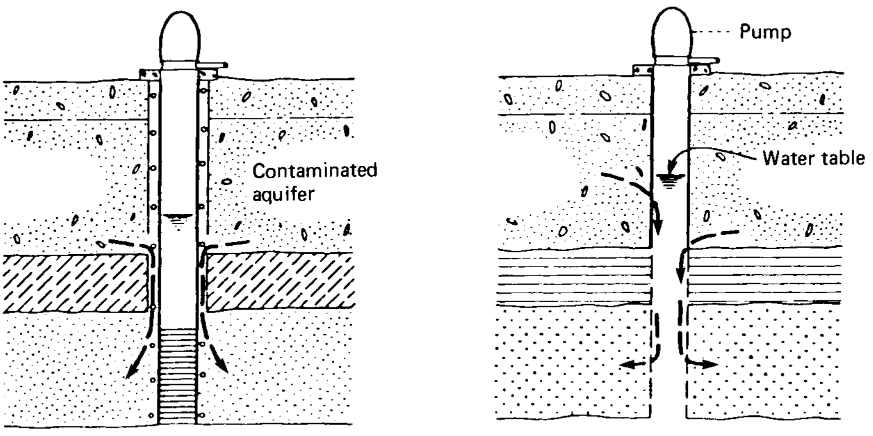


Figure 8.3 Means by which contaminants can enter an aquifer via a well (after Campbell and Lehr[27])

6. Through breaks or leaks in the discharge pipes leading to scour and failure of the cement-grout seals.

When a borehole penetrates only one aquifer the principal concern is the transfer of pollution from the ground surface. However, with multiple aquifers there is the additional possibility of inter-aquifer flow. In such cases, each aquifer (or potential source of pollution) must be isolated using cement-grout seals in the intervening strata. Failure to do this, particularly when the well incorporates a gravel pack, provides a conduit through which water can be transferred from the ground surface and from one aquifer to another. This could be potentially disastrous where, say, a shallow contaminated aquifer overlies a deeper, unpolluted aquifer (*Figure 8.4(a)*). In such a situation pollution could also occur as a result of leakage under the bottom of the surface casing, or through the casing itself if it has seams, welded joints or is severely corroded (*Figure 8.4(b)*).



(a) Through the gravel pack (b) Through the casing

Figure 8.4 Transfer of contaminants from one aquifer to another via a well (after Campbell and Lehr[27])

Even after a well has ceased to be productive and has been abandoned, it still provides a means of entry to the aquifer for pollutants and may even be a greater hazard than it was during its operational life. Abandoned wells make suitable receptacles for all kinds of wastes and also refuges for vermin. Additionally, as the well becomes progressively older, the strength and effectiveness of the casing and sanitary seals will deteriorate, thus increasing the possibility of the structural failure of the well, with pollution occurring subsequently as a result of one of the mechanisms described above. If the well has been abandoned, any pollution that does happen might go unnoticed until detected at a nearby well, by which time it may be too late.

8.4 Leachate from refuse disposal sites

A major threat to the quality of groundwater arises from the disposal of domestic and commercial wastes in landfill sites. According to Barber[35], approximately 18.6 million tonnes of domestic solid waste are produced each year in the UK, with 76 per cent of the untreated rubbish being placed in landfills. Typically this rubbish comprises 60 per cent biodegradable solids (such as paper, metal, vegetable and putrescible matter), 16 per cent inert solids (such as glass and plastic) and 24 per cent other varied and unclassified material. Landfills are likely to remain the major method of disposal for domestic refuse in the near future, and so represent a continuing threat to water quality[36].

Leachate is formed when liquids such as rainfall infiltrate the landfill and dissolve the soluble fraction of the waste and the soluble products formed as a result of the chemical and biochemical processes occurring within the decaying wastes. Generally, the conditions within a landfill are anaerobic, so leachates often contain high concentrations of dissolved organic substances resulting from the decomposition of organic material such as vegetable matter and paper. Recently emplaced wastes may have a Chemical Oxygen Demand (COD) of around 11 600 mg/l and a Biochemical Oxygen Demand (BOD) in the region of 7250 mg/l. The concentration of chemically reduced inorganic substances like ammonia, iron and manganese varies according to the hydrology of the site and the chemical and physical conditions within the site. However, Barber[35] listed the concentration of ammoniacal nitrogen in a recently emplaced waste as 340 mg/l, chloride as 2100 mg/l, sulphate as 460 mg/l, sodium as 2500 mg/l, mangnesium as 390 mg/l, iron as 160 mg/l and calcium as 1150 mg/l. Brown *et al.*[8] calculated that a tip containing 1000 m^3 of rubbish can yield 1.25 tonnes of potassium and sodium, 0.8 tonnes of calcium and magnesium, 0.7 tonnes of chloride, 0.19 tonnes of sulphate and 3.2 tonnes of bicarbonate. Additionally, Barber[35] has estimated that a small landfill site with an area of 1 hectare located in southern England could produce up to 8 m^3 of leachate per day, mainly between November and April, from a rainfall of 900 mm/year assuming that evaporation is close to the average for the region and run-off is minimal. A site with an area ten times as large would produce a volume of effluent with approximately the same BOD load per year as that received by a small rural sewage treatment works. Thus, it can be appreciated that the disposal of domestic wastes in landfill sites can produce large volumes of effluent with a high pollution potential[37]. For this reason the location and management of these sites must be carefully controlled. In Britain such considerations are governed by Part 1 of the Control of Pollution Act, 1974[38] which has the following major objectives.

1. To ensure that adequate provisions are available for the disposal of controlled waste.
2. To carry out a survey of waste arising and to produce a plan to cope with future waste disposal.
3. To license all disposal operations including landfill sites.
4. To operate a special authorization procedure for hazardous or difficult wastes.

Under the Act, regional water authorities have to be consulted as part of the licensing procedure in order to safeguard water resources in the vicinity of the disposal site. Barber[35] identified three classes of landfill site based upon hydrogeological criteria (see *Table 8.2*). When assessing the suitability of a site, two of the principal considerations are the ease with which the pollutant can be transmitted through the substrata and the distance it is likely to spread from the site. Consequently, the primary and secondary permeability of the formations underlying the landfill area are a major factor. It is unlikely that a landfill licence would be granted to a site of the type listed first in *Table 8.2*. There would also be grounds for objection to a landfill site falling within the second category of the table if the site were located within the area of diversion to a water supply well (see *Figure 4.15*). Generally, the third classification, in which the leachate is contained within the landfill area, is to be preferred. Since all natural materials possess

TABLE 8.2. Classification of landfill sites based upon their hydrogeology (after Barber[35])

Designation	*Description*	*Hydrogeology*
Fissured site, or site with rapid subsurface liquid flow	Material with well developed secondary permeability features	Rapid movement of leachate via fissures, joints, or through coarse sediments. Possibility of little dispersion in the groundwater, or attenuation of pollutants
Natural dilution, dispersion and attenuation of leachate	Permeable materials with little or no significant secondary permeability	Slow movement of leachate into the ground through an unsaturated zone. Dispersion of leachate in the groundwater, attenuation of pollutants (sorption, biodegradation, etc.) probable
Containment of leachate	Impermeable deposits such as clays or shales, or sites lined with impermeable materials or membranes	Little vertical movement of leachate. Saturated conditions exist within the base of the landfill

some degree of permeability, total containment can only be achieved if an artificial impermeable lining is provided over the bottom of the site. However, there is no guarantee that clay, soil cement, asphalt or plastic linings will remain impermeable; they may be ruptured by settlement, for example. Thus, the migration of materials from the landfill site into the substrata will occur eventually, only the length of time before this happens being in doubt. In some instances this will be sufficiently long for the problem of pollution to be greatly diminished.

In order to reduce the amount of leachate emanating from a landfill, it is advisable to construct cut-off drains around the sites to prevent the flow of surface water into the landfill area. The leachate that originates from direct precipitation, or other sources, should be collected in a sump and either pumped to a sewer, transported away by tanker, or treated on site[39,40,41]. Under no circumstances should untreated leachate be allowed to enter a surface watercourse, or be allowed to percolate to the water table if this is likely to pose a significant threat to quality. An account of controlled tipping was given by Bevan[42] and Gass[43] also described certain design criteria and operational methods that can further reduce the danger of contamination.

The additional hazards involved when a pollutant is added directly to the zone of saturation were described in Section 8.2. This applies especially to wet tipping of refuse in landfill sites and the practice should be avoided. Gray *et al.*[44] recommended that at dry sites, tipping should take place on granular material which has a thickness of 15 m or more, while any water wells should be located at least 0.8 km away. The Severn–Trent Water Authority reserves the highest degree of potential protection to the area within a 1 km radius of a water supply borehole. This distance is modified to suite local site conditions but is generally assumed to represent a retention period in the Triassic sandstones of between 50 and 100 days in saturated flow to a typical pumping source if fissuring is well developed. Within this protected area, referred to as Zone 1, it is considered likely that any pollution will affect the abstracted water rapidly and significantly. A lesser degree of protection is awarded to three other defined zones[23].

This is in contrast with two previous precedents which involved a 3 km radius of protection[45] or a 50-day residence period (a common European practice).

More recently it has been argued that pollution plumes around landfill sites are often quite limited in extent, so there is no need to be overcautious[25]. This view has been supported by Tester and Harker[26], but disputed by Selby and Skinner[23]. The latter argue that the apparent absence of groundwater pollution in the vicinity of landfills is due to the fact that in the past, landfill has not been allowed in close proximity to water supply boreholes. In any other situation there are insufficient facilities to enable groundwater pollution to be detected and no need to carry out routine monitoring. Selby and Skinner also recorded an instance of groundwater pollution at a borehole in the Triassic Sandstone in Nottinghamshire which was located 1.5 km from a waste disposal facility that is underlain by 5 m of unsaturated aquifer. It is claimed, therefore, that there is evidence that persistent contaminants can travel substantial distances through aquifers.

If the landfill sites that are used for the disposal of domestic waste are properly managed they should not present a serious threat to groundwater quality. However, when sites have to cope with liquid or industrial wastes, or wastes of indeterminate composition, the danger is increased. Gray *et al.*[44] considered that the presence of a toxic or oily liquid waste constitutes a serious risk. The range of toxic wastes varies from industrial effluents to chemical and biological wastes from farms. Unfortunately, the effects of depositing several types of waste together are often unknown. Consequently, the best policy where toxic wastes are involved is probably containment, along similar lines to those outlined for domestic wastes. An additional requirement recommended by Gray *et al.* is that a site handling toxic wastes should be underlain by at least 15 m of impermeable strata. Any well abstracting groundwater for domestic use and confined by such impermeable strata should be more than 2 km away.

Almost any waste disposal site represents a potential source of groundwater pollution, while the risk increases when toxic chemicals are involved. There is, however, an added danger in the form of the illegal dumping of sometimes highly toxic wastes. Although this activity is difficult to stop, it may be detected by a suitable groundwater monitoring scheme (see Section 9.2), and certain precautions should always be carried out. These include

1. Locating water wells up the hydraulic gradient, if possible, from any waste disposal site, or at least ensuring that the well is not directly down gradient of the source of pollution. A flow net may provide a suitable means of study initially, followed by tracer surveys if a serious risk is involved.
2. Locating the well so that the cone of depression or area of diversion to the well does not reach the source of pollution. In fact, the greater the distance between the well and the source of pollution, the better.
3. Monitoring groundwater quality at points around the periphery of the disposal site and between the site and any water supply wells (see Section 9.2). Over 20 boreholes may be required to satisfactorily identify the hydrogeology and plume migration at a particular site[38,46]. Typical indicators of deteriorating groundwater quality include increases in water hardness and in the concentration of sulphates and chlorides. Concentrations of free CO_2 greater than 20 mg/l create corrosive conditions, nitrate in excess of 50 mg/l is considered dangerous, while chloride levels of 200 mg/l may cause taste problems in potable water. By the judicious use of routine water quality surveys it may be possible to detect groundwater deterioration at an early stage. This should ensure that any water wells in the affected area are closed down before the supply becomes contaminated.

4. Maintaining a careful check of land use, or changes in land use, in critical areas such as the recharge zone of the aquifer and in the vicinity of shallow wells. This may detect illegal dumping or other harmful practices.
5. Obtaining an estimate of the probable rainfall at the landfill site and thus the volume of leachate[47]. The dilution with the water within the aquifer can then be calculated. Flow lines can be used to estimate the proportion of the leachate that will arrive at the supply wells and thus the quality of the abstracted water. While it is difficult to assess the effects of attenuation, dilution and dispersion on the pollutant, such calculations are better than nothing[25,48].
6. Having an emergency plan that can be implemented quickly if contaminated water appears in the monitoring or supply wells. If the plan exists then the worst (and best!) thing that can happen is that it will never be used. If such a plan has not been developed and there is an accident, widespread illness and perhaps even fatalities could result. There may then be charges of negligence to answer[49].

8.5 Induced infiltration

Induced infiltration occurs where a stream is hydraulically connected to an aquifer and lies within the area of influence of a well. When the well is pumped, water is initially released from the part of the aquifer immediately adjacent to the well. As the cone of depression spreads, water is withdrawn from storage over a progressively increasing area of influence while water levels are lowered beneath the surface source adjacent to the well. Eventually the aquifer is recharged by the influent seepage of surface water, so that some proportion of the pumpage from the well is now obtained from the surface source. As a result, the cone of depression will be distorted with the hydraulic gradient between the source and the well being steep in comparison to that on the side away from the source. In effect the stream constitutes a recharge boundary and the situation is similar to that shown in *Figure 7.20*. Assuming relatively uniform conditions, the flow in the distorted cone will generally conform to Darcy's law, so that the flow towards the well will be greatest on the side nearest the source where the gradients are steepest. As pumping continues the proportion of water entering the cone of depression that is derived from the stream will increase progressively[50,51].

If the influent seepage of surface water is less than the amount required to balance the discharge from the well, the cone of depression will spread up and downstream until the drawdown and the area of the stream bed intercepted are sufficient to achieve the required rate of infiltration. If the stream bed has a high permeability, the cone of depression may extend over only part of the width of the stream; if it has a low permeability the cone may expand across and beyond the stream. If pumping is continued over a prolonged period, a new condition of equilibrium will be established with essentially steady flows. Most of the abstracted water will then be derived from the surface source. Since the stream bed infiltration occurs as a result of groundwater abstraction and would not occur otherwise, this has been termed induced infiltration.

Induced infiltration is significant from the point of view of groundwater pollution in two respects. First, the new condition of equilibrium that has been established may involve the reversal of the non-pumping hydraulic gradients, particularly when groundwater levels have been significantly lowered by the abstraction of groundwater (see *Figure 8.5*). This may result in pollutants travelling in the opposite direction to that normally expected. Secondly, surface water resources are often less pure than the underlying groundwater so induced infiltration introduces the danger of con-

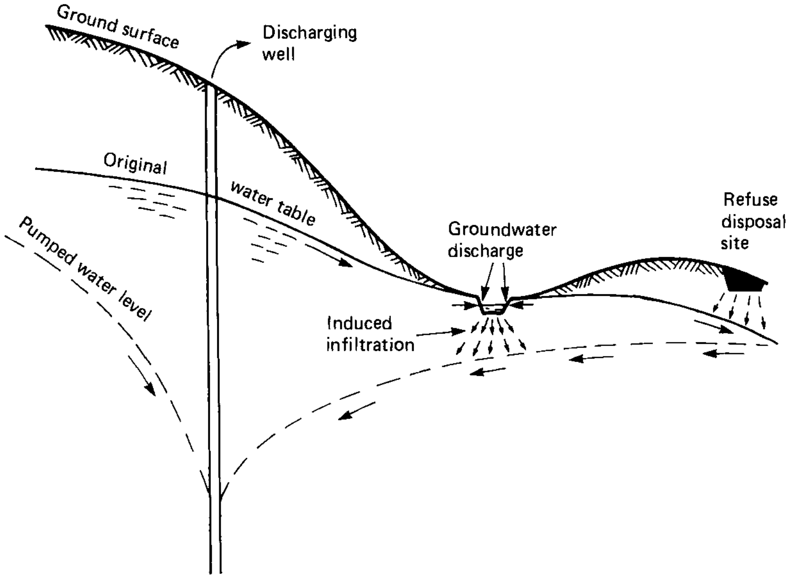


Figure 8.5 Example of induced infiltration as a result of pumping. Over much of the area the original hydraulic gradient has been reversed, so pollutants could travel in the opposite direction to that expected. Additionally, the aquifer has become influent instead of effluent, as it was originally

tamination. However, induced infiltration does not automatically equate with pollution. On the contrary, induced infiltration is a traditional method of augmenting groundwater supplies in Europe, America and elsewhere[52,53,54]. The experience gained from the operation of more than 200 Ranney-Collector Wells (see *Figure 6.30*) throughout the world has shown that, under normal conditions, the water produced by induced infiltration is generally free from turbidity, pathogenic bacteria, organic matter, tastes and odours. Consequently, the evaluation of situations involving induced infiltration is not as straightforward as might be expected. There are, perhaps, three pertinent questions that should be considered

1. Will induced infiltration occur?
2. Will induced infiltration cause pollution of the groundwater resource?
3. What proportion of the well discharge will originate as induced infiltration?

Questions 2 and 3 are inter-related, but it is most convenient to proceed by considering each of the questions in turn.

Very few materials in nature are completely impermeable. Even relatively impermeable formations such as clay vary significantly in character and frequently contain pockets of permeable material or fissures that allow the passage of water. When the clay is thin, surface water recharge of an aquifer is likely to occur[55,56]. If the clay is quite thick, surface–groundwater interconnection still cannot be ruled out completely. Consequently, the safest policy is to assume that surface water infiltration or induced infiltration can occur until it is proved otherwise. This applies to other types of surface formation as well, not just clay and is most important when the surface streams are polluted.

The answer to the second question depends upon the quality of the surface water source, the nature of the aquifer, the quantity of infiltration involved and the intended use of the abstracted groundwater. As Hamill[57] pointed out, induced infiltration can either have potentially disastrous consequences or provide a valuable addition to the overall water resources of an area if a conjunctive use philosophy is adopted (see Section 9.4). Which of these two alternatives applies in a given location rather depends upon whether or not the possibility of induced infiltration was anticipated when the well field was designed.

As far as estimating the proportion of well discharge that originates from a surface source is concerned, there are basically two approaches that can be adopted. The first uses field data to form a correlation between groundwater level and river leakage, determined from the analysis of gauging station records[57]. In most situations this approach will not be feasible, so recourse must be made to one of the many theoretical techniques that are available. One of the first useful methods was outlined by Theis[50], who considered the stream as an idealized straight line of infinite extent. By using an image well technique and assuming that the groundwater level under the stream does not change, Theis was able to show that

1. Over half the well discharge derived directly from the stream (or derived indirectly by preventing or diverting groundwater flow to the stream) always originates between points at a distance, x, upstream and downstream of the well, where x is the distance between the well and the stream. The proportion originating from beyond these two points diminishes rapidly with distance.
2. If there is no significant increase in recharge or decrease in evapotranspiration (which would significantly reduce the effect of the well on the stream) the proportion of water taken from streamflow varies widely and depends mainly upon the coefficient of transmissivity of the aquifer and the distance of the well from the stream.

Theis also presented a graph which enabled the proportion of well discharge derived from the surface source to be calculated, based upon values of the aquifer characteristics and the duration of pumping[50,51]. Such relationships are interesting because there is a tendency to underestimate both the area of influence of a well and the amount of water likely to originate from the surface source. This point can be illustrated using the examples given by Theis, which form the basis of *Table 8.3*.

The technique introduced by Theis[50] has been extended and modified by many investigators who are far too numerous to mention. However, representative contributions have been made by Kazman[58,59], Glover and Balmer[60] and

TABLE 8.3. Proportion of well discharge derived from a surface source (based on data given by Theis[50])

Aquifer transmissivity	*Distance from well to stream* (km)	*Duration of pumping* (year)	*Proportion from stream* (%)
$620\ m^2/d$	1.6	1	30
(i.e. near the lower limit		20	90
of the coefficient for a	8	5	2
productive non-artesian aquifer*)		20	20
$5000\ m^2/day$	1.6	1	70
(i.e. near the upper		3	90
limit of the coefficient)	8	1	5
		20	75

* The table is applicable to an unconfined aquifer with a specific yield of 20 per cent

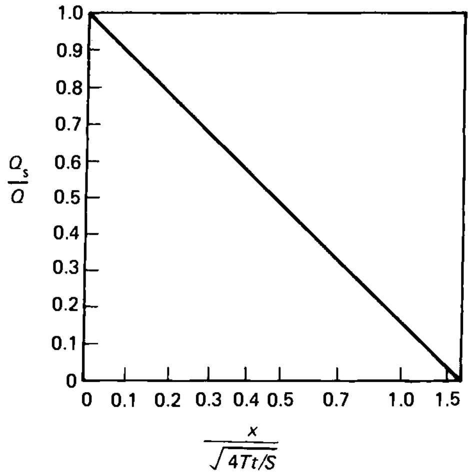


Figure 8.6 Graph for determining the proportion of well discharge derived from a near stream (after Glover and Balmer[60]). x is perpendicular distance from stream to well, T is coefficient of transmissivity, S is coefficient of storage, t is duration of pumping and Q_s/Q is proportion derived from stream

Hantush[61,62,63]. A graph for determining the proportion of well discharge derived from a nearby stream was presented by Glover and Balmer[60] and this is shown in *Figure 8.6.* The graph is valid for any consistent system of units.

Once the quantity and quality of the water derived from the surface source has been established, it is possible to assess whether or not this constitutes a significant threat to groundwater quality.

8.6 Saline intrusion

Although saline water, originating typically as connate water or from evaporitic deposits, may be encountered (see *Figure 8.2*), the problem of saline intrusion is specific to coastal aquifers. A useful introduction to the subject is provided by Cooper *et al.*[64], Jackson[6] and Todd[65].

Near a coast an interface exists between the overlying fresh groundwater and the underlying salt groundwater (*Figure 8.7*). Excessive lowering of the water table along the coast leads to saline intrusion, the salt water entering the aquifer via submarine outcrops thereby displacing fresh water. However, the fresh water still overlies the saline water and continues to flow from the aquifer to the sea[66]. In the past, the two groundwater bodies have usually been regarded as immiscible so that a sharp interface exists between them. In fact there is a transition zone which may vary from 0.5 m or so to over 100 m in width, although the latter figure may be atypically high.

The shape of the interface is governed by the hydrodynamic relationship between the flowing fresh and saline groundwater. However, if it is assumed that hydrostatic equilibrium exists between the immiscible fresh and salt water then the depth of the interface can be approximated by the Ghyben–Herzberg formula[67,68]. The depth below sea level to a point on the interface is given by

$$Z = \frac{\rho_w}{\rho_{sw} - \rho_w} \times h_w \qquad (8.1)$$

where ρ_{sw} is the density of sea water, ρ_w is the density of fresh water and h_w is the head of fresh water above sea level at the point on the interface (*Figure 8.7(a)*). If ρ_{sw} is taken as

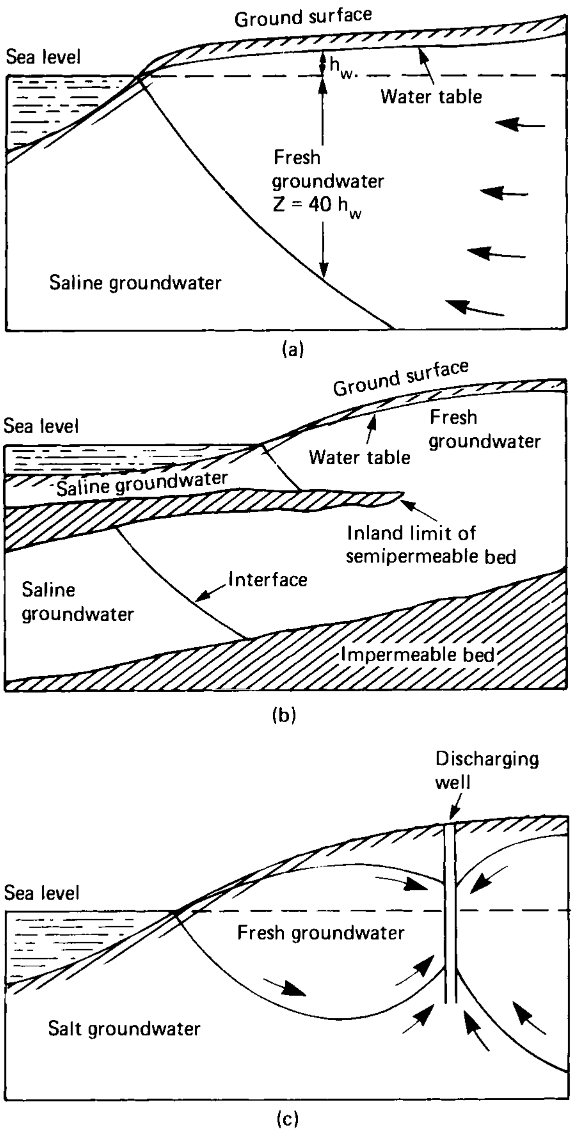


Figure 8.7 Diagrams illustrating, in a simplified manner, the Ghyben-Herzberg hydrostatic relationship in (a) a homogeneous coastal aquifer; (b) a layered coastal aquifer and (c) a pumped coastal aquifer

1025 kg/m^3 and ρ_w as 1000 kg/m^3 then

$$Z = 40h_w \tag{8.2}$$

Thus, at any point in an unconfined aquifer there is approximately 40 times as much fresh water below mean sea level as there is above it. This relationship can also be applied to confined aquifers if the height of the water table is replaced by the elevation of the piezometric surface above mean sea level. If the aquifer overlies an impermeable stratum, this formation will intercept the interface and prevent any further saline intrusion (*Figure 8.7(b)*).

The Ghyben–Herzberg relationship applies quite accurately to two-dimensional flow at right angles to the shore line. When flow is three-dimensional, such as when a well is discharging near the coast, most formulae for the position of the interface and the radius of influence of the well are inaccurate. However, the problem is amenable to mathematical modelling and quite a lot of work has been conducted on this subject in recent years[69,70,71,72].

The problem of saline intrusion often starts with the abstraction of groundwater from a coastal aquifer, which leads to the disruption of the Ghyben–Herzberg equilibrium condition. Generally saline water is drawn up towards the well and this is sometimes termed 'upconing' (*Figure* 8.7(*c*)). This is a dangerous condition that can occur even if the aquifer is not overpumped and a significant proportion of the fresh water flow still reaches the sea. A well may be ruined by an increase in salt content even before the actual 'cone' reaches the bottom of the well. This is due to 'leaching' of the interface by freshwater[6]. Again this situation is best studied using modelling techniques[73], although Linsley *et al.*[74] offered the rule of thumb that a drawdown at the well of 1 m will result in a salt water rise of approximately 40 m. This is a particular problem on small oceanic islands where the fresh water lens is shallow and everywhere floats on salt water. In such cases shallow wells or horizontal infiltration galleries may be adopted[65]. Of course, saline intrusion is not restricted to oceanic islands; it is a worldwide problem that exists in Britain, America, Israel, the Netherlands, Germany, Japan and elsewhere.

The encroachment of salt water may extend for hundreds of metres inland. The first sign of saline intrusion is likely to be a progressively upward trend in the chloride concentration of the water obtained from the affected wells. Typically chloride levels may increase from a normal value of around 25 mg/l to something approaching 19 000 mg/l, which is the concentration in sea water. The recommended limit for chloride concentration in drinking water in Europe is 200 mg/l, after which the water will have a salty taste. Encroaching sea water, however, may be difficult to recognize as a result of chemical changes, so Revelle[75] recommended the use of the chloride–bicarbonate ratio as an indicator. An additional complication is that there are likely to be frequent fluctuations in chloride content as a result of tides, varying rates of fresh water flow through the aquifer, meteorological phenomena and so on. This also means that the saline–fresh water interface is not static but mobile, moving seaward or landward according to the prevailing conditions.

In many respects the salt groundwater is analogous to the saline wedge that intrudes from the sea into a river estuary. Here a tongue of salt water, located on the bed of the estuary because of its greater density, moves upstream on the flood tide and retreats on the ebb tide. Fresh water in the river discharges continuously over the top of the saline wedge. The interface between the fresh and saline bodies of water becomes diffused so that a transition zone exists between the two. This is very similar to the hydrodynamic balance between fresh and saline groundwater.

Saline intrusion can be considered as a particular type of pollution and, as with other forms, once it happens it is very difficult to control and reverse. Overpumping is not the only cause of salt water encroachment, continuous pumping or the inappropriate location and design of wells may also be contributory factors. However, it should be remembered that the saline–fresh water interface is the result of a hydrodynamic balance, so if the natural flow of freshwater to the sea is interrupted or significantly reduced by abstraction, then saline intrusion is almost certain to occur. Once salt water encroachment is detected, pumping should be stopped, whereupon the denser saline water will tend to return to the lower levels of the aquifer. Thus, continuous pumping represents the worst method of operation, intermittent abstraction being preferable. Indeed Ineson[76] described a system of selective pumping from the Chalk aquifer in

Sussex which represented an attempt to control groundwater deterioration due to saline intrusion. In this area of the south coast of England, groundwater development after 1945 had been limited due to increasing salinity. Ineson recorded that the chlorine ion concentration at one point reached 1000 mg/l, as compared with a normal value not exceeding 40 mg/l. The scheme adopted involved pumping from the wells near the coast during the winter months, whereas output was reduced from the inland stations. This led to the interception of fresh groundwater at the coastal boundary which would otherwise flow into the sea. As a result, the storage of groundwater in the landward part of the aquifer increased. During the summer months the pumping scheme was reversed. This method of operation not only controlled saline intrusion but also achieved an overall increase in abstraction[73].

Although it is difficult to control saline intrusion and to effect its reversal, the encroachment of salt water can be checked by maintaining a fresh water hydraulic gradient towards the sea (by Darcy's law this means that there must also be fresh water flow towards the sea). This gradient can be maintained naturally, or by some artificial means[65] such as

1. Artificial recharge, which involves either spreading water on the ground surface or injecting water into the aquifer via wells so as to form a groundwater mound between the coast and the area where abstraction is taking place[77] (see Chapter 9). This inhibits the landward movement of sea water. However, the technique requires an additional supply of clean water, while the process of pumping water into the aquifer is rather contrary to the general objective which is to abstract water. There are also problems associated with the operation of recharge wells (see, for instance, Wood and Bassett[78]).
2. An extraction barrier which abstracts encroaching salt water before it reaches the protected inland wellfield. This is analogous to a well-point dewatering system around an excavation. Thus, an extraction barrier consists of a line of wells parallel to the coast. The abstracted water will be brackish and is generally pumped back into the sea. There will probably be a progressive increase in salinity at the extraction wells. These wells must be pumped continuously if an effective barrier is to be created and, apart from being expensive, this can cause logistical problems.

Neither of these two methods of artificially controlling salt water intrusion offers a cheap or a foolproof solution. Thus, when there is a possibility of saline intrusion the best policy is probably to locate the wells as far from the coast as possible, select the design discharge with care and use an intermittent seasonal pumping regime[6,73,76]. Ideally, at the first sign of progressively increasing salinity, pumping should be stopped.

8.7 Nitrate pollution

While saline intrusion has been a cause for concern for perhaps a hundred years or more, the problem of nitrate pollution is more contemporary and not so clearly defined. However, there are at least two ways in which nitrate pollution is known or suspected to be a threat to health[79,80]. Indeed, it has been appreciated for some time that the build-up of stable nitrate compounds in the bloodstream reduces its oxygen-carrying capacity. Infants under one year old are most at risk and excessive amounts of nitrate can cause methaemoglobinaemia, commonly called 'blue-baby'. Consequently if the limit of 50 mg/l of NO_3 recommended by the World Health Organisation (WHO) for European countries is exceeded frequently, or if the concentration lies within the minimum acceptable range of 50 to 100 mg/l, bottled low-nitrate water should be provided for infants. As a result of such precautions, according to White[81], methaemoglobinaemia has been virtually eliminated from Britain; the last recorded case was in 1972.

A more recent health risk concerns the possibility that the combination of nitrates and amines through the action of bacteria in the digestive tract results in the formation of nitrosamines, which are potentially carcinogenic[79,80,82]. The nitrate may be derived from drinking water, although some foods also have a high nitrate content. Nevertheless, Jenson[83] reported that the water supply of Aarlborg in Denmark has a relatively high nitrate content of approximately 30 mg/l, and there was a slightly greater frequency of stomach cancer in Aarlborg than in other towns during the period 1943 to 1972. This appeared to be due to the nitrate content of the drinking water.

Nitrate pollution is basically the result of intensive cultivation[81,84,85]. The major source of nitrate is the large quantity of synthetic nitrogenous fertilizer that has been used since around 1959, although over-manuring with natural organic fertilizer can have the same result. Foster and Crease[86] estimated that in the 11 years between 1956 and 1967 the application of fertilizer nitrogen to all cropped land in certain areas of east Yorkshire increased by about a factor of four (from around 20 to 80 kgN/ha/year). Regardless of the form in which the fertilizer nitrogen is applied, within a few weeks it will have been transformed to $(NO_3)^-$. This ion is neither adsorbed nor precipitated in the soil and is therefore easily leached by heavy rainfall and infiltrating water. However, the nitrate does not have an immediate affect on groundwater quality, possibly because most of the leachate that percolates through the unsaturated zone as intergranular seepage has a typical velocity of about 1 m/year[28]. Thus, there may be a considerable delay between the application of the fertilizer and the subsequent increase in the concentration of nitrate in the groundwater. So although the use of nitrogenous fertilizer increased sharply in east Yorkshire after 1959, a corresponding increase in groundwater nitrate was not apparent until after 1970, when the level rose from around 14 mg/l NO_3 (about 3 mg/l NO_3—N) to between 26 and 52 mg/l NO_3 (6 to 11.5 mg/l NO_3—N). This just exceeds the WHO's recommended limit of 50 mg/l NO_3 (11.3 mg/l NO_3—N).

The effect of the time lag, which is frequently of the order of 10 years or more, is to make it very difficult to correlate fertilizer application with groundwater nitrate concentration. In this respect one of the greatest concerns is that if nitrate levels are unacceptably high now, they may be even worse in the future because the quantity of nitrogenous fertilizer has continued to increase steadily. Foster and Young[87] studied various sites in England and found that in the unsaturated zone the nitrate concentration of the interstitial water was closely related to the history of agricultural practice on the overlying land. Sites subjected to long-term, essentially continuous, arable farming were found to yield the highest concentrations, while lower figures were generally associated with natural grassland. In many of the worst locations nitrate concentrations were found to have exceeded 100 mg/l NO_3 (22.6 mg/l NO_3—N) while individual wells with values in excess of 440 mg/l NO_3 (100 mg/l NO_3—N) were encountered.

In an update of the paper by Foster and Crease[86], Lawrence *et al.*[88] reported on nitrate pollution of chalk groundwater in east Yorkshire a decade after the original study. It appears that there is a rising trend of nitrate concentrations in most of the public groundwater supplies obtained by abstraction from the Chalk. There is also a marked seasonal variation in groundwater quality which shows a strong correlation with groundwater level. This was attributed tentatively to either rapid recharge directly from agricultural soils, or elusion of pore water solutes from within the zone of groundwater level fluctuation. Since nitrate concentrations in the pore water above the water table beneath arable land were found to be two to four times higher than in groundwater supplies, the long-term trend must be towards substantially higher levels of nitrate in groundwater sources.

Goulter[89] observed nitrate concentrations in New Zealand and found that the prevalence of nitrate-fixing crops has more effect than the application of fertilizers. A sharp increase in nitrate levels in the 1970s was attributed to a rise in the level of the water table following a succession of wet winters.

Concern over nitrate pollution was expressed in Britain in the mid 1970s, but interest in the problem appeared to wane after this. However, a directive issued by the European Economic Community (EEC), which becomes effective in 1985, states that the concentration of nitrate in the drinking water of the member countries should not exceed 50 mg/l NO_3. This has stimulated press interest again[90] since it has been stated that in some parts of Britain there will be severe problems in meeting this standard and that at least 100 groundwater sources are either consistently or intermittently over this limit[14].

Schenk[91] studied the problem in Germany and concluded that nitrate levels could fluctuate by large amounts over short periods of time. It was suggested also that transient peaks in nitrate concentration could lead to false conclusions concerning the quality of groundwater. On the other hand, Gabel *et al.*[92] found that in West Germany, the 50 mg/l limit is exceeded for about 10 per cent of the population and that nitrate pollution is increasing. Darimont *et al.*[93] obtained drinking water samples from taps at 200 sites in the wine growing areas of Baden and Wurttemberg (West Germany) and found that 13 per cent of the samples had a nitrate content above the 50 mg/l limit. The high levels of nitrate in the tap water were attributed to the use of nitrogen fertilizers in the vineyards.

In France the problem of nitrate pollution is being taken very seriously. A survey carried out by the Ministry of Health showed that, in 1981, out of the population of 53 million some 2 to 4 per cent had more than 50 mg/l of nitrate in their water supplies[94]. However, it is expected that by 1985 around 2 to 5 million inhabitants will receive drinking water with a nitrate content in excess of this figure. The highest levels are found in areas of intensive cultivation. Proposals to remedy this situation include the reduction of point sources of pollution, modification of the rates of abstraction, dilution of high nitrate waters with less polluted supplies, the development of new sources and denitrification treatment prior to distribution. In addition, the better use of nitrogen fertilizers, improved crop rotation and changes in crop type are advocated as future courses of action[95].

It is apparent that many countries in Europe, and probably in the world, are suffering from nitrate pollution or are likely to do so in the near future. Measures that can be taken to alleviate this problem include better management of land use, mixing of water from various sources, or the treatment of high nitrate water before it is put into supply. In general, the ion exchange process has been recommended as the preferred means of treating groundwaters, although this may not be considered cost-effective at all sources[96]. Clearly, the problem of nitrate pollution, with its attendant health risk, is potentially a very emotive issue and not one to be taken lightly. Until it is definitely established whether or not nitrates–nitrosamines are cancer producing, an enlightened policy may well be to ensure that water supplies always contain less than 50 mg/l NO_3 (bearing in mind the apparent evidence from Aarlborg).

8.8 Other causes of pollution

The list of potential groundwater pollutants is almost endless and those quoted in the introduction to this chapter are only a representative selection. Some of the most common sources of pollution have already been described, but there are others that merit close attention, such as sewage sludge disposal. The sludge arises from the separation and concentration of most of the waste materials found in sewage. Since the sludge contains nitrogen and phosphorous it has a value as a fertilizer. As a result about 50 per cent of the 1.24 million dry tonnes per year of sludge produced in the UK is used on agricultural land. While this does not necessarily lead to groundwater pollution, the presence in the sludge of contaminants such as metals, nitrates, persistent organic compounds and pathogens, does mean that the practice must be carefully controlled[97,98,99,100]. In particular cadmium, which can also originate as metal wastes from mine workings, may give cause for concern. Too much cadmium can cause kidney damage in humans and this metal was implicated in *itai-itai* disease in Japan. Food is the usual source of the cadmium found in humans, although small amounts are also present in water. Thus, sewage sludge disposal on land must be carefully managed to ensure that this metal, or any other constituent, does not constitute a health hazard[101,102]. The European standard for drinking water recommends a cadmium concentration of less than 0.01 mg/l.

The widespread use of chemical and organic pesticides and herbicides is another possible source of groundwater contamination. For instance, Zaki *et al.*[3] recorded a groundwater pollution incident in Suffolk County, New York, involving the pesticide aldicarb. Previously laboratory and field studies had suggested that the use of this pesticide would not result in groundwater contamination. Not surprisingly, Zaki *et al.* called for more comprehensive testing of pesticides and regular monitoring of groundwater quality in sensitive areas.

In Britain there is also concern about the number of organic contaminants in both raw and treated waters. Fielding *et al.*[103] conducted a survey of treated water (some from groundwater sources) and identified 324 organic compounds. Some of the substances found, if present in much higher concentrations, would have grave medical implications. However, many of the organic materials were unidentified due to the limitations of the analytical techniques. The source of this contamination was described as sewage effluent containing treated domestic and industrial wastes, surface run-off from roads and agricultural land and atmospheric fall-out and rain.

In some parts of the world such as Canada and Scandinavia 'acid rain', originating from the combination of industrial gases with atmospheric water, is acidifying not only the surface water resources but groundwater as well. The results from several sites in Norway indicate that in the affected areas the groundwater has a pH lower than normal, but is less acidified than the surface water[104,105]. Acid rain may pose a significant threat to groundwater quality in the future.

The potential of the run-off from roads to cause pollution is often overlooked[106]. This water is often thought to be relatively pure, whereas it can contain chemicals from many sources, including those that have been dropped, spilled, or deliberately spread on the road. For instance, hydrocarbons from petroleum products and urea and chlorides from de-icing agents are all potential pollutants and have caused groundwater contamination[6,65,107]. There is also the possibility of accidents involving vehicles carrying large quantities of chemicals. The run-off from roads can cause bacteriological contamination as well.

Cemeteries and graveyards form another possible health hazard. According to

Bouwer[108] the minimum distance between a potable water well and a cemetery required by law in England is 91.4 m (100 yards). However, it was suggested that a distance of around 2500 m is better because the purifying processes in the soil can sometimes break down and also for aesthetic reasons. Decomposing bodies produce fluids that can leak to the water table if a non-leakproof casket is used. Typically the leachate produced from a single grave is of the order of 0.4 m^3/year and this may constitute a threat for about 10 years. It is recommended that the water table in cemeteries should be at least 2.5 m deep and that an unsaturated depth of 0.7 m should exist below the bottom of the grave (for the reasons explained in Section 8.2).

While most of the cases of groundwater pollution so far considered have been the result of chemical or inorganic compounds, it is worth remembering that overpumping an aquifer can also be regarded as a potential hazard. The danger lies in the natural variations of water quality with depth within the aquifer, the reversal of hydraulic gradients so that pollutants travel in the opposite direction to that normally expected, induced infiltration, saline intrusion and the exhaustion of supplies. All these can have serious consequences, so careful consideration must be given to the selection of the pumping rate and the duration of the abstraction. Overpumping or 'groundwater mining' during periods of water shortage, on the assumption that groundwater levels will recover at a later date, should be avoided if possible. The groundwater level may recover, but the quality may not. Essentially, this is a question of good aquifer management, part of which includes the routine analysis and monitoring of groundwater quality. These topics are considered in Chapter 9, while Jackson[6] describes many other potential sources of groundwater pollution.

8.9 Detection of groundwater pollution

A mass of fluid introduced into an aquifer tends to remain intact rather than mix with, or be diluted by, the groundwater. In many instances of groundwater pollution, the contaminating fluid is discharged into the aquifer as a continuous or nearly continuous flow. Hence the contaminated groundwater often takes the form of a plume. An important part of an investigation of groundwater pollution is to locate and define the extent of the contaminated body of groundwater in order to establish the magnitude of the problem and, if necessary, to design an efficient abatement system. This may be accomplished by drilling but is costly and time consuming. However, because earth resistivity is inversely proportional to the conductivity of groundwater, the location of groundwater which has been contaminated by relatively high concentrations of conductive wastes may be quickly and accurately traced[109].

Under favourable conditions the resistivity method can be used to determine the boundaries of the plume of contaminated groundwater. Vertical electrical soundings are made in areas of known pollution in an attempt to define the top and bottom of the plume. Drillhole logs are used to establish geological control. Next, resistivity profiling is carried out to determine the lateral extent of the contaminated groundwater. In this way a quantitative assessment can be made of groundwater contamination[110]. The method is based on the fact that formation resistivity depends on the conductivity of the pore fluid as well as the properties of the porous medium. There must be a contrast in resistivity between the contaminant and groundwater in order to obtain useful results. Contrasts in resistivity may be attributed to mineralized groundwater with a higher than normal specific conductance due to contamination. However, if the depth to water is too great, then the thickness of overlying unsaturated sediments can mask any contrasts between contaminated and natural groundwater. In addition, geology of the area has to be relatively uniform so that the resistivity values and profiles can be compared with others. As an example, Oteri[111] used the results of an electrical resistivity survey, with a Wenner configuration, to delineate the extent of saline intrusion into the shingle aquifer at Dungeness, Kent.

8.10 References

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Chapter 9

Groundwater management

Groundwater forms an integral part of the hydrological cycle. Most groundwater recharge is the result of precipitation. Less significant, though important locally, is the direct contribution from inland waters (including rivers) and from adjacent, commonly less permeable, water-bearing strata by leakage.

The first step in the development, conservation and optimum management of groundwater resources is a regional appraisal of the hydrogeological and groundwater conditions (see Chapter 3). This requires an overall assessment of the aquifer, including geographical distribution, dimensions, hydraulic and geochemical characteristics and the factors affecting natural recharge and discharge (see Chapter 4). Existing groundwater usage must be fully quantified and understood, while the implications of future developments must also be considered[1].

The quantity of water available from an aquifer depends basically upon the amount of natural recharge, the rate at which groundwater is abstracted by wells and the underground storage available. Downing *et al.*[2] recognized three principal stages in the development of an aquifer.

1. Initial development stage when groundwater abstraction is much less than natural recharge. Spring and river-flows are only affected marginally.
2. Full development stage when groundwater abstraction equals natural recharge over a period of years. The effect of groundwater abstraction on surface water bodies (rivers, lakes, oceans) may be critical during the later stages of development if the aquifer is not managed efficiently.
3. Over-development stage when groundwater abstraction exceeds natural recharge. This may occur as a result of controlled dewatering or 'mining' of the aquifer, or as a result of an inadequate knowledge of recharge volumes and bad management. Any further long-term development of the aquifer must incorporate either artificial recharge or conjunctive use, otherwise there will be a progressive decline in the quality and quantity of water abstracted.

During the initial development stage, groundwater management is fairly straightforward. So long as abstraction is less than natural recharge, few problems will be encountered, provided that the wells are sensibly located and operated (for example, not too near the sea, not overpumped). Under these conditions it would be normal practice to abstract groundwater, pump it to a reservoir, give the water whatever treatment it requires and then distribute it to the consumer.

When the aquifer is apporaching full development the management problems increase significantly. As explained in Chapter 4, over a long period of time the recharge to an aquifer must balance the total discharge, which comprises natural discharge and any abstraction. If this balance is not achieved then groundwater levels will fall progressively, possibly leading to increased mineralization of the water abstracted, dry wells, ground subsidence, induced infiltration, saline intrusion and so on. Under these conditions efficient management is essential and in order to optimize the use of the aquifer, complex patterns of pumping and detailed model studies may be necessary. For instance, Section 8.6 describes a seasonal pumping pattern in which abstraction was alternated between inland and coastal wells. This was designed to intercept groundwater that would otherwise flow into the sea while keeping saline intrusion under control. Of course, one of the most difficult problems can be to recognize the fact that the aquifer is actually fully developed. Estimates of groundwater recharge and natural discharge tend to be rather imprecise, while the total quantities of recharge, discharge and abstraction change every year. Consequently, it may be appreciated that an aquifer is approaching full development, but almost impossible to determine exactly when this state is reached. Nevertheless, before this condition is attained, management decisions should already have been made concerning what happens next. Either groundwater abstraction must be frozen at existing levels, or plans must be advanced to operate artificial recharge or conjunctive use schemes. Neither of these last two options are without their problems and both require some form of detailed pilot study to ascertain their feasibility. Thus, planning must be many years ahead of requirements.

To reiterate, once a wellfield is in operation it must be managed efficiently in order to optimize both the quantity and the quality of the resource. Thus, the primary objective of groundwater management is to ensure that the aquifer continues to provide economically, water of an adequate quality at a rate equal to the demand, provided that this does not exceed the perennial yield of the aquifer. Overall management strategies may range from the direct supply of water to the consumer (after treatment, of course), to the adoption of more complex and sophisticated concepts incorporating the conjunctive use of surface and groundwater resources. In most situations, the simplest management option is preferable, although one of the skills of management is to ensure a smooth transition from one option to another in the face of rising demand. Inefficient management may have many consequences, one of which is the inability to meet the demand for water over long periods of time.

Groundwater management, therefore, is a comprehensive term that encompasses both the routine day-to-day aspects of wellfield operation, and the long-term considerations relating to future water demand, potential sources of supply, economics and other related topics.

Many of the routine aspects of groundwater management have been covered adequately in earlier chapters, since they are also fundamental to the design and operation of a water well. Typically these include the requirement that

1. The drawdown in a confined aquifer should not extend below the base of the confining stratum.
2. The drawdown in an unconfined aquifer should not exceed around 50 to 60 per cent of the saturated thickness.
3. The well must be designed and operated in such a fashion that the well never becomes dry, that is, the water level never falls below the pump intake. This means that the effect of nearby wells and any long-term decline in water level must be anticipated.
4. Well structures, including abandoned wells, should be inspected regularly for signs

of deterioration so as to reduce the risk of pollutants entering the aquifer.
5. Routine monitoring of groundwater quality should be undertaken.
6. Predetermined procedures should exist to deal with any pollution emergency that may arise.

Although many of these requirements appear routine, they have an important role to play in groundwater management. These considerations form a constant input to the decision-making process and may provide operational constraints that necessitate a modification of the original management objectives (*Figure 9.1*). For instance, the original objective may be to increase the output from a wellfield by constructing new wells in a previously undeveloped part of the aquifer. However, the planned expansion may be curtailed if it transpires that the water quality from a pilot well is deteriorating as a result of induced infiltration. Water must, therefore, be obtained from an alternative source. In this example the data from the monitoring system are used to modify the original objective and to create a new operational strategy involving, perhaps, increased abstraction from existing wells, artificial recharge or conjunctive use. Thus, objectives and strategy must be constantly reviewed in the light of information being recovered from the monitoring network. Rigid adherence to an inflexible plan will almost certainly lead to failure.

It is also important to recognize other constraints upon the management process.

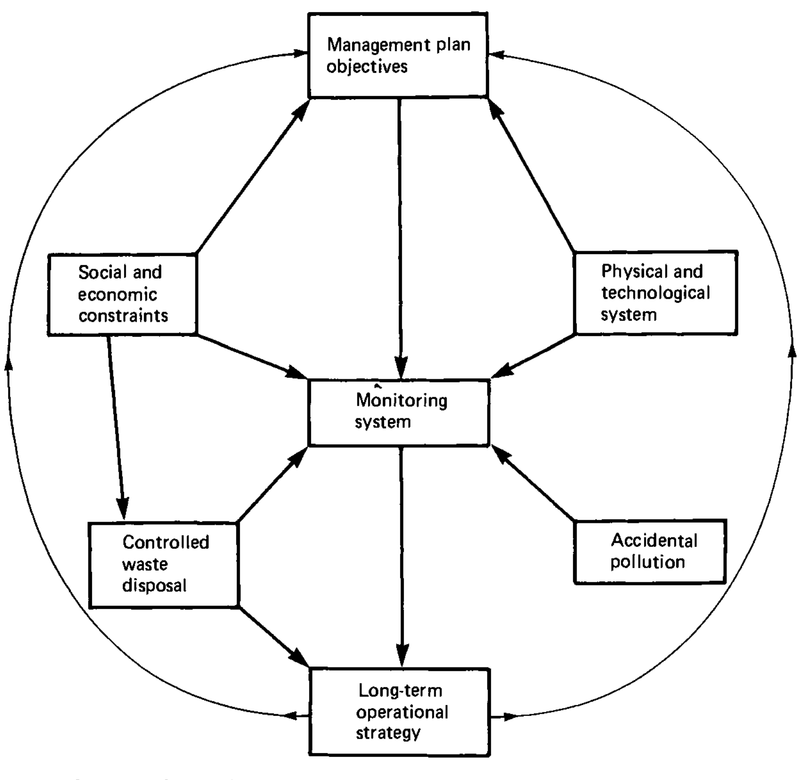

Figure 9.1 Some factors in groundwater management and the evolution of a long-term operational strategy

These include social considerations, the protection of the environment and the ecology of an area affected by groundwater abstraction, agricultural requirements, technical and economic limitations and so on (*Figure 9.1*). In particular, the repercussions of large changes in groundwater level must be appreciated. This topic is considered below.

9.1 Groundwater level considerations

A progressive decline in groundwater level is frequently an indication of future management problems, since this is often the consequence of exceeding the 'safe' or perennial yield of the aquifer. The result, as described in Chapter 4, is likely to be an unacceptable pumping lift, a reduced yield due to the restricted drawdown available and possibly a deterioration in water quality. The latter often occurs as a result of old, highly mineralized water being drawn from deep in the aquifer into the wells, or through induced infiltration or saline intrusion. These problems can necessitate a reduction in the output of a wellfield, or even its abandonment. However, falling groundwater levels may also result in the loss of natural marshes and wetlands, with potentially serious agricultural and ecological implications[3].

It may not always be appreciated that a reduction of the groundwater level can lead to subsidence of the ground surface. Subsidence due to the withdrawal of groundwater has developed with most effect in those groundwater basins where there was, or is, intensive abstraction. Such subsidence is attributed to the consolidation of sedimentary deposits in which the groundwater is present, consolidation occurring as a result of increasing effective stress. The total overburden pressure in partially saturated or saturated deposits is borne by their granular structure and the pore water. When groundwater abstraction leads to a reduction in pore water pressure by draining water from the pores, this means that there is a gradual transfer of stress from the pore water to the granular structure. Put another way, the effective weight of the deposits in the dewatered zone increases since the buoyancy effect of the pore water is removed. For instance, if the water table is lowered by 1 m, then this gives rise to a corresponding increase in average effective overburden pressure of 10 kPa. As a result of having to carry this increased load the granular structure may deform in order to adjust to the new stress conditions. In particular, the porosity of the deposits concerned undergoes a reduction in volume, the surface manifestation of which is subsidence. Scott[4] pointed out that subsidence does not occur simultaneously with the abstraction of water from an underground reservoir. In fact, it occurs over a larger period of time than that taken for abstraction.

The significance of the forces transmitted through the granular structure of a deposit was first recognized by Terzaghi[5], who advanced the classic relationship

$$\sigma' = \sigma - u \tag{9.1}$$

where σ is the total stress, σ' is the effective stress and u is the pore water pressure. *Figure 9.2* illustrates the effect of lowering the water table on the effective and pore water pressures. In aquifers composed of sand and/or gravel the consolidation which takes place due to the increase in effective pressure is more or less immediate.

Consolidation may be elastic or non-elastic depending upon the character of the deposits involved and the range of stresses induced by a decline in the water level. In elastic deformation, stress and strain are proportional, and consolidation is independent of time and reversible. Non-elastic consolidation occurs when the granular structure of a deposit is rearranged to give a decrease in volume, that decrease

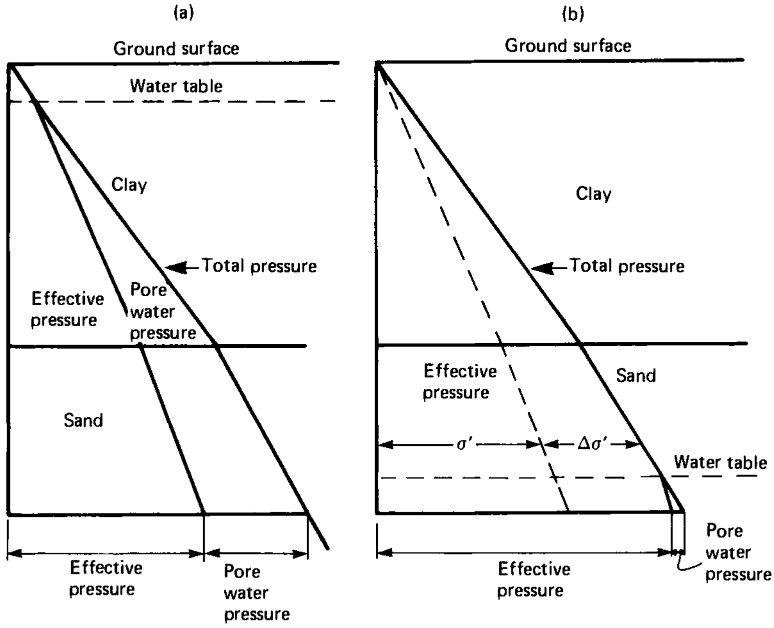

Figure 9.2 Pressure diagrams to illustrate the influence of lowering the water table on effective pressure. (a) Water table just below the ground surface. (b) Water table has been lowered into the sand and effective pressure is increased accordingly. In the clay the effective pressure and total pressure are the same

being permanent. Generally recoverable consolidation represents compression in the pre-consolidation stress range, while irrecoverable consolidation represents compression due to stresses greater than the pre-consolidation pressure.

According to Lofgren[6] the storage characteristics of compressible formations change significantly during the first cycle of groundwater withdrawal. Such withdrawal is responsible for permanent consolidation of any fine-grained, interbedded formations. The water released by consolidation represents a one-time, and sometimes important, source of water to wells. During a second cycle of prolonged pumping overdraft, much less water is available to the wells and the water table is lowered much more rapidly. As an illustration, drawdown in the San Joaquin Valley takes place during a few months each year, for the rest of the time the water levels are recovering. Because of the cyclic nature of the groundwater abstraction, the elastic component of storage change for a given net decline in water level may be reduced and restored many times while the inelastic component is removed but once. *Figure 9.3* illustrates the relationship between fluctuations in water level in semi-confined and confined aquifers, changes in effective stress consolidation, and surface subsidence recorded by Lofgren[7] at a site near Pixley. It can be seen that each year consolidation commenced during the period of rapid decline in head, continued through the pumping season and ceased when the head began to recover.

The amount of subsidence which occurs is governed by the increase in effective pressure, the thickness and compressibility of the deposits concerned, the length of time over which the increased loading is applied, and possibly the rate and type of stress applied. For example, Delflache[8] reported that the most noticeable subsidence in the

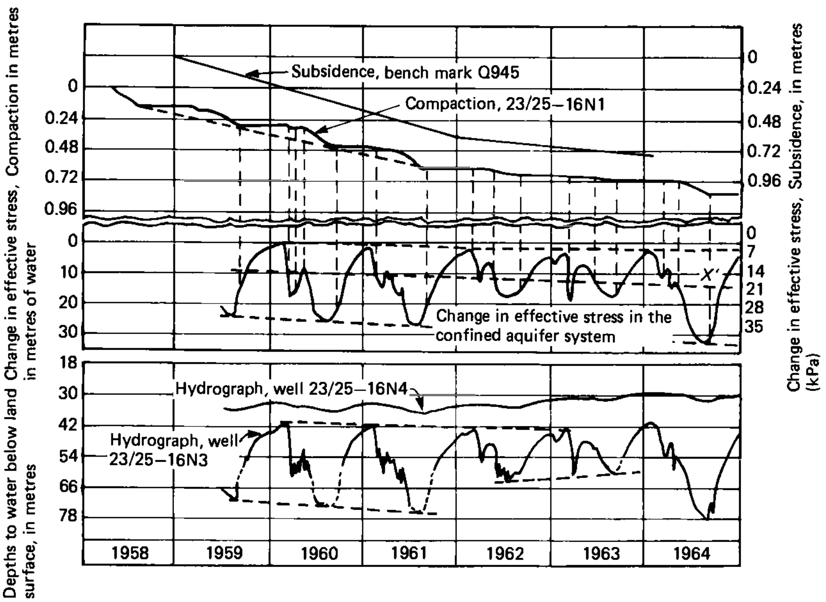

Figure 9.3 Land subsidence, compaction, water level fluctuations and change in effective stress 4.8 km south of Pixley (after Lofgren[7])

Houston–Galveston region of Texas has occurred where the declines in head have been largest and where the thickness of clay in the aquifer system is greatest (*Figure 9.4*). Furthermore, the ratio between maximum subsidence and groundwater reservoir consolidation is related to the ratio between depth of burial and the lateral extent of the reservoir[9]. In other words, small reservoirs which are deeply buried do not give rise to noticeable subsidence, even if subjected to considerable consolidation. By contrast, extremely large underground reservoirs may develop appreciable subsidence.

The rate at which consolidation occurs depends on the rate at which the pore water can drain from the system which, in turn, is governed by its permeability. For instance, the low permeability and high specific storage of aquitards and aquicludes under virgin stress conditions means that the escape of water and resultant adjustment of pore water pressures is slow and time-dependent. Consequently in fine-grained beds the increase in stress which accompanies the decline in head becomes effective only as rapidly as the pore water pressures are lowered toward equilibrium with the pressures in adjacent aquifers. The time required to reach this stage varies directly according to the specific storage and the square of the thickness of the zone from which drainage is occurring and inversely according to the vertical permeability of the aquitard. In fact, it may take months or years for fine-grained beds to adjust to increases in stress. Moreover, the rate of consolidation of slow-draining aquitards reduces with time and is usually small after a few years of loading.

Over 20 years of precise field measurements in the San Joaquin and Santa Clara Valleys in California have demonstrated, as mentioned above, a close correlation between hydraulic stresses induced by groundwater abstraction and consolidation of water-bearing deposits. Lofgren[6] noted that the stress–strain characteristics of the producing aquifer systems have been established and that their storage parameters have been determined from such data. This, in turn, has provided a means by which the

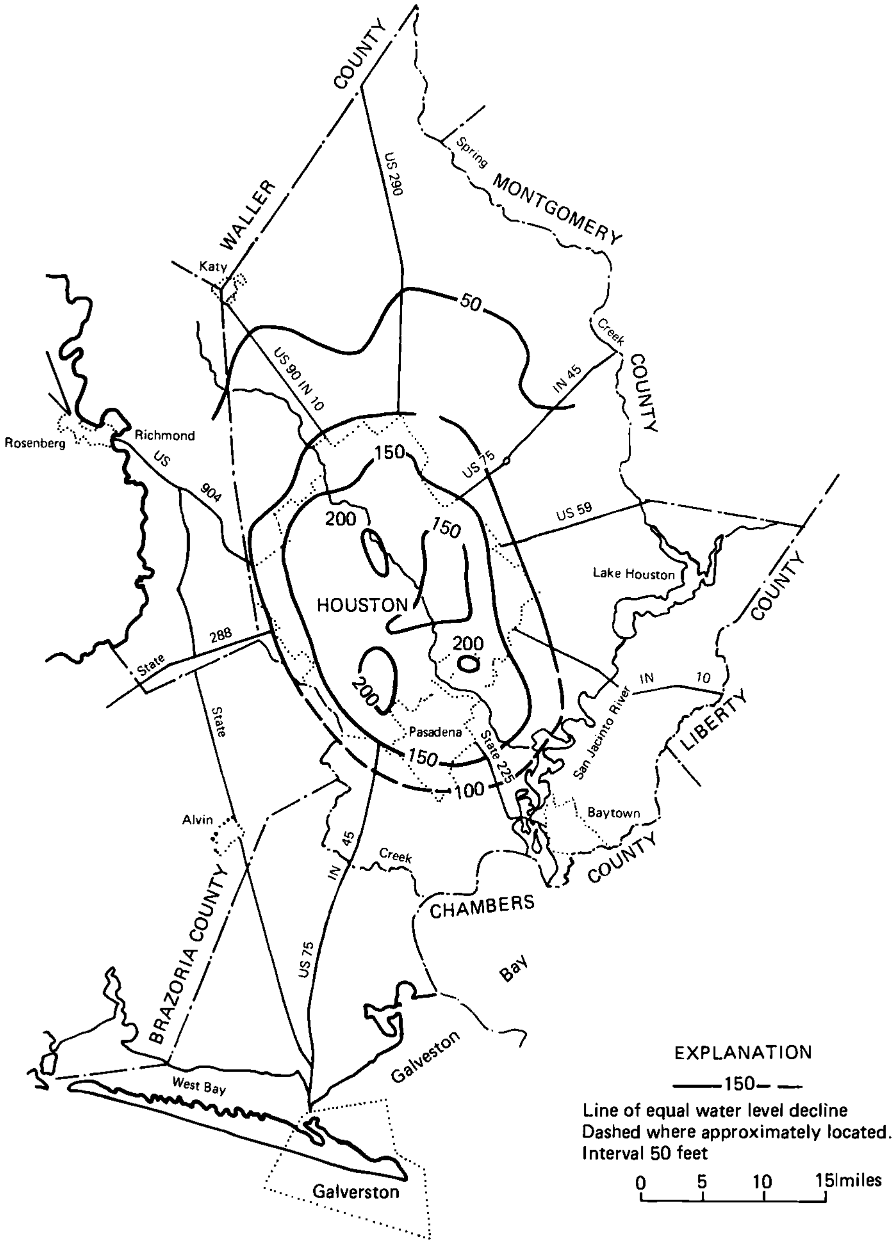

Figure 9.4 (a) Decline of water levels in the Houston District, Texas, 1943–64 (after Gabrysch, R. K., *Land Surface Subsidence in the Houston-Galveston region Texas,* Proc. 1st Int. Symp. Land Subsidence, Tokyo, Int. Ass. Hydrol. Sci., UNESCO Publ. No. 88, Vol. 1, 43–54 (1969))

response of the groundwater system to future pumping stresses can be predicted.

A number of steps which can be taken to evaluate the potential subsidence likely to occur as a result of withdrawal of groundwater from an aquifer have been outlined by Saxena and Mohan[10] and are as follows

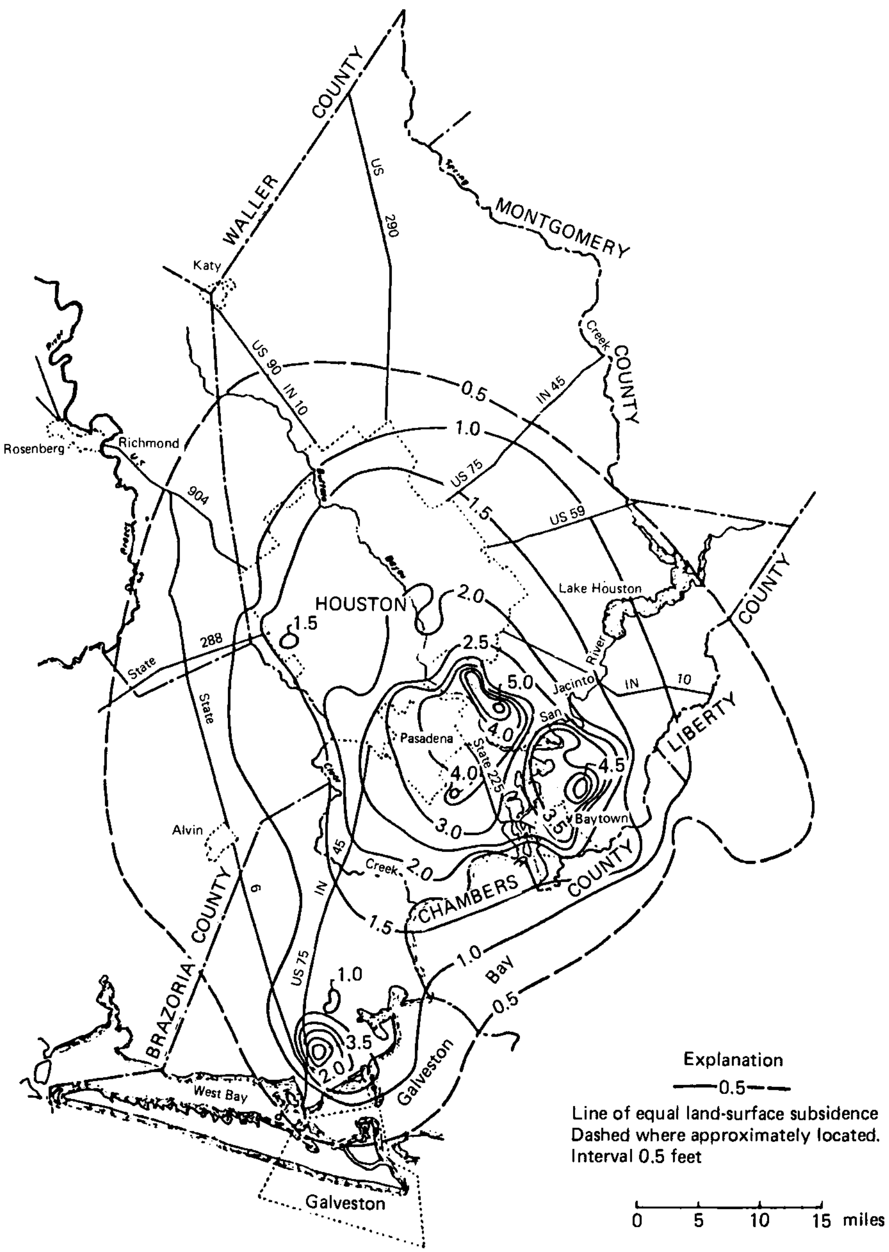

Figure 9.4 (b) Subsidence of the land surface in the Houston District, Texas, 1943–64 (after Gabrysch, R. K., *ibid.*)

1. Define the *in situ* hydraulic conditions.
2. Compute the reduction in pore water pressure due to the removal of the required quantity of water.

3. Convert the reduction in pore water pressure to the equivalent increase in effective pressure.
4. Determine the deformation in the aquifer zone and the confining layers and translate it into the resultant ground subsidence.

Gambolati and Freeze[11] developed a mathematical model which related the occurrence of subsidence in the Venice region to groundwater withdrawal. They noted that subsidence attributable to such action frequently has been analysed in terms of the Terzaghi[12] theory of one-dimensional consolidation (see, for instance, Domenico and Mifflin[13]). Ideally, however, subsidence requires the determination of deformation which occurs in all three dimensions. Subsidence prediction in relation to the rate of groundwater pumping should therefore involve relating the consolidation model to a two- or three-dimensional hydrological model based on the groundwater flow equation. Variations in the hydraulic head in both time and space in response to groundwater abstraction are obtained from the hydrological model. These values can then be used to derive the time-dependent consolidation curve at any point in the system. This provides an indication of the amount of subsidence likely to occur.

A water management study undertaken in the Harris–Galveston Coastal Subsidence District (HGCSD) to predict the effect of groundwater abstraction on surface subsidence has been described by Pollard and Johnson[14]. The first phase of the study involved the compilation of data relating to subsidence, hydrogeology and water supply within the area. The second phase used these data to develop computer models to predict subsidence, the models being based on three possible scenarios of future groundwater abstraction. The study indicated that because of the restrictions placed upon groundwater withdrawal in the eastern part of the District, rates of subsidence have been minimized. However, the models predict that the western part of the District will continue to subside by up to as much as 1.8 m in the centre of the area.

In addition to being the most prominent effect in subsiding groundwater basins, surface fissuring and faulting (*Figure 9.5*) may develop suddenly and therefore pose a greater potential threat to surface structures[15,16,17,18]. In the USA such fissuring and faulting has occurred especially in the San Joaquin Valley, the Houston–Galveston region and in central Arizona. These fissures, and more particularly the faults, frequently occur along the periphery of the basin. The faults are high-angled, normal faults, with the downthrow on the side towards the centre of the basin. Generally displacements along the faults are not great, less than a metre, but movements may continue over a period of years. Holzer[17] related the annual variations in the rate of faulting to annual fluctuations in groundwater levels. Geophysical investigations in central Arizona have revealed that upward projections of bedrock lie beneath many fissures[19].

Holzer and Thatcher[20] argued that such faults did not extend beneath the zone where stresses due to lowering of the groundwater level occur. Hence their depth is shallow. They quoted as an example the Pichacho fault in central Arizona. This fault originated as a result of groundwater abstraction but only extends to a depth similar to the thickness of the alluvium affected by the decline in water level[21].

Subsidence attributable to the withdrawal of groundwater has occurred in numerous regions throughout the world[22] (see *Table 9.1*). One of the classic areas affected by such subsidence is Mexico City. According to to Carillo[23] subsidence took place at a rate of 1 mm/day in some parts of the city. Water has been abstracted for over 100 years from several sand aquifers occurring in very soft clay. These extend beneath the city from an approximate depth of 50 m below ground level to about 500 m. In 1933 the rate of

Figure 9.5 Earth fissure in south-central Arizona. Fissure results from erosional enlargement of tension caused by differential subsidence. The subsidence is caused by groundwater level decline (courtesy of Dr. Thomas L. Holzer, US Geological Survey)

withdrawal was 7 m^3/s from 2200 registered wells. The decline in head in the wells ranged from 0.4 to 2.05 m/year. Overall, the piezometric level had fallen by some 30 m, corresponding to an increase in vertical effective stress of approximately 300 kPa. By 1959 most of the old city had undergone at least 4 m of subsidence and in the north east part as much as 7.5 m had been recorded. This has had serious consequences for both structures and drainage[24]. A prohibition order came into effect in 1953 which meant that no further wells were to be sunk in the Valley of Mexico. Subsequently there has been a slow reduction in the rate of abstraction of groundwater.

Chi and Reilinger[25] identified 48 localities in the USA which have undergone measurable subsidence due to the decline in groundwater levels in subsurface aquifers. They maintained that such subsidence was more widespread in America than had hitherto been reported. More detailed accounts of notable subsidences which have occurred in California have been given by Bull and Poland[26] and Poland *et al.*[27]; in Texas and the Gulf Coast by Gabrysch[28] and Davis and Rollo[29]; and in Arizona by Winnika and Wold[30]. Accounts of other notable subsidences have been provided by Carbognin *et al.*[31,32] for the Venice region; by Brand and Paveenchana[33] and Rau and Nutalaya[34] for Bangkok; and by Nakano *et al.*[35] for the Tokyo plain. Wilson and Grace[36] described subsidence which occurred in many areas of London as a consequence of groundwater abstraction from the Chalk during the period 1865 to 1931.

It is not only falling or low groundwater levels that cause problems, a rising or high water table can be equally troublesome. Since the mid 1960s the rate of abstraction from the Chalk below London has decreased significantly so that water levels are now increasing by as much as 1 m/year in places[37] (see *Figure 9.6*). The potential consequences of this includes leaks in tunnels and deep basements and a reduction in pile capacity. Similar problems exist in the Witton area of Birmingham, England where

TABLE 9.1. Areas of major land subsidence due to groundwater overdraft (from Poland[22])

Location	*Depositional environment and age*	*Depth range of compacting beds* (m)	*Maximum subsidence* (m)	*Area of subsidence* (km^2)	*Time of principal occurrence*	*Remedial or protective measures taken*
Japan						
Osaka	Alluvial and shallow marine; Quaternary	9–400	3	192	1928–68	Reduced groundwater pumpage; built dykes, drainage pumping plants
Tokyo	As above	9–305	4.3	192	1920–70+	Reduced groundwater pumpage; built dykes; drainage pumps; pumping stations
Mexico						
Mexico City	Alluvial and lacustrine; late Cenozoic	9–49	8.6	128	1938–70+	Reduced pumpage; imported water; built recharge wells
Taiwan						
Taipei basin	Alluvial and lacustrine; Quaternary	9–244	1.3	128	1961–69+	Groundwater management code adopted; recharge planned
United States						
Arizona, central	Alluvial and lacustrine; late Cenozoic	91–550	2.3	640	1948–67	—
California						
Santa Clara Valley	Alluvial and shallow marine; late Cenozoic	55–305	4	640	1920–70	Built detention dams; increased local recharge; built dikes; imported water
San Joaquin Valley (three subareas)	Alluvial and lacustrine; late Cenozoic	61–915	2.9–8.5	10 750 (> 0.3 m)	1935–70+	Built reservoirs and imported water to reduce groundwater pumpage
Lancaster area	Alluvial and lacustrine; late Cenozoic	61–305	0.9	384	1955–67+	—
Nevada						
Las Vegas	Alluvial; late Cenozoic	61–305	0.9	512	1935–63	Moved well field away from fine-grained deposits; imported Colorado River water
Texas						
Houston–Galveston area	Fluvial and shallow marine; late Cenozoic	61–610	0.9–1.5	6784 (> 0.15 m)	1943–64+	Plans for surface-water imports under way
Louisiana						
Baton Rouge	Fluvial and shallow marine; Miocene to Holocene	46–610	0.3	640	1934–65+	—

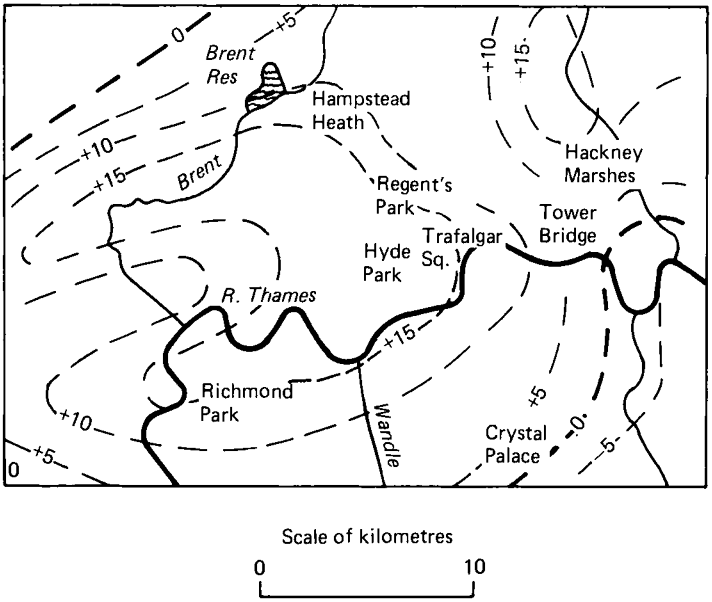

Figure 9.6 Changes in groundwater levels in the Chalk below London, 1965–80 (all contours in metres) (after Marsh and Davies[37])

factory basements are being flooded by rising groundwater. Cheap water from the mains is reported to have resulted in reduced groundwater abstraction, allowing the water table to rise to its natural level[38]. In central Liverpool a similar problem affects British Rail tunnels. In Louisville, Kentucky, increasing groundwater levels are causing concern over the possibility of structural settlement, damage to basement floors and the disruption of utility conduits[39].

The control of groundwater levels has, therefore, an importance which extends beyond water supply considerations. Clearly, if structures are built during a period when the water table is at a particular level (possibly an artificial or atypical level) then care must be taken to ensure that changing water levels do not diminish the integrity of such structures. This obviously requires skillful long-term management of the groundwater resource to ensure that there are no large fluctuations in level. However, it should not be forgotten that aquifers are sometimes deliberately overpumped or 'mined' during a period of water shortage. The usual assumption is that groundwater levels will recover during the following wet season and that no harm will be done. While this may be generally true, it is important that the risks involved in this type of operation are fully realized. This includes not only the risk of subsidence, but also the possibility that mineralized groundwater of considerable age may be drawn up into the well, or that saline intrusion or induced infiltration will occur. Consequently, overpumping should be carried out only infrequently, if at all, and only for short periods of time. Since the long-term hazards potentially far outweigh the short-term gains, the decision to overpump should be taken with caution. Routine monitoring of water quality should always be carried out in such cases and pumping should stop if the quality deteriorates significantly.

9.2 Groundwater monitoring

The routine monitoring of groundwater level and water quality is a fundamental part of aquifer management[40,41]. Not only does this provide an early warning system for pollution incidents and phenomena such as over-abstraction, induced infiltration and saline intrusion, it also provides essential background data that may be required for comparison purposes as well as information that is vital to the effective management of the aquifer (*Figure 9.1*). Todd *et al.*[42] outlined a method of groundwater quality monitoring which consisted of a number of steps taken in a given order. The method has been described subsequently in much more detail by Everett[43].

Because groundwater recharge and the subsequent flow of water through the aquifer is a slow and imprecise process, it is not always possible to calculate the perennial yield accurately. However, a detailed analysis of the chemicals present in the groundwater and the changes that occur over a period of many years, may give a valuable insight into aquifer recharge–discharge mechanisms. This may also indicate whether or not the aquifer has reached a fully developed state and if any alterations to the long-term management objectives are necessary[44].

In almost all situations where groundwater monitoring is undertaken, it is important that adequate background samples are obtained before the groundwater abstraction scheme, waste disposal operation, or whatever, is inaugurated. Without this background data it will be impossible to assess the effects of the new development. Thus routine all-year-round monitoring is essential. It should also be remembered that groundwater can undergo cyclic changes in quality, so any apparent changes must be interpreted with caution[45]. In particular, the extrapolation of short-term data should be viewed with suspicion.

Pfannkuch[46] considered unanticipated contamination and pointed out that the first important step in designing an efficient groundwater monitoring system is the proper understanding of the mechanics and dynamics of contaminant propagation (e.g. soluble or multiphase flow), the nature of the controlling flow mechanism (e.g. vadose or saturated flow) and the aquifer characteristics (e.g. permeability, porosity). Pfannkuch then listed the objectives of the monitoring programme required to investigate an accidental spill (although many of the objectives are equally applicable to known sources of pollution). These include

1. Determination of the extent, nature and degree of contamination (source monitoring).
2. Determination of the propagation mechanism and hydrological parameters so that the appropriate countermeasures can be initiated.
3. Detection and warning of movement into critical areas.
4. Assessment of the effectiveness of the immediate countermeasures undertaken to offset the effects of contamination.
5. Recording of data for long-term evaluation and compliance with standards.
6. Initiation of research monitoring to validate and verify the models and assumptions upon which the immediate countermeasures were based.
7. Initiation of case preparation for litigation or other regulatory activities.

Obviously these objectives may have to be changed to suit the physical, political, or other conditions prevailing at the site of a particular pollution incident. Frequently it is not practical to initiate countermeasures to combat groundwater pollution once it has occurred, the eventual attenuation of the pollutant being the result of time, degradation, dilution and dispersion. In the case of a recent accidental spill, however, it may be possible to excavate affected soil and use scavenger wells to intercept and recover some of the pollutant before it has dispersed significantly[40]. Alternatively, in a few situations, artificial recharge may be undertaken in an attempt to dilute the pollutant, or to form a hydraulic pressure barrier that will divert the pollutant away from abstraction wells. None of these options provides a reliable method of dealing with groundwater contamination and they should only be considered as a last resort.

9.2.1 Construction of a groundwater quality monitoring well

The construction of a water quality monitoring well is very similar to that for an ordinary observation hole and many of the same principles apply. For instance, the design of the well and the method of construction must be related to the geology of the particular site. The depth and diameter of the well should be as small as possible so as to reduce the cost, but not so small that the well becomes difficult to use or ineffective. However, an additional requirement when water quality is concerned is that, if possible, the well structure should not react with the groundwater. Clearly, if the groundwater does react with the well casing or screen, any subsequent analysis of samples taken from the well will be affected. Diefendorf and Ausburn[47] pointed out that the quality of a groundwater sample is only as good as the well from which it is obtained. Thus, monitoring wells are frequently constructed using plastic casings and screens, partly for economy and partly because plastic is relatively inert. Plastic does react with some types of pollutant, however, so the well materials must be selected to suit the anticipated conditions.

The screen should be provided with a gravel or sand pack to prevent the migration of fine material into the well. The selection of a suitable screen slot width and particle size distribution for the pack is accomplished using the same procedures as for a water supply well, although the design entrance velocity should be lower for a monitoring well than a water well. Generally, a sample of the aquifer material is obtained, its grain size distribution determined and the appropriate particle size for the pack decided. The pack should extend at least 0.3 m above and below the screened zone. In very fine sand, silt or clay it may be advisable to wrap a filter fabric around the well screen. A typical monitoring well is shown in *Figure 9.7*.

When deciding the diameter of a monitoring well, some consideration must be given to the method that is to be used to obtain the water sample. If a bailer or some form of sampler is to be inserted down the casing, then obviously the well must be of a sufficiently large diameter to permit this. There are, however, several alternatives. Water samples can be obtained using a specially designed submersible pump which is small enough to fit into a 50 mm diameter well. The pump is made of materials that are essentially chemically inert and it may be run from 12 V rechargeable battery pack. Other proprietary systems are available that use pneumatic pressure to obtain water samples via high-pressure nylon or plastic tubing. Up-to-date information regarding such devices can be obtained from the technical advertisements in any of the journals dedicated to groundwater monitoring.

After a well has been completed it should be developed, by pumping or bailing, until the water becomes clear. Background samples should then be collected over a lengthy period prior to the commencement of whatever it is that the well has been constructed to monitor.

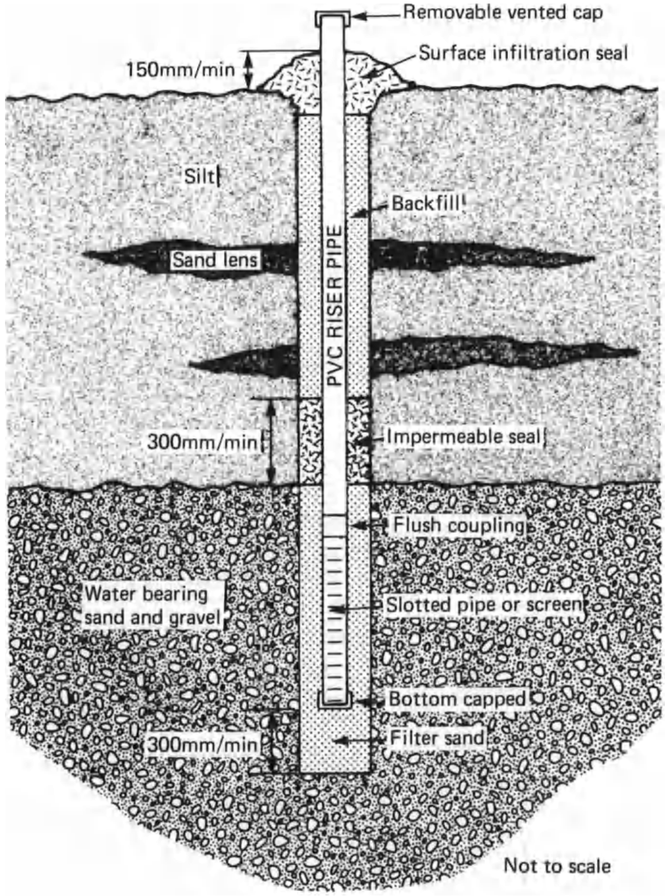

Figure 9.7 Typical details of a monitoring well installation (after Diefendorf and Ausburn[47])

9.2.2 Monitoring well location and the number of wells required

A good water quality monitoring well is quite expensive to construct. When designing a monitoring network the problem is to ensure that there are sufficient wells to allow the extent, configuration and concentration of the contamination plume to be determined, without incurring the unnecessary expense of constructing more wells than are actually required[48]. As with the design of an observation hole network for a pumping test, each site is unique with different requirements. Thus, the network of monitoring wells must be designed to suit a particular location and modified, as necessary, as new information is obtained or in response to changing conditions.

Diefendorf and Ausburn[47] recommended that at least three monitoring wells should be used to observe the effects of a new development such as a landfill site. However, as a result of aquifer heterogeneity, non-uniform flow, variations in quality with depth and so on, it was suggested that three well clusters (rather than individual wells) may be required. Each well cluster should contain two or more wells located at various depths within the aquifer, or within different aquifers in the case of a multiple aquifer system (*Figure 9.8*). One cluster should be located close to the source of the pollution, for early warning purposes, with another installation some suitable distance down gradient to

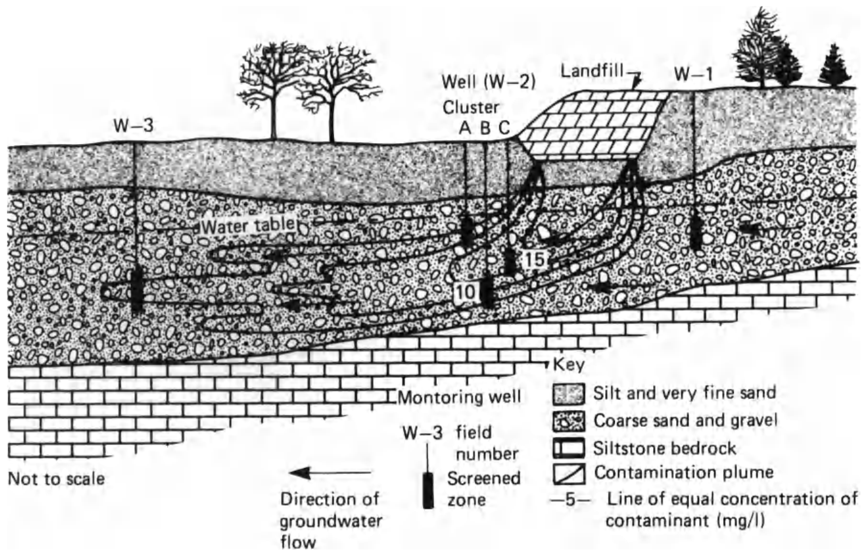

Figure 9.8 Hypothetical example of a well field to monitor a landfill contamination plume (after Diefendorf and Ausburn[47])

assess the propagation of the plume. A third installation should be located up gradient of the monitored site to detect changes in background quality attributable to other causes.

Williams[49] described several instances where over 20 boreholes were needed to identify fully the hydrogeology and plume development originating from a landfill site. Pfannkuch[46] gave an example where 26 boreholes were drilled to monitor an unanticipated spill, 15 of the holes being equipped as monitoring wells. The procedure for establishing a suitable monitoring network for an accidental spill, according to Pfannkuch, is to first of all undertake a rapid preliminary survey. While background water samples are being taken for comparison purposes, the direction of movement of the contaminant is established. As a first approximation it can be assumed that the water table is a subdued replica of the ground surface. To obtain more detailed information, three wells should be sunk in a triangular pattern that encloses the site of the spill (*Figure 9.9*). One well must be up gradient from the source of pollution so that uncontaminated background samples can be obtained. Using three-point interpolation it is possible to draw the groundwater level contours and to construct flow lines, so that a more exact estimate of groundwater movement can be obtained. The flow line(s) that pass underneath the spill site can be identified and at least two monitoring wells should be sunk on this line. The first should be located as close as possible to, but not actually in, the extending plume and the second at some suitable distance downstream. This enables the time of travel of the contaminant between the two wells to be measured, and its propagation velocity assessed. Finally, two wells, or two sets of well should be constructed perpendicular to the central flow line so that the lateral spread of the contamination plume can be investigated (*Figure 9.9*). Again these wells should be close to the advancing plume but not actually in it, so that time of travel data can be obtained. This completes the rapid preliminary survey and the monitoring phase begins with samples being taken at frequent intervals initially. As time progresses and the contaminant begins to disperse and degrade, the monitoring frequency can be

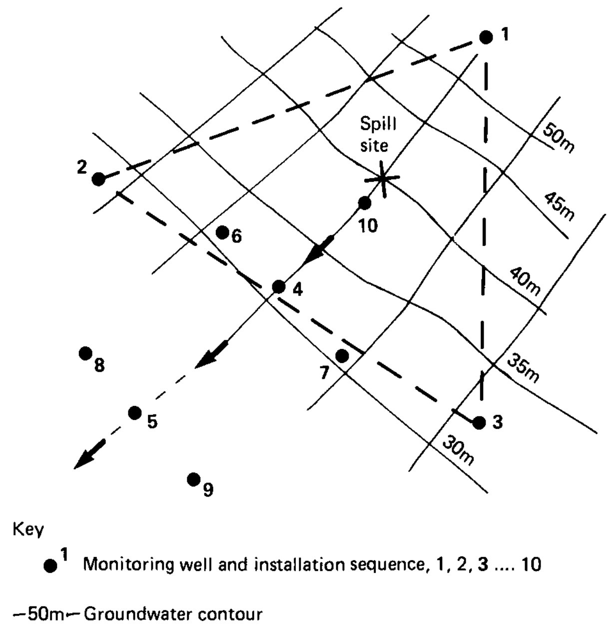

Figure 9.9 Monitoring network required to investigate an unanticipated pollution incident such as an accidental spill

reduced. After a number of months or years, only routine sampling need be undertaken to assess the long-term effects of the spill.

As mentioned previously, over 20 boreholes may have to be sunk to successfully investigate one groundwater contamination incident. However, ine some situations it may be possible to install multiple piezometers and groundwater sampling devices in a single well, so reducing the number of boreholes required. An alternative means of optimizing the design of a monitoring network is to make use of surface geophysical techniques to three-dimensionally delineate the contaminated groundwater. The wells may then be selectively located[50]. Resistivity techniques have proved very successful for this purpose and several case studies are given by Yazicigil and Sendlein[51]. Reported uses of this technique include tracing contaminated groundwater from landfills, acid mine drainage, sewage treatment effluent, septic tanks, oil field brine disposal pits, industrial process waters and spent sulphur liquor. A review of resistivity methods with applications to groundwater was given by Zohdy *et al.*[52]

The monitoring of groundwater quality, regardless of whether this is strictly routine, associated with a known source of pollution, or an emergency response to some form of incident, has an important role to play in groundwater management (*Figure 9.1*). On the basis of these data, long-term plans may be made regarding the aquifer usage, groundwater abstraction, waste disposal operations and future developments. This is, of course, a cyclic process, with plans and options constantly being reviewed in the light of new data from the monitoring network.

9.3 Artificial recharge

Artificial recharge of aquifers has been defined as the practice of increasing by artificial means the amount of water that enters an aquifer[53]. Artificial recharge may be undertaken to

1. Supplement the amount of natural recharge to an aquifer. If the natural recharge (or perennial yield) of the aquifer is less than the average annual demand for groundwater, then artificial recharge may provide a means of increasing the yield of the aquifer.
2. Store water underground for retrieval some time later. In some regions there may be a lack of suitable sites for surface reservoirs, or there may be strong opposition to the construction of surface reservoirs. Under these circumstances, the storage of water in natural underground reservoirs may be an attractive proposition.
3. Optimize water use in an area which suffers seasonal water shortages. In times of plenty, surface water may be diverted to recharge works and the water stored underground for later use during rainless months. This is the basis of most conjunctive use schemes, which have the objective of optimizing the use of the total water resources of an area by adopting an integrated surface–groundwater management policy.
4. Check or reverse saline intrusion. If water is injected into a coastal aquifer this can form a hydraulic pressure barrier between the coast and inland abstraction wells[54,55].
5. Prevent large reductions in groundwater levels as a result of over-abstraction and thus reduce the possibility of ground subsidence, saline intrusion, and so on.
6. Improve the standard of water in a poor-quality aquifer.

Artificial recharge is not a new concept. Indeed, in the Lee Valley, direct recharge through wells was first carried out by the East London Water Company around 1890[56]. Comprehensive details of artificial recharge practice in countries such as Germany, Sweden, USA, Holland, Switzerland, Australia and the UK were given by Buchan[57], Taylor[53], Brown and Signor[58] and Huisman and Olsthoorn[59].

One essential pre-requisite of artificial recharge is, of course, a supply of water. The source of water may be storm run-off, river or lake water, water used for cooling purposes, industrial waste water, or sewage effluent. Many of these sources require some form of treatment before use, not just because of the risk of polluting the aquifer, but because of the possibility of an interaction between the recharged water and the groundwater. Interaction may lead to precipitation, for example, of calcium carbonate or iron and manganese salts. Bacterial action may lead to the development of sludges. These two processes tend to clog the well screen and reduce the permeability of the aquifer, resulting in lower rates of recharge. Nitrification or denitrification and possibly sulphate reduction, may occur during the early stages of recharge[60]. Thus, artificial recharge is not necessarily a straightforward process. To some extent the problems encountered depend upon the nature of the recharge works.

There are several ways in which artificial recharge may be accomplished. These include recharge basins or ditches, spray irrigation and recharge wells. Recharge basins are one of the most widely adopted means of artificially recharging groundwater. To be effective, however, the ground must have a high infiltration capacity and the aquifer must be unconfined and preferably within 2 to 3 m of the ground surface. The bottom of the basin or trench may penetrate the aquifer provided that a filter is employed, although in some countries it is considered safer to have a natural unsaturated zone between the water table and the bottom of the recharge works[53]. Typically, one or more interconnected basins are constructed near to a river or surface reservoir so that surplus water may be diverted into them for recharge purposes. Obviously, recharge basins cover quite a considerable area. An output of about 100 000 m^3/day can be expected from 1 km^2 of land[61]. Average recharge rates through individual basins can be of the order of about 0.3 to 0.5 m/day at water depths of 1 to 2 m, although recharge rates where recharge is practiced intermittently may be between 0.15 and 0.3 m/day.

The quality of the water used in basin recharge is not critical. In fact, basin recharge can constitute an important element in water treatment, acting rather like a slow sand filter, leading to the elimination of suspended solids, ammonia, bacteria and viruses. Under such circumstances the basin requires periodic cleaning, the top 10 to 30 mm of the floor being replaced by clean material. Recharge basins have been in operation for decades in some parts of the world and they appear to be capable of yielding good-quality water despite some pollution of the original water supply to the basin[62,63].

Recharge basins are often of permanent construction, but temporary basins or ditches can be constructed by building dykes across a river at the beginning of each dry season, or by digging ditches leading from the river on to the floodplain. In these cases the intention is simply to spread the river water over as large an area as possible so as to increase the natural rate of infiltration. Old gravel pits can also be utilized successfully as recharge basins.

Spray irrigation needs to be carried out over large areas of land. Recharge rates are generally quite low, because no head of water can be applied, although enhanced water quality is obtained. Spray irrigation is limited to ground with a high infiltration capacity, areas where the water table is near the ground surface and to periods of active plant growth. Care has to be taken not to flush salts and nutrients from the soil into the groundwater. In regions with hot climates, there may also be a danger that excessive evaporation of the recharge water will cause a build-up of salts in the soil. Both of these mechanisms could result in decreased fertility and reduced crop yields.

Recharge wells are most frequently employed when the aquifer to be recharged is deep or confined, or when there is insufficient space for recharge basins. When the flow is intergranular recharge wells are typically between 500 and 900 mm in diameter and penetrate a good way below groundwater level with a screened section extending up to the highest anticipated groundwater level under recharged conditions.

The effect on the water table of artificial recharge (and in many instances natural recharge) is a groundwater mound or inverted cone of depression. In the case of an unconfined aquifer, there really is an increase in the elevation of the water table at the point of recharge (*Figure 9.10*). With a confined aquifer, however, the saturated thickness cannot increase so the mound represents a local increase in the piezometric pressure. The shape of the mound depends upon the method of recharge, such as basins or wells, the geometry of the recharge works, the volume of water recharged and the characteristics of the aquifer. The dimensions of the recharge mound resulting from basin recharge can be calculated, while the cone of recharge arising from the operation of a well can be analysed by using the Dupuit assumptions in the same way as for a cone of depression around a discharging well[66]. However, these equations tend to be rather inaccurate. In the case of a recharge basin the recharge rate is likely to be affected by precipitation and clogging of the filter and aquifer. This is also true of a well recharge. Additionally, the difference between a recharging well and a discharging well is rather more than the fact that the flow is reversed. With an unconfined aquifer the saturated thickness and hence the transmissivity, increases around a recharging well, so theoretically the recharge rate could exceed the maximum abstraction rate. In reality this rarely happens because whereas a discharging well carries silt and suspended solids out of the aquifer, a recharging well pushes them into the aquifer so that problems with clogging are worsened. Consequently, some form of well rehabilitation, such as surging or jetting, must be undertaken regularly. Problems with the clogging of wells and recharge basins can be reduced by using water of potable quality and by chlorination. However, if the water has been treated to this standard, it may be more economical to pipe it straight to the consumer[67].

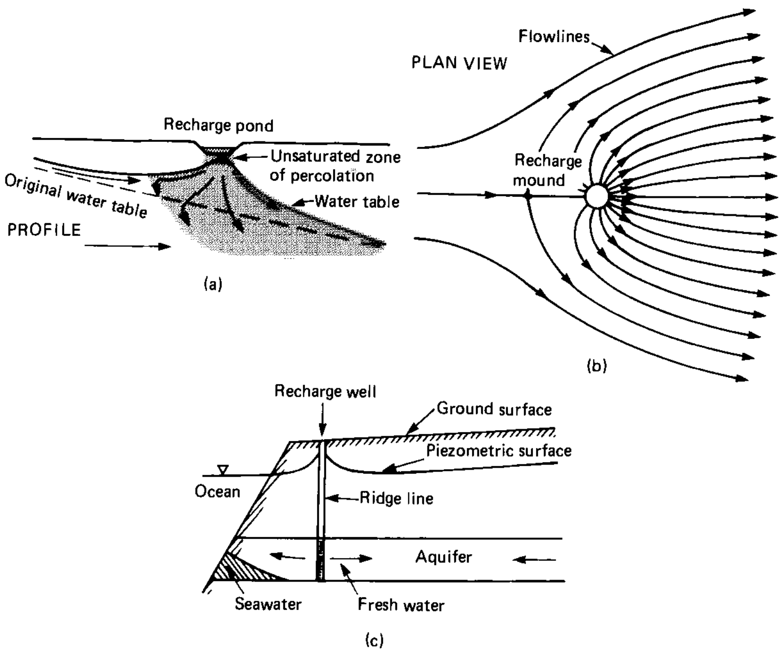

Figure 9.10 The effect of groundwater recharge on aquifer water levels. (a) Groundwater mound on a sloping water table (after Deutsch[64]). (b) Pattern of streamlines associated with a recharge mound (after Smith[65]). (c) Recharge of a confined coastal aquifer via wells forming a pressure ridge or barrier that controls saline intrusion (after Todd[66])

Since a recharge mound represents either an increase in the elevation of the water table or a raised piezometric pressure this brings about a realignment of the flow lines in the affected area. Some flow lines may originate from the mound since this signifies a source of water, while other flow lines originating some distance up-gradient from the area of recharge may be diverted around the mound. Locally the hydraulic gradient may even be reversed. These effects can be useful if it is desired to change the direction of flow in part of the aquifer. For example, the artificial recharge of a confined aquifer through a line of wells parallel to the coast may produce a pressure ridge or hydraulic barrier that will prevent saline intrusion and the discharge of groundwater to the sea.

Before embarking upon an artificial recharge scheme, the suitability of the aquifer for this type of operation must be investigated. In particular, the aquifer must have an adequate storage potential and the bulk of the water recharged should be recoverable and should not be lost rapidly by discharge, say, to a nearby river. Attention must also be paid to the compatibility of the recharge water with respect to the groundwater, otherwise chemical interaction may lead to operational problems. The importance of the initial investigation can be illustrated by an example.

The Bunter Sandstone occurs widely in Britain, particularly the Midlands, but this porous rock has never been used for the storage of water. To establish the feasibility of artificial recharge, a £2 million prototype scheme was started in May 1981. The plan envisages taking water from the river Severn for approximately 90 per cent of the year, treating it, then pumping it underground through a recharge borehole. The water will be retrieved by another borehole and pumped to the consumer (*Figure 9.11*). It is hoped that about 9000 m^3/day will be made available as a result of this project.

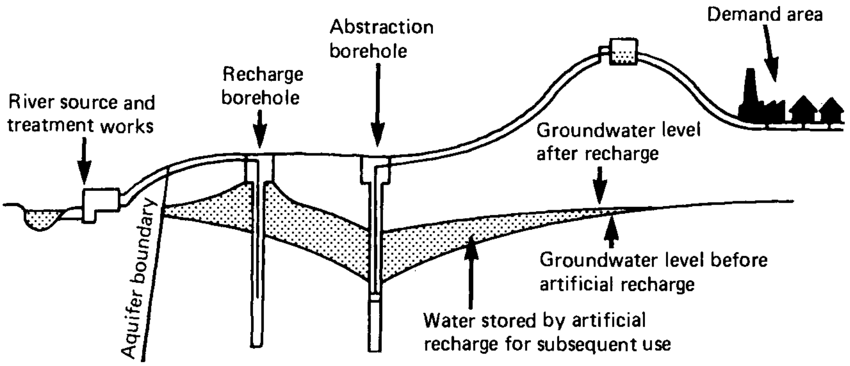

Figure 9.11 Schematic diagram of the artificial recharge works for the Bunter Sandstone. The abstraction and distribution systems are also shown diagrammatically (after Anon[68])

The advantages of the scheme are claimed to be economy, little or no effect on the environment, small land take, and reduced abstraction costs due to the higher water table[68]. Thus, artificial recharge appears to offer significant advantages over other methods of operation when the conditions are favourable, but it should be remembered that the pilot scheme will incorporate something like 15 observation holes in addition to the recharge and abstraction wells. Additionally, the response of the aquifer to recharge has been investigated using modelling techniques, and the scheme will take five years to implement fully. These facts, together with the estimated cost of £2 million in 1981, indicate that artificial recharge projects do not necessarily form a quick, cheap, easy panacea for all problems. If a recharge project is to be successful, it must be carefully investigated, monitored and efficiently managed. If insufficient attention is paid to the preliminary feasibility studies, the result may be a waste of time, effort and money.

9.4 Conjunctive use

The use of surface reservoirs to regulate river flow, usually for the benefit of downstream abstraction and water treatment works, is quite common. The idea of using groundwater for the same purpose is not so well known. The combined use of surface and groundwater in this fashion is known as conjunctive use. By adopting a conjunctive use approach the differing characteristics of surface and groundwater can be used to optimize the yield of the total water resource. For instance, surface waters are available seasonally, but usually some uncertainty surrounds the time and amount available. Additionally, surface systems are characterized by floods that cannot be captured by impounding reservoirs or used for water supply. While surface reservoirs can be filled rapidly, they are subject to losses by evaporation and seepage. On the other hand, groundwater is usually available in large aquifers in large quantities, with relatively little variation over time. Groundwater reservoirs tend to react comparatively slowly to changes in inflow or outflow. Thus, less uncertainty is involved in predicting future groundwater availability than in predicting surface stream flow. A conjunctive use approach to water supply aims to manage jointly the surface and groundwater resources of an area to obtain a net gain in yield. As demand levels increase towards the upper limits of available resources, a conjunctive use strategy becomes more and more attractive[69].

The concept of the integrated use of surface and groundwater, and of optimizing the resources of a particular area, was described by Ineson and Rowntree[1], Buchan[67] and Maknoon and Burges[69]. A more detailed treatment of the subject was provided by Downing *et al.*[2] Some of the considerations involved in the design and operation of a conjunctive use scheme include the following

1. Groundwater can be used to augment river flow during the dry part of the year, that is summer and autumn in Britain, or during droughts. The quantity of groundwater required will depend upon the variability of the river flow and the level of river regulation adopted, for example 60, 70, 80, 90 per cent of mean flow.
2. The drawdown experienced by the aquifer and thus the time required for groundwater levels to recover, will depend not only upon the properties of the aquifer but also the level of river regulation adopted.
3. Some idea regarding the rate at which groundwater levels will recover naturally can be obtained from a consideration of the aquifer response time. This parameter also gives an indication of the seasonal variation in groundwater flow to a river. The response time can be defined as T/SL^2, where T is the coefficient of transmissivity, S the coefficient of storage and L is the distance from the river to the impermeable boundary of the aquifer or to a groundwater divide which is parallel to the line of the river[2,70].
4. Groundwater levels could be increased during periods of surplus river flow—winter and spring in Britain—using artificial recharge techniques if natural recharge is insufficient or too slow.
5. A reduction in river flow would probably accompany groundwater abstraction from the wells. Pumping would lower the groundwater level with the result that spring discharge and any other effluent discharge from the aquifer would be diminished, while losses through the river bed may be increased (possibly by induced infiltration) and some river baseflow may be intercepted. Obviously the reduction in aquifer discharge will depend upon whether or not the aquifer is hydraulically connected to the river, the hydraulic characteristics of the aquifer (S and T), the response time of the aquifer and the distance between the wells and the river.
6. The efficiency of the conjunctive aquifer–river system is expressed as the net gain, that is the net increase in river flow taking into account any reduction in river flow that occurs as a result of groundwater abstraction. Thus

$$\text{Net gain} = \frac{\text{Groundwater abstraction rate} - \text{Reduction in river flow}}{\text{Groundwater abstraction rate}}$$

7. According to Downing *et al.*[2], the best results are obtained when the aquifer has a relatively low permeability and a high storage coefficient (that is a slow response time). Additionally, the wells should be concentrated in restricted areas, to limit the area affected by pumping and hence reduce the length of river over which decreased flow can be expected.
8. With unconfined aquifers, particularly when they have a fast response time, it may be desirable to site the abstraction wells some distance away from the river. If the wells are too near to the river, induced infiltration may cause a very rapid circulation of water around the aquifer–river system with a neglibible net gain. Siting wells remote from the river does, unfortunately, increase pumping and pipeline costs however.
9. Confined aquifers, because of their small coefficient of storage and fast response time, are not always suitable for conjunctive use, although the apparent isolation of the

surface and groundwater resources may, at first, make them appear to be attractive propositions.

To evaluate fully all the factors listed above and assess the hydrogeology of a site, some form of pilot scheme is necessary. Downing *et al.*[2] described the requirements of such a scheme, while Backshall *et al.*[3] gave a very good account of a pilot study as applied to the river Thet basin in Norfolk, England. A pilot scheme should commence with pumping tests on the individual wells to assess the yield–drawdown relationships and well efficiencies, the hydraulic properties of the aquifer and the relationship between individual wells and the river or any other hydrological boundary. The next stage involves the testing of the whole wellfield to establish the effect of abstraction on the aquifer and on the river. Actual conditions observed during the test are compared with those that would have been observed had pumping not occurred. To understand the system fully, the abstraction rate must be large enough to produce a measurable effect on the river flow and decrease the significance of any predicative errors. Backshall *et al.*[3] used an abstraction rate some three times the average infiltration rate during the proving stage of a pilot scheme. One objective is to understand how each well affects river flow, since this information may be essential to the efficient management of the scheme later on. Other objectives may be to assess the consequences of the abstraction on the ecology of the area, natural wetlands, agriculture and so on. The effect on aquatic plants and animals of adding groundwater to river water should also be studied, since these waters will have differing temperatures and chemical compositions.

Many aspects of a conjunctive use scheme can be studied using modelling techniques and they may be essential for the predictive parts of the pilot study. Possibly the only way in which a conjunctive use scheme can be operated at maximum efficiency is by employing models[71,72]. Mathematical, analog and digital computer models were developed by the Thames and Anglian Water Authorities to assist in the investigation of a scheme to replenish the Thames during severe droughts with water from the Chalk (*Figure 9.12*). The models were used to reproduce the inter-relationships between groundwater level, stream flow and the related factors of rainfall and groundwater abstraction and to help the understanding of observed data. They can also be used to

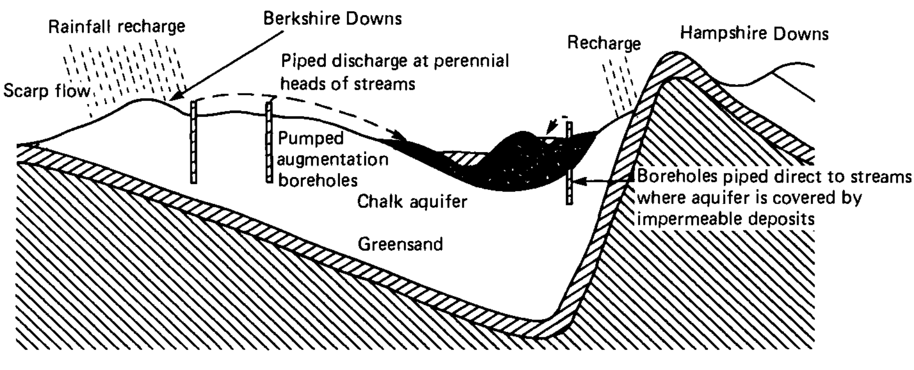

Figure 9.12 North–south section of the Chalk under the Berkshire Downs from which groundwater will be abstracted to replenish the Thames during severe droughts. Where the aquifer is covered by impermeable deposits the wells may be located close to the surface streams so as to reduce pipeline and pumping costs. In the unconfined part of the aquifer the wells must be some distance from the surface streams to prevent induced infiltration. Consequently, the groundwater must be piped down to the perennial heads of the intermittent streams to prevent losses through the dry stream bed (after Anon[73])

predict the effects of different groundwater abstractions and artificial recharge options on the overall hydrological cycle[74].

Conjunctive use schemes are quite complex to operate and require a detailed preliminary study to assess their feasibility. Consequently, before deciding to initiate conjunctive use, it should be certain that the advantages of doing so outweigh the disadvantages. The principal merits and demerits of conjunctive use are summarized below.

Advantages

1. Optimization of water use. Employing both surface and underground reservoirs provides a larger storage capacity and reduces 'wasted' run-off.
2. Smaller impounding reservoirs are needed since groundwater storage can satisfy the additional demand during critical drought periods.
3. Greater flood control. Since water can be transferred from impounding reservoirs to underground storage, the level in surface reservoirs can be dropped to allow for increased flood storage.
4. Greater flexibility when responding to an increase in demand, since more than one source is available. This can lead to greater efficiency when the travel distance of releases is reduced.

Disadvantages

1. Higher running costs as a result of greater power consumption through increased pumping. Most surface regulating schemes operate under gravity, whereas conjunctive use schemes need pumps to recover water from underground, transport it to the river and possibly to artificially recharge groundwater as well. Monitoring costs would also be increased.
2. Decreased pumping efficiency due to large fluctuations in groundwater levels. This is most significant when artificial recharge is practicsed.
3. Management problems are increased because there is a greater variety of options, such as which source to use at any time, when to stop groundwater abstraction and switch to a surface source, when to initiate groundwater recharge and so on. The operation of a large scheme necessitates that a large number of management decisions have to be made, possibly at short notice as both demand and river flows can change rapidly. This may mean that expensive computer systems are required to operate real time monitoring and to control remotely various pumps, valves and switches. Additionally, computer programs may be required to assist in optimizing the management of the resource. Such computer technology is expensive to purchase, program and maintain.
4. Economic assessment of the scheme is more difficult because there are a number of surface and groundwater sources that can be used independently or simultaneously. Selecting the least cost option at any time may be difficult and this may not always provide the most efficient use of water or satisfy other management constraints. Again, some form of modelling is probably required to indicate the optimum strategy at any particular time.
5. If water is derived from different sources at different times, the water supplied to the consumer may change from soft moorland water to hard groundwater. This may cause problems or be unsatisfactory. Some blending of the water from the different sources may be required[67].

As can be appreciated from the above, conjunctive use schemes are complex and have some significant disadvantages. Consequently, it may be advisable not to attempt such a scheme unless there is adequate justification, namely a shortage of water that cannot be satisfied by any other reasonable means. If such a scheme is undertaken, then it should be on a small scale initially and models should be developed to help predict future trends, select the most appropriate options and generally assist the management process.

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Chapter 10

Groundwater modelling techniques

10.1 Introduction

Prickett[1] defined a groundwater model as any system that can duplicate the response of a groundwater reservoir. The operation of the model and the manipulation of the results is termed simulation.

In earlier chapters reference has been made to various kinds of model that have been used for a range of purposes such as the investigation of regional groundwater flow[2], the evaluation of optimal and long-term pumping rates[3,4], saline intrusion[5,6], pollution occurring from highway run-off[7], investigations of conjunctive use schemes[8,9] and so on. In fact, there are now very few areas of hydrogeology that are not amenable to investigation using modelling techniques.

However, models have not always been so commonplace. Their popularity has increased as a result of two factors. First, the increased availability of relatively cheap powerful digital computers (and programmers), and secondly the progressive increase in the demand for water which has necessitated more efficient aquifer management. When an aquifer is in an early stage of development there may be little or no justification for the use of sophisticated models. As the aquifer becomes fully developed (i.e. abstraction roughly equals the recharge) then the need to optimize the management of the groundwater resource naturally leads to the adoption of models to predict the consequences of different regimes of pumping.

Although models are extremely numerous and quite diverse in form, it is possible to group them into certain categories according to their objective or function as follows[10].

1. *Prediction models* Generally simulate groundwater flow in an aquifer. They require information (see Section 10.3) on aquifer characteristics, boundary conditions and pumping rates, while they yield data regarding the direction and rate of groundwater flow, changes in water level, surface–groundwater interconnection and the effects of abstraction. This is perhaps the most common type of model.
2. *Resource Management models* Can be used in tandem with prediction models, include optimization as well as simulation techniques. This type of model is designed to indicate the best course of action to achieve a particular objective, such as minimizing costs or ensuring the maximum rate of supply.
3. *Identification models* Determine input parameters for both of the above types. Any model is only as good as the data upon which it is based. Thus, identification models are

used to determine the hydrogeological input parameters for other models from observations of field data. For example, given the rate of abstraction from a well and drawdown data for several nearby observation holes, it is a relatively simple matter to change the hydraulic characteristics of the aquifer in the model until it responds in a similar fashion to the prototype. These values can then be used in a prediction model, which would simulate the effect of pumping the well in a manner or at a rate for which no field data exist.

4. *Data manipulation models* These handle the data collection networks, process the field data, identify critical data, determine the inputs to other models and store all relevant data.

There are many publications which give a useful introduction to modelling techniques, including examples and case histories. Among the most notable are Prickett and Lonnquist[11], Prickett[1], Davis[12], Rushton and Redshaw[13] and Bredehoeft *et al.*[10]. However, before considering groundwater flow models in more detail, it is worth considering exactly what a model is. It is any system that can duplicate the response of a groundwater reservoir, regardless of whether the basis of the model is physical (that is a scaled-down version of the aquifer) or mathematical. One of the simplest types of mathematical model is the regression or factor model. The objective of such a model is to determine the relationship between two or more variables by conducting a regression or least squares analysis on the data. This effectively achieves the same result as plotting the data and drawing the best fit line through the points. Using regression analysis merely puts this process on a more quantitative basis. The resulting regression equation can then be regarded as a simple model describing the relationship between the variables.

10.1.1 Example of a simple regression (or factor) mathematical model

A factor model is based upon the assumption that variations in a groundwater resource are determined by the influence of a set of factors. The problem is to determine the response function, y, from a set of factors, $x_1, x_2, x_3, \ldots, x_n$. This can be achieved by using regression analysis. A standard computer program can be employed to conduct the necessary calculations, although relatively simple linear regressions can be solved with a calculator. A typical analysis could be concerned with the relationship between groundwater level and abstraction rate, or rainfall and groundwater level as in the example below.

Example

The first three columns of *Table 10.1* show the annual rainfall recorded at a rain gauge located in the recharge area of an aquifer and the corresponding groundwater level at the end of December. The problem is to determine the relationship between groundwater level, y, and the annual rainfall, x, and to decide whether or not the relationship is significant.

Solution

The data are used to calculate a regression equation using the simple linear model $y = a + bx$. *Table 10.1* shows the means and the sums of the cross-products and squares. The individual values of the cross-products and squares do not need to be recorded; the sums of these two values can be accumulated using the memory of a calculator. There are 9 years of data so $N = 9$.

TABLE 10.1. Linear regression data and analysis

Year	*Groundwater level* y (mAOD)	*Annual rainfall* x (m)	xy	x^2	y^2
1967	69.30	0.8344			
1968	68.20	0.7490			
1969	68.45	0.7639			
1970	66.60	0.6252			
1971	65.90	0.5928			
1972	66.25	0.5310			
1973	64.79	0.4873			
1974	66.08	0.6025			
1975	65.96	0.5156			
Sum	601.53	5.7017	382.392	3.7313	40221.595
Mean	66.837	0.6335			

$$b=\frac{\Sigma xy-\dfrac{\Sigma x\Sigma y}{N}}{\Sigma x^2-\dfrac{(\Sigma x)^2}{N}}=\frac{382.392-\dfrac{5.7017\times 601.53}{9}}{3.7313-\dfrac{(5.7017)^2}{9}}$$

$\underline{b=10.99}$ *Regression coefficient*

$a=\bar{y}-b\bar{x}=66.837-10.99\times 0.6335$

$\underline{a=59.875}$ *Intercept*

Then

$y=a+bx$

$\underline{y=59.875+10.990x}$ *Regression equation*

$$Sx^2=\frac{\Sigma x^2-\dfrac{(\Sigma x)^2}{N}}{N-1}=\frac{3.7313-\dfrac{(5.7017)^2}{9}}{8}$$

$\underline{Sx^2=0.01489}$ *Variance of x*

$$Sy^2=\frac{\Sigma y^2-\dfrac{(\Sigma y)^2}{N}}{N-1}=\frac{40221.595-\dfrac{(601.53)^2}{9}}{8}$$

$\underline{Sy^2=2.16688}$ *Variance of y*

$$r=b\frac{Sx}{Sy}=10.99\times\frac{0.122}{1.472}$$

$\underline{r=0.91}$ *Correlation coefficient*

With $v=(N-2)$ or 7 degrees of freedom this relationship is significant at the 0.1% level

The calculations below *Table 10.1* indicate that the regression equation is

$y=59.875+10.990x$

while the correlation coefficient is 0.91. Correlation analysis is a statistical method which measures the degree of association between samples of two variables. The correlation coefficient can vary from $+1$ (indicating complete functional dependence) through zero (independence) to -1 (implying complete dependence in opposing directions). The significance of the correlation coefficient can be determined from tables (see Appendix I) which relate values of the coefficient to different levels of significance. If the absolute value of the calculated correlation coefficient exceeds the tabulated value, it can be concluded that the correlation exists and the level of significance represents the probability of having drawn the wrong conclusion. When using the table of correlation coefficients, it should be noted that v, the number of degrees of freedom, is generally taken as 2 less than the number of pairs of data in the sample.

Simple linear regression, as conducted in the example, achieves the same result as graphical regression analysis, that is simply drawing a straight line through the data by eye and deciding by inspection whether or not the relationship is significant. This may be quicker and simpler than employing a full regression analysis, but the merit of the least squares procedure employed in *Table 10.1* is that it is not subjective and really does yield the best line through the data. Similarly, the full analysis also gives a good quantitative indication of the significance of the relationship, which cannot be obtained by inspection. Unfortunately, correlation analysis is capable of yielding dubious results under certain circumstances such as, for instance, when one pair of data contains a large error but is treated equally with the other data pairs in the analysis. Consequently there is never any harm in quickly plotting the data at the same time as the regression and correlation analysis is being undertaken.

More complex cases involving multiple and non-linear regression can be tackled using readily available computer programs. In any instance where the equation obtained from a regression analysis, either linear or non-linear, is used to extrapolate the relationship, that is, to calculate a value of y from a value of $x_1, x_2, x_3, \ldots, x_n$ that is outside the range of values originally observed, the result should be viewed with suspicion. The relationship in this region could be very different as a result of phenomena such as aquifer boundaries, overflow from the recharge area, surface–groundwater interflow and so on.

10.2 Development of modelling techniques and types of model available

All groundwater flow models must be capable of conforming to Laplace's equation[14,15] (see Section 2.3.2), while the function of the model is to provide a means of solving the groundwater flow equation, or a set of such equations. The development of modelling techniques reflects scientific advances in the fields of electronics and mathematics as much as the state of the art in hydrogeology. The obvious example of this is, of course, the digital computer which has had a tremendous impact, making possible the solution of quite complex problems. Cheap powerful computers only really became commonly available in the 1970s. Prior to this groundwater models generally had to take some other form. Only very brief details of the various types of model and their development are given below, since many of the techniques are now somewhat dated and rarely used. Further details concerning the theory, form and construction of such models can be found, if required, in Prickett[1].

Physical analog models can be thought of as scaled-down representations of an aquifer. The sand tank model is a true model since it generally has the same scaled-down boundary configuration and permeability as the prototype and the same laws govern the flow of water through both the model and the aquifer. As the name implies, sand tank models consist of a suitably shaped glass or plastic container filled with a porous medium such as sand. Piezometers are installed at points where the head of water is of interest, while pumps and flow rate measuring devices are used to control the flow through the tank. This type of model has been used to study phenomena such as unconfined flow, discharge from wells, saline intrusion and so on. The visual nature of these models is their chief asset, making them useful for studies of dispersion, diffusion and multiphase flow problems. However, scale effects, wall effects, bacterial growth, entrapped air and so on, can cause operational problems and limit their effectiveness. A sand tank model was used by Potter and Baker[16] in 1938, and their use has continued into the 1980s[17,18].

The viscous fluid model also represents a scaled-down version of the aquifer, the model aquifer being formed by the space between two parallel plates. The flow of a viscous fluid between two closely spaced parallel plates for steady conditions is governed by Laplace's equation $\nabla^2 h = 0$, as is the steady two-dimensional flow of groundwater[19,20]. Probably the first groundwater application of this type of model was due to Zamarin in 1931[21], who conducted a study of seepage through earth dams, although it was many years later that small storage vessels, distribution and discharge tubes were connected to the space between the plates to enable groundwater storage, recharge and discharge to be modelled[22]. Viscous fluid models are generally split into two main categories according to whether the plates are vertical or horizontal, although they may be at any angle. These models were frequently used to assess the validity of theoretical formulae and to investigate the flow to ditches, wells and so on. Their use continued into the 1970s and such models are still useful for flow visualization purposes, particularly when flow in an inclined aquifer is concerned. The disadvantages of viscous fluid models include relatively complex construction, operational problems such as the need to maintain a constant temperature so as not to affect the viscosity of the model fluid, scale effects, only two-dimensional flow can be studied, difficulty in modifying the model as new data become available, etc. Consequently for many uses this type of model has been superseded by newer, more flexible techniques.

Electrical analog models are similar to physical analogs, except that the model no longer uses a fluid (as in the prototype) but employs the flow of electricity instead. Since the flow of electrical current through a liquid is governed by Laplace's equation, as is the flow of water through an aquifer, it can be appreciated that a conductive liquid model is an analog of the prototype. These models consist basically of an insulated tank filled with an electrolyte whose electrical conductivity is proportional to the hydraulic conductivity or permeability of the aquifer. Two copper sheet electrodes, with a configuration corresponding to the problem or aquifer to be studied, are immersed in the electrolyte. An electrical potential of magnitude equivalent to the field elevation of the water table is then applied to the electrodes and the electrical potential (or elevation of the water table) is measured at other points of interest in the model. Such models have been used to evaluate regional groundwater resources and the flow to wells[23], although they have decreased in popularity in recent years due to the fact that they are rather inflexible and not very convenient to use compared with other modern techniques. However, the alternatives were rather limited when one of the earliest conductive liquid models was constructed by Wyckoff *et al.*[24]. This used blotting paper soaked in electrolyte to study the movement of a fluid injected into an aquifer. Although this was a 'wet' model, it could perhaps be classed as a conductive solid model.

Continuous conductive solid models are very similar to conductive liquid models, only the conductive medium is solid not liquid. The most commonly used conductive solid is Teledeltos paper, which is carbon impregnated paper that has a relatively uniform conductivity. The paper is cut so as to be geometrically similar to the aquifer under study. Barrier boundaries are formed by making slits in the paper, while recharge or constant head boundaries are formed from a conductive strip of silver paint emulsion. An electrical potential equivalent to the fall in the field elevation of the water table is then applied across the two constant head boundaries. The electrical potential, or voltage, is measured at other points of interest using a potentiometer and galvanometer. Teledeltos paper models have been used to investigate regional groundwater flow patterns and the flow to wells, partially penetrating wells, seepage through dams, and so on[25]. One of the earliest conductive solid models dates from about 1948[26]. They are still used for flow visualization and teaching purposes, although their applicability to groundwater flow problems is limited by the fact that they can only really model two-dimensional uniform steady flow in isotropic and homogeneous aquifers.

Another variety of electrical analog model is the discrete conductive solid model or resistance–capacitance network. The basis of these models is virtually the same as for a conductive solid model except that small squares of Teledeltos paper are replaced by discrete resistors arranged in a square network (*Figure 10.1*). An additional refinement is the addition of capacitors to the nodes or junctions of the network to represent the storage properties of the aquifer (for a steady state model the capacitors are omitted). Capacitors store electrical charge in a manner similar to the way in which an aquifer stores water. Thus a resistance–capacitance network is a scaled-down discretized version of the aquifer. Time is also scaled down or compressed in the model, so problems that may take years to evolve in the aquifer can be studied in the model in seconds. These models, often referred to simply as analog models, were developed initially by the petroleum industry and were not applied to groundwater problems until the late 1950s. Their use probably reached a peak in the 1960s when they were employed to study a wide range of complex problems including regional groundwater flow and the flow to wells[28,29].

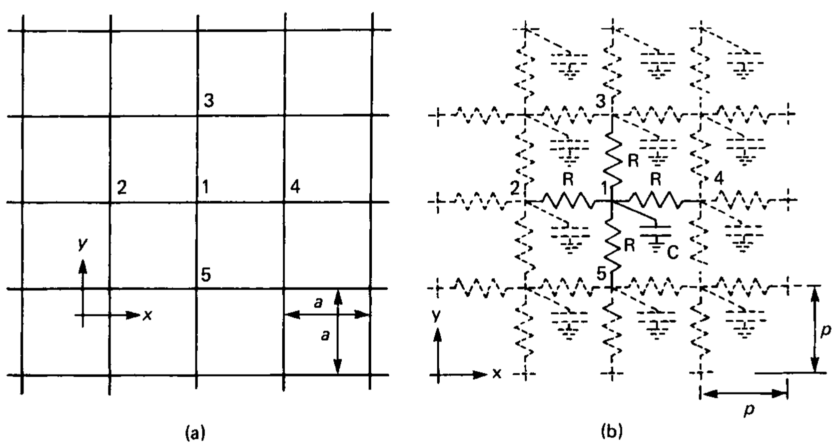


Figure 10.1 Finite difference grid. (a) On aquifer. (b) Equivalent resistor-capacitor network

The analogous quantities between an aquifer and a resistor–capacitor model are as follows

Change in head (m) = change in electrical potential (V)
Hydraulic rate of flow (m^3/day) = electrical current (A)
Hydraulic transmissivity (m^2/day) = electrical conductivity (mhos or seimens)

These relationships can be formalized by the derivation of the following scale factors[30,31].

$$k_1 = \frac{q \text{ m}^3}{Q \text{ coul}} \qquad k_2 = \frac{h \text{ m}}{V \text{ volts}} \qquad k_3 = \frac{Q \text{ m}^3/\text{day}}{I \text{ amps}} \qquad k_4 = \frac{t_d \text{ days}}{t_s \text{ sec}} \qquad k_5 = \frac{a \text{ m}}{p \text{ mm}}$$

and $k_1 = k_3 k_4$. The scale factors must be chosen so as to allow inexpensive and readily available equipment, resistors and capacitors to be used. The values of the latter two can be calculated from

$$R = \frac{k_3}{k_2 T} \text{ ohms} \qquad \text{and} \qquad C = \frac{a^2 S k_2}{k_1} \text{ farads}$$

where T and S are the local aquifer transmissivity and storage coefficients respectively. The value of k_4, that is, the ratio of a time period in the aquifer to the equivalent period in the model, determines whether a fast or slow time model is constructed[29]. Both have their merits, but the advantage of a slow time model is that the equipment required is somewhat simpler than that required for a fast time model and consists only of constant current generators, ammeters and voltmeters[28]. A fast time model is similar but requires the use of a waveform generator, pulse generator and oscilloscope[31].

The problem with resistance–capacitance networks is that the associated electrical equipment and the operation of the model can become quite complex. This can be partially overcome by using a hybrid analog–digital model in which a digital computer is used to perform various switching and control operations[13,32,33]. However, this does not entirely eliminate the problem and by the early 1970s many hydrogeologists felt that the electronics required for anything other than a simple model were too complex and outside their range of competence. This coincided with the increasing availability of digital computers, so many resistor–capacitor models were converted to digital models, which is quite an easy process since the basis of a discrete analog model is very similar to a finite difference model (see below). Nevertheless, analog models probably still have something to offer. Simple models are relatively cheap to construct and cost nothing to operate, yield results almost instantaneously, can be used almost anywhere by anybody and give a very good 'feel' of the problem or aquifer under investigation. Possibly analog models could be developed further and may be suitable for use in Third World countries.

Mathematical models are rather different from those previously described in that they are abstract rather than physical in form. For instance, the equations governing the flow of groundwater are themselves mathematical models. In the simplest case the differential form of Darcy's equation, $q_x = k_x \, \partial h/\partial x$, can be regarded as a model relating groundwater flow to aquifer permeability and hydraulic gradient. A more complex model could, for example, include the Laplace equation (Equation 2.29), but the principle would be the same. Of course, as the problem becomes more complicated the equations become more complex, and as the size of the model increases then the number of equations also increases. In order to solve large problems, a digital computer and appropriate software and numerical techniques may be required to expedite the repetitive calculations involved. For this reason mathematical or digital computer models of groundwater flow problems really became popular around the end of the 1960s and the start of the 1970s as computer availability increased. Generally these models utilize either a finite difference or a finite element technique, although very simple mathematical models such as that described in Section 10.1.1 can also prove very useful.

Finite difference equations have the advantage that they can be derived either by adopting Darcy's law and the principle of conservation of mass (or continuity of flow) or by a conventional mathematical treatment[11,34,35] Both methods of derivation yield identical results. Thus, finite difference techniques provide a useful introduction to modelling for hydrogeologists, since the equations can be derived from principles with which they are already familiar. These methods are considered in more detail in Section 10.4.

Finite elements represent a newer technique that has been used mainly for the analysis of problems in structural engineering. At the present time these methods have not been so widely adopted by hydrogeologists as have finite difference techniques. This is possibly due to the fact that the basis of finite elements is more difficult for hydrogeologists to grasp, being based upon unfamiliar concepts such as the principle of minimum energy.

If an aquifer is divided into a number of triangular elements ((*Figure 10.2*), or any other suitable shape, the properties within an element are regarded as being uniformly constant while the properties may vary from element to element. According to Prickett[1], the solution of the two-dimensional steady flow equation

$$\frac{\partial}{\partial x}\left(T_x\frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(T_u\frac{\partial h}{\partial y}\right)=\pm W \tag{10.1}$$

is equivalent to finding a function h that minimizes the integral

$$F=\frac{1}{2}\iint\limits_R\left[T_x\left(\frac{\partial h}{\partial x}\right)^2+T_u\left(\frac{\partial h}{\partial y}\right)^2\right]\mathrm{d}x\,\mathrm{d}y+\iint\limits_R Wh\,\mathrm{d}x\,\mathrm{d}y-\int\limits_s q_b Hh\,\mathrm{d}s \tag{10.2}$$

where F is a functional, q_b is the discharge per unit area across and normal to an

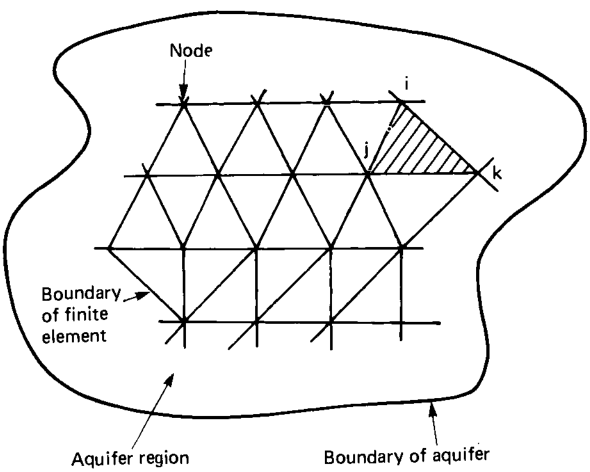


Figure 10.2 Sub-division of an aquifer (continuum) into a mesh of two-dimensional finite elements. The elements, which can be any shape, are identified by the nodal points on their boundaries, such as *i*, *j*, *k*

element, R denotes the limits of the region under study and s is a line along which q_b is specified. For triangular elements, the contribution of each element to the integrals of Equation (10.2) can be expressed as a function of the heads at the corner points and the coordinates of the nodes. If the unknown head values of the distribution, h, at the triangular node positions, define the function throughout the entire aquifer under study, then differentiating the function F of Equation (10.2) with respect to each of the nodal values and equating them to zero results in a system of simultaneous equations[34,36]. This set of simultaneous equations can be solved using one of many available techniques to find the values of h over the mesh.

Although finite elements currently may not be as popular as finite difference methods where groundwater modelling is concerned, this position may change in the near future. Certainly in some branches of engineering, finite elements are regarded as superior and have almost superseded finite difference techniques[37]. However, Davis[12] suggested that finite difference and finite element models of two-dimensional groundwater flow performed equally well, although the finite element techniques were generally less economical. The report concluded that the most accurate method was the backward difference technique known as the Alternating Direction Implicit Procedure with Iteration within each Time-step (ADIPIT). A form of this technique is considered in Section 10.4.2. Aberra[38] also compared finite element and finite difference methods of analysis and found that with large time increments there were considerable discrepancies in the solutions obtained from the two numerical methods. Finite element models are considered in more detail in Section 10.5.

10.3 General data requirements of groundwater models

Any groundwater flow model is only as good as the data upon which it is based. In general, the more data that are available initially, the better will be the completed model. This is true regardless of what type of model is constructed. Most groundwater flow models require data relating to the following if they are to effectively simulate the aquifer prototype[39,40].

1. The extent of the aquifer and the location and nature of any aquifer boundaries.
2. The flow of water into and out of the aquifer.
3. The rest water levels in the aquifer.
4. Variations in the thickness and depth of the aquifer and any confining strata.
5. The spatial variation of the coefficients of transmissivity and storage.
6. Data recorded during the pump testing of wells, such as the discharge of the well and the drawdown recorded at various points in the aquifer.
7. Water level fluctuations in the aquifer over a number of years.
8. The rate of infiltration to the aquifer in the recharge area during the same period.
9. The pumping schedules for the same period.
10. River base flows.
11. Spring locations and spring flows.
12. General background information regarding the hydrogeology of the region, such as areas of interconnection between surface and groundwater, interflow between aquifers, artificial recharge, and so on.

It is extremely unlikely that all these data would be available at the beginning of a model study. Indeed, a model is often commissioned with the objective of supplying such information without having to incur the expense of a field investigation. Models which are to be used for predictive purposes can be based upon quite scanty data initially and updated as more field results become available.

During the construction of a model, two distinct stages of development are often recognized: calibration and verification. The calibration stage requires information relating to items (1) and (5) above and usually involves, for a given rate of groundwater flow, adjusting the transmissivity of the model aquifer until the elevation of the water table or piezometric surface in the model is analogous to that in the prototype. As a first approximation transmissivity values may be estimated from the spacing of the groundwater contours[39] (see Section 4.5.5). Wide contour spacing is generally indicative of a high transmissivity.

The verification stage of development uses information relating to items (5) and (12) and involves adjusting the model until it reproduces satisfactorily the recharge and discharge mechanisms of the prototype. In particular, the model must be able to duplicate adequately two sets of prototype data. First, the specific capacity data, which is the ratio of yield to drawdown at the pumped well, and secondly the drawdown recorded in the observation holes for a given discharge from the nearby abstraction well. When trying to reproduce in the model the drawdowns that occur as a result of pumpage, inspection of the well discharge equations gives several useful relationships that are of assistance. For instance

1. For a given drawdown in a well, the discharge from the well is proportional to the transmissivity in the surrounding area, that is, $Q \propto T$. Consequently, decreasing the transmissivity in the model in the vicinity of the well decreases the discharge.

2. For a given well discharge, the drawdown in the well inversely proportional to transmissivity in the surrounding area, that is, $s_w \propto 1/T$. Thus, decreasing the transmissivity in the model in the vicinity of the well increases the drawdown (*Figure 10.3(a)*).

3. For a given drawdown in a well, the hydraulic gradient between the well and an observation hole is inversely proportional to the transmissivity, that is, $i \propto 1/T$. Thus, increasing the transmissivity in the model between the well and the observation hole increases the drawdown in the observation hole (*Figure 10.3(b)*).

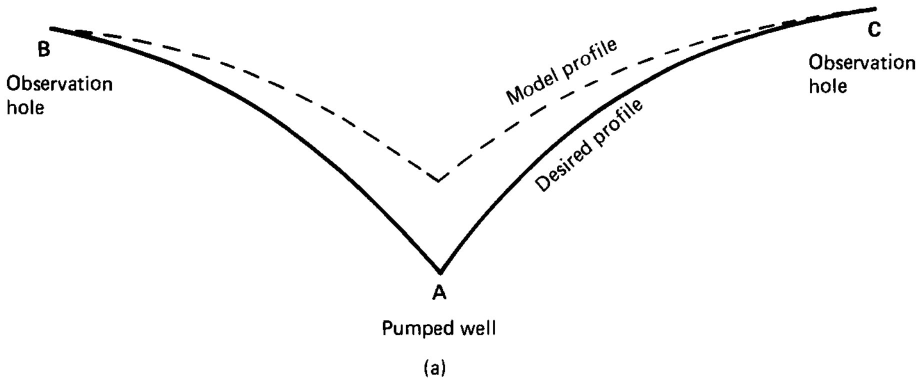
When modelling a discharging well a further consideration is the theoretical direction of flow within a vector volume (Karplus[42] adopted the term vector volume to emphasize that the direction of flow as well as the volume of aquifer under consideration is important). The usual assumptions which are made when assigning the coefficient of transmissivity to an element of the finite difference grid (*Figure 10.1(a)* are

1. All flow lines within a vector volume are parallel to one another (*Figure 10.4(a)*).
2. The cross sectional area of a vector volume is constant over the entire length of flow.
3. The horizontal or plan area of a vector volume is constant and extends throughout the full thickness of the aquifer.

Clearly, these assumptions are not valid where radial flow to a well is encountered (*Figure 10.4(b)*). However, Prickett[27] showed that for an analog model the value of the resistor required at the well nodes can be calculated from

$$R_w = \frac{2}{\pi} R \log_e(a/r_w) \tag{10.3}$$

where R_w is the value of a well node resistor, R is the value of a standard grid resistor, a is the grid spacing and r_w is the well radius. Since transmissivity and resistance values



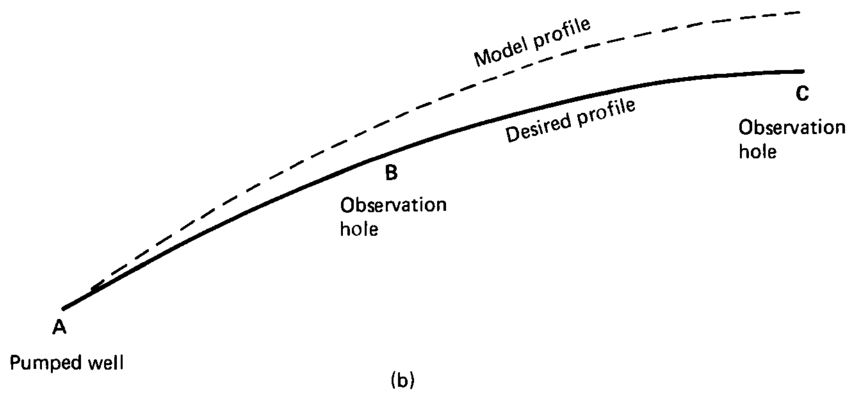


Figure 10.3 Correction of model drawdown in (a) the pumped well and (b) the observation holes (after Blair[41]). (a) Model drawdown is too small and since $s_w \propto 1/T$, the transmissivity in the vicinity of the well should be decreased. (b) Model drawdown is too small and the hydraulic gradient, i, is too large. Since $i \propto 1/T$, the transmissivity values between the well and observation hole, C, should be increased

are inversely proportional, this equation also can be used to calculate the coefficient of transmissivity required at the well nodes of a finite difference model[11].

The squares of a finite difference grid are assumed to extend through the full thickness of the aquifer and the hydraulic constants that are assigned to a node are assumed to be representative of this volume of the aquifer. If the vector volume associated with any particular node is reduced, such as may happen with partial penetration or if a boundary runs from node to node so bisecting a vector volume, then a proportional reduction in the value of the hydraulic constants assigned to the nodes in question must be made. This is exactly the same argument that is used when allocating resistor values to an analog model[42].

The values of the coefficients of transmissivity and storage that constitute a completed model inevitably must be a compromise between the rest water level, specific capacity and pumping test requirements. Thus, the values assigned to various parts of the grid must be changed iteratively until the model adequately duplicates the behaviour of the prototype. This may result in the transmissivity values being unique to a particular part of the grid. On the other hand, the allocation of the storage values is generally relatively straightforward, since the only requirement may be that the time-drawdown data from the pumping tests are reproduced satisfactorily. The values

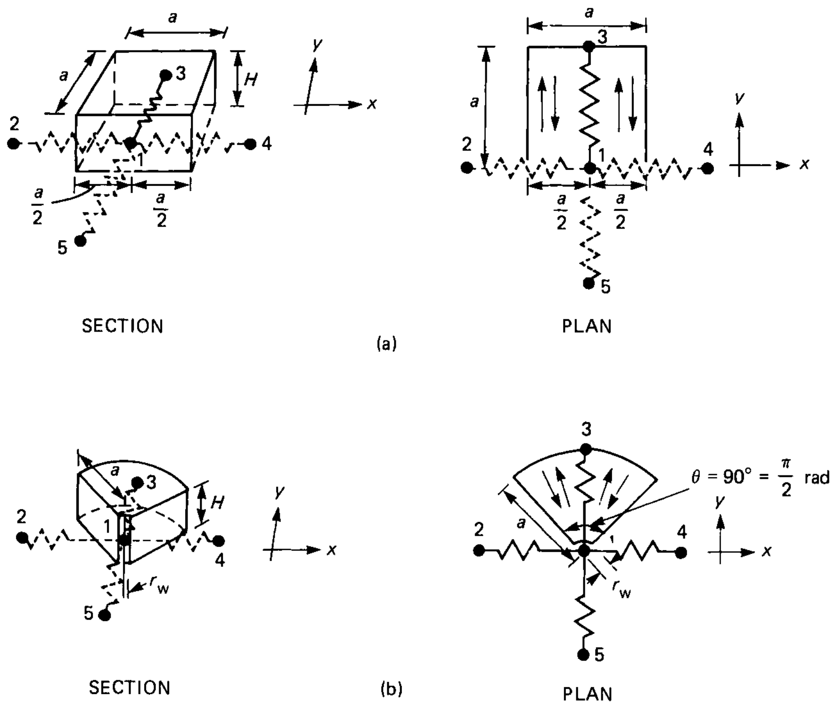


Figure 10.4 (a) Vector volume of a standard grid resistor. (b) Vector volume of a well node (after Prickett[27])

assigned initially would be representative of the particular aquifer material under consideration and these would be modified as additional information becomes available. In most models the same value of storativity can be applied to quite large areas of the grid without reducing the accuracy of the model.

10.4 Finite difference groundwater models

10.4.1 Derivation of the finite difference equations

With digital computer models, both the time and space variables are treated as discrete parameters and the equations governing the flow of water within the discretized model are written in finite difference form. This gives a large set of simultaneous equations that can be solved using any appropriate numerical technique[2,13,34,43,44]

The finite difference equations can be derived either from a consideration of continuity of flow in the aquifer or by substituting finite difference approximations for the derivatives of the two-dimensional equation governing the unsteady flow of groundwater in a homogeneous confined aquifer, which is

$$T_x \frac{\partial^2 h}{\partial x^2} + T_y \frac{\partial^2 h}{\partial y^2} = S \frac{\partial h}{\partial t} \pm W \tag{10.4}$$

The finite difference approach necessitates that the continuous aquifer system is replaced by an equivalent set of discrete values, formed by drawing a uniform (or

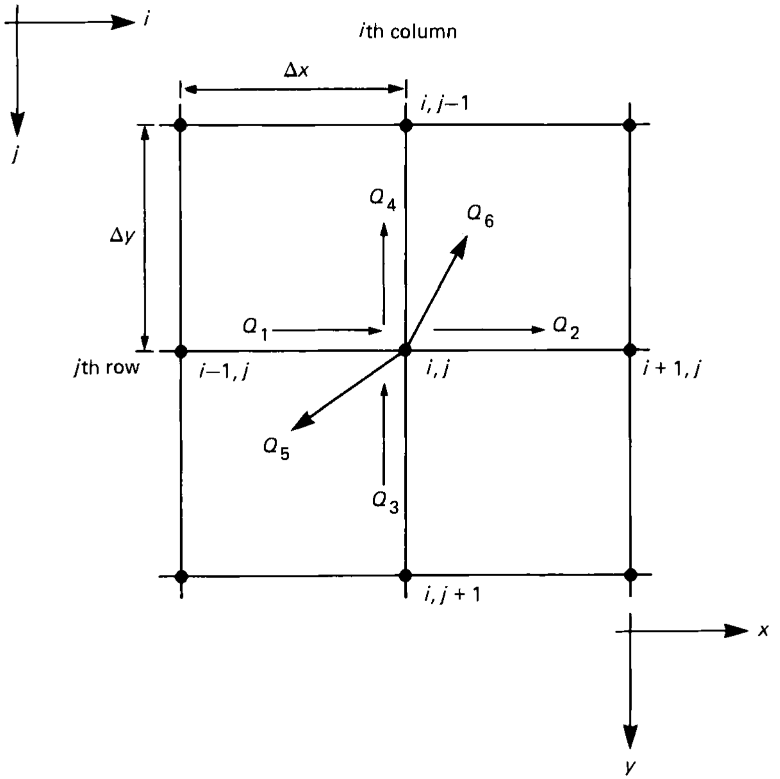


Figure 10.5 Digital model finite difference grid and flow terms (from Prickett and Lonnquist[11])

sometimes non-uniform) mesh or grid over a map of the aquifer (*Figure 10.5*). The aquifer is thus subdivided into volumes with the dimensions $H\,\Delta x\,\Delta y$, where H is the saturated thickness of the aquifer. The flow within any of these volumes should be small compared to that in the aquifer as a whole. The points at which the grid lines intersect are called nodes and these are located by coordinates, i, j, representing (column, row) respectfully as in *Figure 10.5*.

Considering the flow into and out of the node, i, j in *Figure 10.5*, then Q_1, Q_2, Q_3 and Q_4 represent the inter-nodal flows in the i and j directions. The direction of flow has been assigned arbitrarily and has no special significance. Indeed, during the early stages of model construction the direction of flow may not be known. The term Q_5 represents the amount of water released from or taken into storage per unit increment of time, Δt. In this instance water is being taken into storage since the direction of the flow rate term Q_5 in *Figure 10.5* is out of the node i, j. The term Q_6 is equivalent to the source or sink function, W, in Equation (10.4) and is used to represent groundwater abstraction or recharge.

For continuity of flow into and out of node i, j, the following condition must be satisfied

$$\begin{gathered}\text{Inflow} = \text{Outflow} + \text{Change in storage} \\ Q_1 + Q_3 = Q_2 + Q_4 + Q_5 + Q_6\end{gathered} \tag{10.5}$$

The next step is to determine the value of each of the six flow rate terms. This requires

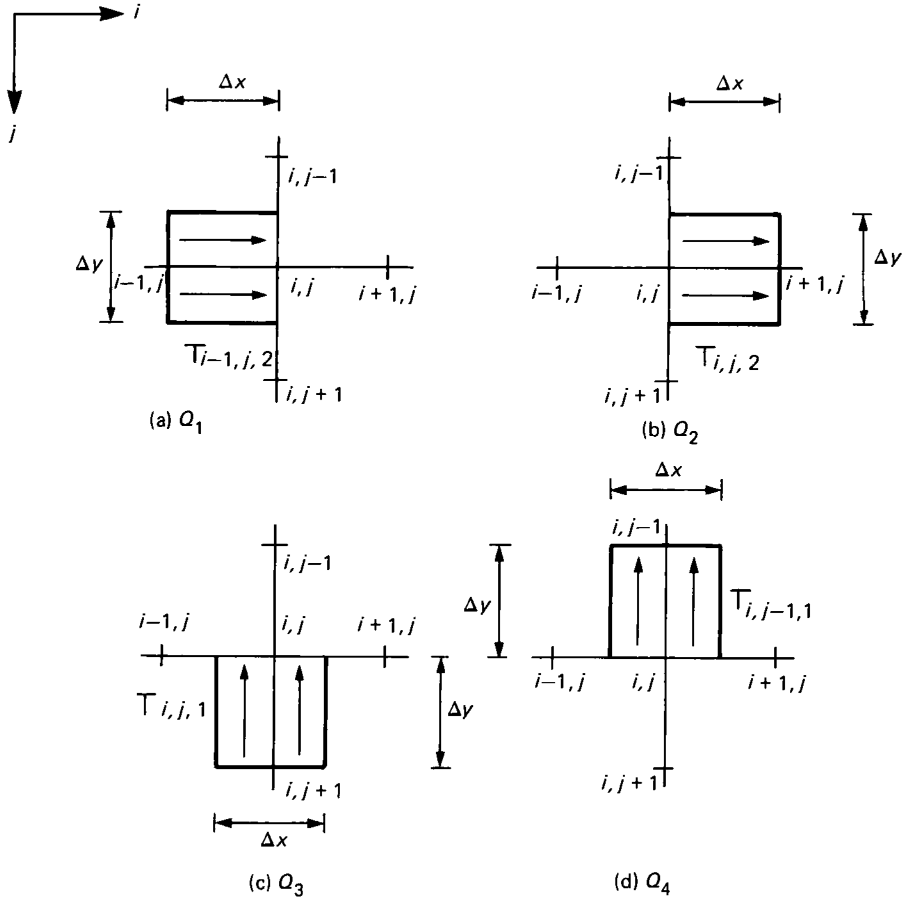


Figure 10.6 Vector volumes for digital model flow terms $Q_1 - Q_4$ (after Prickett and Lonnquist[11])

an appreciation of the fact that the flow into or out of a node, such as Q_1, represents the total flow through a finite volume of the aquifer in the i direction. The vector volumes adopted for the terms Q_1 to Q_4 have the same dimensions as the finite difference grid and are located centrally on the lines joining adjacent nodes (*Figure 10.6*). All the vector volumes extend through the full thickness of the aquifer. When assigning values of transmissivity to the vector volumes it should be noted that the subscripts 1 and 2 are used to denote flow in the j and i directions respectfully, thus $T_{i,j,1}$ and $T_{i,j,2}$. Hence there are two values of transmissivity associated with any given node. When assigning the values of transmissivity to a particular node, the vector volume for $T_{i,j,1}$ always lies below the node in question i,j, that is, in the direction of increasing j. Similarly, the vector volume for $T_{i,j,2}$ lies to the right of i,j, that is, in the direction of increasing i. The apparent flow directions in the vector volumes are not significant in this respect.

Applying Darcy's law to the flow rate terms Q_1 to Q_4 gives

$$Q_1 = T_{i-1,j,2}(h_{i-1,j} - h_{i,j})\frac{\Delta y}{\Delta x} \tag{10.6}$$

$$Q_2 = T_{i,j,2}(h_{i,j} - h_{i+1,j})\frac{\Delta y}{\Delta x} \tag{10.7}$$

$$Q_3 = T_{i,j,1}(h_{i,j+1} - h_{i,j})\frac{\Delta x}{\Delta y} \tag{10.8}$$

$$Q_4 = T_{i,j-1,1}(h_{i,j} - h_{i,j-1})\frac{\Delta x}{\Delta y} \tag{10.9}$$

where $T_{i,j,1}$ is the aquifer transmissivity in the j direction within the vector volume between nodes i,j and $i,j+1$ (see *Figure 10.6*(*c*)), $T_{i,j,2}$ is the aquifer transmissivity in the i direction within the vector volume between nodes i,j and $i+1,j$ (see *Figure 10.6*(*b*)) and $h_{i,j}$ is the calculated head at the end of a time step, t, measured from an arbitrary reference level at node i,j.

For the terms Q_5 and Q_6 the vector volumes have a horizontal area of $\Delta x\,\Delta y$ which is centred around the node point i,j (*Figure 10.7*). These vector volumes extend the full depth of the aquifer. Thus

$$Q_5 = S_{i,j}\,\Delta x\,\Delta y\left(\frac{h_{i,j} - h_{i,j(t-\Delta t)}}{\Delta t}\right) \tag{10.10}$$

where $h_{i,j(t-\Delta t)}$ is the calculated head at node i,j at the end of the previous time step, $(t-\Delta t)$, and Δt is the time increment elapsed since the last calculation of heads.

The term Q_6 represents pumpage or the net withdrawal rate from the vector volume of node i,j so that

$$Q_6 = Q_{i,j} \tag{10.11}$$

Substitution of Equations (10.6) to (10.11) into Equation (10.5) gives

$$\begin{aligned} &T_{i-1,j,2}(h_{i-1,j} - h_{i,j})\frac{\Delta y}{\Delta x} + T_{i,j,1}(h_{i,j+1} - h_{i,j})\frac{\Delta x}{\Delta y} \\ &= T_{i,j,2}(h_{i,j} - h_{i+1,j})\frac{\Delta y}{\Delta x} + T_{i,j-1,1}(h_{i,j} - h_{i,j-1})\frac{\Delta x}{\Delta y} \\ &+ S_{i,j}\,\Delta x\,\Delta y\left(\frac{h_{i,j} - h_{i,j(t-\Delta t)}}{\Delta t}\right) + Q_{i,j} \end{aligned} \tag{10.12}$$

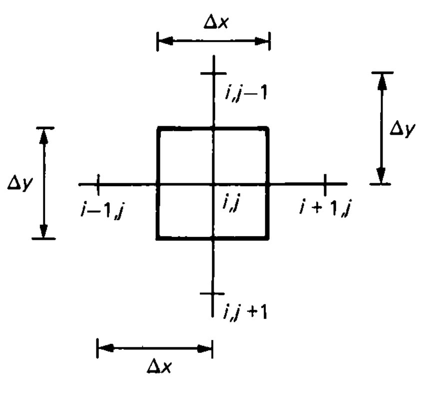


Figure 10.7 Vector volume for digital model flow terms Q_5 and Q_6 (after Prickett and Lonnquist[11])

Dividing both sides of this equation by $\Delta x\,\Delta y$ and rearranging the terms gives

$$T_{i-1,j,2}\left(\frac{h_{i-1,j}-h_{i,j}}{\Delta x^2}\right)+T_{i,j,2}\left(\frac{h_{i+1,j}-h_{i,j}}{\Delta x^2}\right)$$
$$+T_{i,j,1}\left(\frac{h_{i,j+1}-h_{i,j}}{\Delta y^2}\right)+T_{i,j-1,1}\left(\frac{h_{i,j-1}-h_{i,j}}{\Delta y^2}\right)$$
$$=S_{i,j}\left(\frac{h_{i,j}-h_{i,j(t-\Delta t)}}{\Delta t}\right)+\frac{Q_{i,j}}{\Delta x\,\Delta y} \qquad (10.13)$$

This equation represents the finite difference form of the partial differential equation (Equation (10.4)) governing the unsteady two-dimensional flow of water in a heterogeneous artesian aquifer (for a more formal derivation see Pinder and Bredehoeft[2]). If water table conditions are to be modelled, then Equation (10.13) must be modified to allow for the fact that the saturated thickness of the aquifer changes with changing head. With a confined aquifer, of course, the saturated thickness remains constant provided that the piezometric level does not drop below the base of the confining strata. Thus, for unconfined aquifers, the elevation of the base of the aquifer and the level of the water table above an arbitrary datum must be specified for each node. The transmissivity terms can then be modified to take into account the varying saturated thickness. Between nodes i,j and $i+1,j$ (see *Figure 10.6*) the transmissivity is given by

$$T_{i,j,2}=PERM_{i,j,2}\sqrt{(h_{i,j}-BOT_{i,j})(h_{i+1,j}-BOT_{i+1,j})} \qquad (10.14)$$

where $T_{i,j,2}$ is the aquifer transmissivity of the vector volume between nodes i,j and $i+1,j$, $PERM_{i,j,2}$ is the permeability of the aquifer within the vector volume between nodes i,j and $i+1,j$ and $BOT_{i,j}$ is the elevation of the bottom of the aquifer at node i,j above some reference level.

Similarly, the aquifer transmissivity of the vector volume between nodes i,j and $i,j+1$ is given by

$$T_{i,j,1}=PERM_{i,j,1}\sqrt{(h_{i,j}-BOT_{i,j})(h_{i,j+1}-BOT_{i,j+1})} \qquad (10.15)$$

where the terms have more or less the same meaning as for Equation (10.14) except that the vector volume under consideration now lies between nodes i,j and $i,j+1$ (see *Figure 10.6*).

Equations of the type shown in Equation (10.13), modified to allow for unconfined conditions if necessary, can be derived for every node of a model so that a large set of simultaneous algebraic equations is obtained. These equations must be solved for the unknown values of head, $h_{i,j}$.

10.4.2 Solution of the finite difference equations

There are many methods that can be employed to obtain a solution to a set of finite difference equations. Some of the best techniques belong to the group referred to as backward difference methods, so called because the unknown terms of the equation are written as functions of the unknown heads at the present time level, t, and the known value of the heads at the previous time level, $t-\Delta t$, that is, going back in time. This is known as an implicit method. One of the most satisfactory techniques is the Alternating Direction Implicit Procedure with Iteration within each Time-step (ADIP or ADIPIT for short) to predict the head values at the end of the next time interval on the basis of the change in head in the previous time intervals[12,45]. The method is claimed, by Prickett[1], to be fast and accurate with many successful applications.

The ADIP method consists of a number of iterations, with each iteration comprising a series of column calculations followed by a series of row calculations. When the row calculations are complete the absolute value of any changes in head that have occurred since the column calculations were finished is summed and compared with a specified tolerance. If the solution has not converged satisfactorily another iteration is undertaken, but this time the columns and rows are processed in the opposite direction (say decreasing i and j instead of increasing i and j). Hence, successive iterations take place in alternating directions. This method of calculation is more efficient than processing the columns and rows in the same order each time. When convergence has been achieved, the calculated heads are used as the initial conditions for the next time increment. Peaceman and Rachford[43] pointed out that this technique is unconditionally stable regardless of the size of the time increments.

For a given time interval the ADIP method involves reducing a large set of simultaneous equations down to a number of small sets. This is accomplished by solving the node equations of an individual column using Gaussian elimination while all the terms related to the nodes in adjacent columns are held constant. According to Peaceman and Rachford[43], the set of column equations is then implicit (i.e. all heads unknown at present time interval) in the direction along the column and explicit (all heads known at present time interval) in the direction orthogonal to the column alignment. Thus, the solution of the flow equation for a particular node i,j is accomplished by writing the equations (say in order of increasing row number j) so that the head at the node of interest $h_{i,j}$ is a function of the unknown head at the next node in the column $h_{i,j+1}$ and two known adjacent heads along the row, $h_{i-1,j}$ and $h_{i+1,j}$. This procedure is followed for all the nodes in the ith column. The head at the last node of the column (on a boundary of specified type) is known, so by back-substitution the other heads in the column can be calculated in order of decreasing row number. The same procedure is followed for all the other columns. When the column calculations have been completed the row calculations are conducted using the same technique. When these too have been completed the calculated heads are used as the initial conditions for the next iteration, and so on until convergence is achieved.

The solution of the finite difference equations requires that the heads at the nodes lying on a boundary are known. With a recharge or discharge boundary the piezometric head can vary node to node, but the head at any particular node is assumed to be constant. Consequently an infinitely large storage factor (SF) of about 0.1×10^{23} would be assigned to the nodes on the boundary. For a node i,j the storage factor is given by $SF_{i,j} = S_{i,j}\,\Delta x\,\Delta y$, with the same notation as above. By this means the head is maintained at the value specified initially, and so it is always known. A third type of boundary is one along which the head is allowed to change freely in response to recharge and discharge. Such boundaries often define the edges of a model between the recharge and discharge boundaries. The nodes on a 'free' boundary are assigned storage factors which are typical of the aquifer material. However, outside the model area, as defined by the boundaries, all values of piezometric head, transmissivity and storativity are taken as zero by default, so that these nodes are effectively eliminated from the calculations.

The first step in the solution of the equations is to put Equation (10.13) into a modified form that will facilitate node solving by columns and rows. To simplify matters it has been assumed that the finite difference grid consists of squares so that $\Delta x = \Delta y$ (for $\Delta x \neq \Delta y$ see Prickett and Lonnquist[11]). Multiplying both sides of

Equation (10.13) by Δx^2 gives

$$T_{i-1,j,2}(h_{i-1,j}-h_{i,j})+T_{i,j,2}(h_{i+1,j}-h_{i,j})+T_{i,j,1}(h_{i,j+1}-h_{i,j})+T_{i,j-1,1}(h_{i,j-1}-h_{i,j})$$
$$=S_{i,j}\frac{\Delta x^2}{\Delta t}(h_{i,j}-h_{i,j(t-\Delta t)})+Q_{i,j} \tag{10.16}$$

Expanding, reversing the signs and grouping terms of $h_{i,j}$ gives

$$h_{i,j}\left(T_{i-1,j,2}+T_{i,j,2}+T_{i,j,1}+T_{i,j-1,1}+S_{i,j}\frac{\Delta x^2}{\Delta t}\right)$$
$$-T_{i-1,j,2}h_{i-1,j}-T_{i,j,2}h_{i+1,j}-T_{i,j,1}h_{i,j+1}-T_{i,j-1,1}h_{i,j-1}$$
$$=S_{i,j}\frac{\Delta x^2}{\Delta t}h_{i,j(t-\Delta t)}-Q_{i,j} \tag{10.17}$$

This equation can be written in two forms; one for solving the node equations by columns and the other for solving the node equations by rows.

For columns

$$-T_{i,j-1,1}h_{i,j-1}+h_{i,j}\left(T_{i-1,j,2}+T_{i,j,2}+T_{i,j,1}+T_{i,j-1,1}+S_{i,j}\frac{\Delta x^2}{\Delta t}\right)-T_{i,j,1}h_{i,j+1}$$
$$=S_{i,j}\frac{\Delta x^2}{\Delta t}h_{i,j(t-\Delta t)}-Q_{i,j}+T_{i-1,j,2}h_{i-1,j}+T_{i,j,2}h_{i+1,j} \tag{10.18}$$

This equation can be reduced to the general form

$$A_jh_{i,j-1}+B_jh_{i,j}+C_jh_{i,j+1}=D_j \tag{10.19}$$

where the constant terms are

$$A_j=-T_{i,j-1,1}$$

$$B_j=T_{i-1,j,2}+T_{i,j,2}+T_{i,j,1}+T_{i,j-1,1}+S_{i,j}\frac{\Delta x^2}{\Delta t}$$

$$C_j=-T_{i,j,1}$$

$$D_j=S_{i,j}\frac{\Delta x^2}{\Delta t}h_{i,j(t-\Delta t)}-Q_{i,j}+T_{i-1,j,2}h_{i-1,j}+T_{i,j,2}h_{i+1,j}$$

For rows, Equation (10.17) can be rearranged, using a similar procedure, to

$$A_ih_{i-1,j}+B_ih_{i,j}+C_ih_{i+1,j}=D_i \tag{10.20}$$

where the constant terms are

$$A_i=-T_{i-1,j,2}$$

$$B_i=T_{i-1,j,2}+T_{i,j,2}+T_{i,j,1}+T_{i,j-1,1}+S_{i,j}\frac{\Delta x^2}{\Delta t}$$

$$C_i=-T_{i,j,2}$$

$$D_i=S_{i,j}\frac{\Delta x^2}{\Delta t}h_{i,j(t-\Delta t)}-Q_{i,j}+T_{i,j-1,1}h_{i,j-1}+T_{i,j,1}h_{i,j+1}$$

By adopting these generalized expressions (Equations (10.19) and (10.20)) an equation containing three unknown heads is obtained for each node along a column or row (remembering that the heads in the orthogonal direction are known). This forms a tridiagonal matrix which is relatively easy to solve. The solution of a set of column or row head equations is accomplished by Gaussian elimination incorporating arrays, which have been termed M and N below (Peaceman and Rachford[43] called them G and B arrays). This procedure can be illustrated by a simple example.

Example

Figure 10.8 shows 12 nodes arranged in four columns and three rows. The problem is to arrange the equations of flow in a form that will facilitate the calculation of the heads in the jth row by a digital computer and to illustrate the general method of solution.

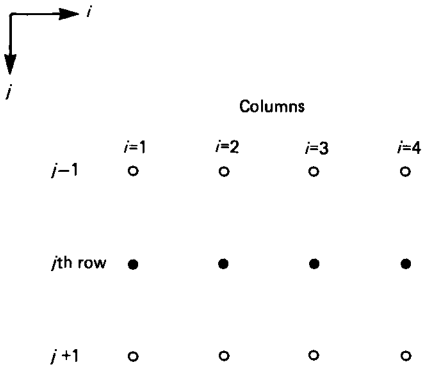


Figure 10.8 A four node row for use in the example solution of the finite difference equations presented in Section 10.4.2 (after Prickett and Lonnquist[11])

Solution

The calculation of the heads at the four nodes along the jth row involves writing the flow equation (in the generalized form of Equation (10.20)) for each of the nodes in order of increasing column number i.

For $i=1$

$$A_1 h_{0,j} + B_1 h_{1,j} + C_1 h_{2,j} = D_1 \tag{10.21}$$

Now $A_1 = 0$ since $h_{0,j}$ does not exist, so that

$$h_{1,j} = \frac{D_1}{B_1} - \left(\frac{C_1}{B_1}\right) h_{2,j}$$

Introducing $M_1 = D_1/B_1$ and $N_1 = C_1/B_1$ gives

$$h_{1,j} = M_1 - N_1 h_{2,j} \tag{10.22}$$

The head at the node of interest $h_{1,j}$ is now a function only of the known parameters M_1 and N_1 and the head at the next node in the row, $h_{2,j}$.

For $i=2$

$$A_2 h_{1,j} + B_2 h_{2,j} + C_2 h_{3,j} = D_2 \tag{10.23}$$

Substituting $h_{1,j}=M_1-N_1h_{2,j}$ (Equation (10.22)) in the above equation and solving for $B_2h_{2,j}$ gives

$$B_2h_{2,j}=D_2-C_2h_{3,j}-A_2(M_1-N_1h_{2,j})$$

whereupon multiplying out and rearrangement of terms gives

$$h_{2,j}=\left(\frac{D_2-A_2M_1}{B_2-A_2N_1}\right)-\left(\frac{C_2}{B_2-A_2N_1}\right)h_{3,j}$$

Thus, the known parameters are

$$M_2=\left(\frac{D_2-A_2M_1}{B_2-A_2N_1}\right) \quad \text{and} \quad N_2=\left(\frac{C_2}{B_2-A_2N_1}\right)$$

So

$$h_{2,j}=M_2-N_2h_{3,j} \tag{10.24}$$

Again the head at the node of interest, $h_{2,j}$, is a function of the known parameters M_2 and N_2 and the head at the next node $h_{3,j}$.

For $i=3$, following exactly the same procedure yields the following equation for the third node of the row, $h_{3,j}$.

$$h_{3,j}=M_3-N_3h_{4,j} \tag{10.25}$$

For $i=4$, the last node in the row

$$A_4h_{3,j}+B_4h_{4,j}+C_4h_{5,j}=D_4 \tag{10.26}$$

Now $C_4=0$ since there is no node at $h_{5,j}$ so

$$B_4h_{4,j}=D_4-A_4h_{3,j}$$

Substitution of Equation (10.25) into the above equation gives

$$h_{4,j}=\left(\frac{D_4-A_4M_3}{B_4-A_4N_3}\right)$$

Thus, the head $h_{4,j}$ is a function of only the known parameter

$$M_4=\left(\frac{D_4-A_4M_3}{B_4-A_4N_3}\right)$$

so that

$$h_{4,j}=M_4 \tag{10.27}$$

Therefore the head at $h_{4,j}$ is known, so substitution of this value back into Equation (10.25) allows the calculation of $h_{3,j}$. Again, by back-substitution of the value of $h_{3,j}$ into Equation (10.24) the term $h_{2,j}$ can be evaluated. Finally, if $h_{2,j}$ is substituted into Equation (10.22) the value of $h_{1,j}$ is obtained. All the heads in the jth row are then known. A similar procedure can be followed for all the other rows and indeed for column calculations.

Inspection of the above equations shows that their general form is

$$M_x=\left(\frac{D_x-A_xM_{x-1}}{B_x-A_xN_{x-1}}\right) \quad \text{and} \quad N_x=\left(\frac{C_x}{B_x-A_xN_{x-1}}\right)$$

where $x=i$ for row calculations, $x=j$ for column calculations, $A_x=0$ for the first node in a row or column and $C_x = 0$ for the last node in a row or column.

Additionally, for *row* calculations the general form is

$$h_{i,j} = M_i - N_i h_{i+1,j} \tag{10.28}$$

while for *column* calculations

$$h_{i,j} = M_j - N_j h_{i,j+1} \tag{10.29}$$

To summarize, the method of solving the finite difference equations in the digital model involves calculating heads along columns or rows by computing the values of M and N for the nodes of a column or row in order of increasing j or i, respectively. When the value of the head at the last column or row has been determined this is back-substituted to find all the other heads in the column or row in order of decreasing j or i, respectively. After completing the calculation of all the heads in a column or row, the next column or row is considered, and so on until they have all been considered. This type of repetitive calculation is ideally suited for a digital computer and is relatively easy to program.

10.4.3 Comments on finite difference models

The techniques described above may not represent the ultimate in digital modelling, but there are several factors in their favour. They are generally reliable, easy to use and there is comprehensive documentation available so that almost anyone can construct a working model of a relatively complex groundwater problem including variable pumping rates, induced infiltration, leaky artesian conditions, and so on[1,11,13,35,46]. In particular, the provision of a program listing and instructions for its application is a very useful feature of the literature. This is not the case with analog or finite element models for which, despite the numerous papers describing such models, there is often no specific guidance concerning the choice of scale factors, resistor and capacitor values, control equipment and the problems of model construction and operation. In fact, once one model has been constructed and operated successfully, the creation of a second model is relatively simple. The difficulty for an inexperienced modeller lies in constructing the first model. Indeed, some of the electronic control equipment that is required to operate an analog model is very complex and frequently beyond the ken of most hydrogeologists. Finite element techniques can also prove quite baffling to the uninitiated. These problems are not so apparent with finite difference models for which there is a greater availability of software and relevant literature.

One disadvantage of digital models is that they do not physically resemble an aquifer or the problem that is being studied, unlike an analog or sand tank model for example. Consequently, they do not greatly assist in the visualization of fluid flow through an aquifer. In addition, they are very specific and only give data, in the form of a printout, on the variables specified in the program. This is not the case with a physical model which may have been constructed for one purpose but which is also capable of yielding additional (possibly unexpected) information about other related problems or phenomena.

With any discretized model the question of scale and grid interval is always important[47]. The accuracy of the data obtained from a model is highly dependent upon the ability of the model to reproduce the aquifer and its associated boundary conditions. Thus, the smaller the grid spacing the more accurate will be the simulation of the aquifer. Similarly, the coarseness of the grid makes it difficult to reproduce complicated groundwater flow patterns, since large variations in drawdown have to be accommodated using perhaps only two or three elements of the grid. In parts of the model it may be almost impossible to achieve an adequate compromise between the transmissivity and storativity values required to satisfy both the rest water level and the pumping conditions. A reduced grid interval could be incorporated in problem areas to reduce the errors incurred through the discretizing process, but this does make the model more complex[11].

A further limitation is due to the fact that flow in a two-dimensional model can only occur in two mutually perpendicular directions, whereas in the prototype three-dimensional flow in any direction is possible. This can be very significant in the vicinity of a discharging well where the flow is radial, although it is possible to compensate for this to some extent[11,13,27].

In digital models, where time is also discretized, the size of the time increment adopted must be considered in order to achieve a valid solution[11,12,48].

One problem which can arise towards the final stages of constructing a model is actually deciding when it is finished. Sometimes it is tempting to go on making changes indefinitely in the belief that the model could always be a little better. It would be very convenient if there were some general rule that could be applied to indicate when a valid and accurate model had been attained, but this is not possible. Every model is unique and the criteria which are used to judge the effectiveness of a model can be widely different. Two important considerations are the accuracy of the data upon which the model was based and the order of accuracy actually required from the model. Some idea regarding the sort of accuracy attainable can be obtained from *Table 10.2.* This may be of value when deciding what constitutes an acceptable error.

Stallman[50] listed the most frequent sources of error encountered in model studies as

1. Truncation and round-off errors.
2. Observational errors made in the collection of model or protype data.

TABLE 10.2. Accuracy of some model investigations

Author	*Model type*	*Quoted accuracy*
Blair[41]	Analogue (steady state). Regional pumping scheme at Otterbourne	Predicted water levels agree to within ± 1.2 m of the average observed water level
Hirsch[39]	Analogue (steady state). Regional groundwater level analysis of Santiago Groundwater Basin	Most model rest water levels within ± 10 m of actual values. (Model constructed to a relatively small scale)
Oakes and Skinner[8]	Digital. Lancashire conjunctive use scheme	Computed and observed levels agree to within ± 2 to ± 3 m. (Pumping test data)
Johnston and Leahy[49]	Digital. Regional analysis, Delaware, USA	70 per cent of computed and observed heads differed from the measured water table values by less than 0.8 m. (Rest water level values only)
Hamill (1977), unpublished	Analogue and digital. Regional analysis of Magnesian Limestone	Rest water levels accurate to ± 1.5 m. Average error in pump test data approximately ± 0.7 m

1. Average error is difficult to assess from published results. The figures given above were quoted by the authors themselves.
2. Since all the models were to different scales and simulated aquifers of varying complexity for different purposes, the figures quoted are not truly comparable.
3. The table is intended only to give an indication of the order of accuracy that can be achieved with a groundwater model.

3. Errors due to the instability of the model.
4. Errors due to representing a continuous aquifer as a group of finite volumes and as a discrete system of 'lumped' elements.
5. Inaccurate proportioning between the aquifer properties and the model values.

With experience, these problems do not detract from the accuracy of digital models[45,51]. Computers have developed so quickly in recent years that it is now possible to adopt smaller grid intervals and time steps without significantly affecting the length of computer time required. It is also possible to run groundwater flow simulation programs on relatively small computers[46].

Guidance regarding the formulation of digital models, the programs required to solve the finite difference equations, and their operation can be found in the literature cited above. Numerous examples and case histories are also given. However, when using a computer model the famous phrase 'garbage in, garbage out' must always be remembered. This certainly applies to groundwater models and any model is only as good as the data on which it is based. Consequently the collection of field data and the subsequent processing of the data, is very important while the type of model used to simulate the prototype sometimes can be of lesser importance.

10.5 The finite element method

The aim of this section is to give a brief outline of the steps involved in the construction of a finite element model. A full explanation of the finite element technique is beyond the scope of this book, since familiarity with both variational calculus and matrix algebra is an essential pre-requisite.

The most common numerical modelling techniques in current use are the finite difference and the finite element methods. Basically, both methods are based on the principle of approximating the set of partial differential equations which govern the behaviour of the system with a set of algebraic equations. The essential difference between the two techniques can be explained as follows. The finite difference method is a 'direct' technique in which the differential operators are approximated by a set of equations called difference formulae, as already outlined in Section 10.4. These formulae are constructed from values of the dependent variable(s) at a given number of known points. The finite element method takes a more 'indirect' route to the solution since it is the variation of the dependent variable or variables that is approximated, instead of the differential operators. The algebraic equations are then formed either by minimizing the appropriate energy functional, which is a characteristic of the given physical system, or, in cases where the definition or identification of the natural energy functional is too difficult, by the piecewise application of a weighted residual method to the differential equations[37,52].

The difference between the finite difference and the finite element approach was considered by Remson *et al.*[34], who stated that the finite difference technique achieves a solution by working directly with the governing differential equations. Finite element techniques, on the other hand, attempt to derive an approximate solution using variational calculus (which is concerned with determining the maxima and minima of functionals). However, it should be appreciated that the finite element and finite difference techniques are related and that one can be regarded as a special case of the other[53,54]. Indeed, if the elements of a finite element model are rectangular then the results will be identical to those obtained from a finite difference model. This being so, why bother with the more complex finite element technique? The answer lies in the greater flexibility of the finite element method which can cope easily with boundary conditions. With finite difference models artificial boundary conditions have to be created. Additionally, it is very easy to adopt a graded finite element mesh so that the more complex areas of the model can be studied in greater detail.

Although the mathematical basis of finite element techniques is quite complex, it is possible to approach the construction of a finite element model in a step by step manner using widely available computer programs. Often all the user of the program is required to do is input the appropriate data.

10.5.1 The basic concept of the finite element method

The basic concept of the finite element method is to 'discretize' a particular problem in a 'continuum' and to obtain an approximate solution to the problem at these discrete points. Discretization is achieved by dividing the continuum (region of interest) into a number of distinct non-overlapping pieces known as elements, hence the name finite elements. The dividing lines between the elements are similar to those which make up a finite difference mesh, but in this case the elements may be almost any shape and the nodes may be located at any point along the periphery of an element, or even within it. The variables are identified at the nodal points. The approximation or discrete model consists of a set of values of a given function at a finite number of nodes which make up the elements, together with the piecewise approximation of the function over the region within the elements. These approximations are uniquely defined by the discrete values of the functions at the nodes of each individual element. It is important to note that the given function is approximated 'locally' over each element by continuous functions which are uniquely defined in terms of the values of the function (and often its derivatives) at the nodes of each element. Thus, each element can be considered as a separate or 'local' case, and each element is independent of all other elements. It is only when each element is assembled into a complete system that they become interdependent.

The technique used to formulate the local problem within an element varies. The most common technique is the method of weighted residuals, and in particular the Galerkin method. This method can be illustrated by considering a simple one-dimensional problem in which the following differential equation applies, subject to some boundary condition over the one-dimensional domain Δ.

$$k\frac{\partial^2 h}{\partial x^2}+q=0 \tag{10.30}$$

where both k and q are constants representing the coefficient of permeability and any inflow/outflow of water. This equation describes the variation of groundwater level, h, with distance in the x direction. Applying the method of weighted residuals, the exact solution $h(x)$ is approximated by $h^*(x)$ hence

$$h(x)\cong h^*(x) \tag{10.31}$$

Now $h^*(x)$ can be written as a number of trial functions, ϕ_i, continuous over the domain, which satisfy the boundary conditions and a set of unknown parameters C_i.

$$h^*(x)=\sum_{i=1}^{T}\phi_i(x)C_i \tag{10.32}$$

where T is the total number of elements in the domain. However, since $h^*(x)$ is not the *exact* solution

$$k\frac{\partial^2 h^*(x)}{\partial x^2}+q\neq 0=R(x) \tag{10.33}$$

where $R(x)$ is the error incurred in the approximation of the function. For an exact solution $R(x)=0$. Nevertheless, the optimum approximate solution is obtained by minimizing the value of $R(x)$ over the whole domain. The Galerkin method of weighted residuals specifies that the average of this weighted error, or residual, is equal to zero when integrated over the domain of the equation. In addition, the weighting factors are taken as the trial functions $\phi_i(x)$. Thus

$$\int \phi_i(x)R(x)\,\mathrm{d}x=0 \tag{10.34}$$

Substituting Equation (10.33) into Equation (10.34) gives

$$\int\left\{k\frac{\partial^2 h^*(x)}{\partial x^2}+q\right\}\phi_i(x)\,\mathrm{d}x=0 \tag{10.35}$$

The domain Δ is divided into a set of elements $[X_1, X_2,\ldots,X_T]$ where T is the total number of elements in the domain. Discretizing for each element, for example element i, gives

$$\int_{X_i}^{X_{i+1}} \phi_i R\,\mathrm{d}x=\int_{X_i}^{X_{i+1}}\left\{k\frac{\partial^2 h^*}{\partial x^2}+q\right\}\phi_i\,\mathrm{d}x=0 \tag{10.36}$$

The trial functions, ϕ_i, must be continuous over the domain and satisfy the boundary conditions imposed on $h(x)$. Normally, polynomials of low degree are chosen to represent the trial functions, so that $h^*(x)$ can be written as

$$h^*(x)=a_0+a_1x+a_2x^2+a_3x^3+\cdots+a_ix^i \tag{10.37}$$

However, if linear trial functions are selected these have the form

$$h^*(x)=a_0+a_1x \tag{10.38}$$

where $a_0, a_1,\ldots,a_i$ are unknown constants. These constants can be determined using the known values of $h(x)$ at the nodes of the elements. For instance, at node x_i of an element $h_i=h(x_i)$. Assuming that for the ith element $h^*(x)$ is linear (Equation (10.38)), then

$$h_i=a_0+a_1x_i \tag{10.39}$$

$$h_{i+1}=a_0+a_1x_{i+1} \tag{10.40}$$

and so on.

Combining Equations (10.38), (10.39) and (10.40) gives

$$h^*(x)=\left\{\frac{x-x_{i+1}}{x_i-x_{i+1}}\right\}h_i+\left\{\frac{x-x_i}{x_{i+1}-x_i}\right\}h_{i+1} \tag{10.41}$$

for instance, if $i=2$,

$$h^*(x)=\left[\frac{x-x_3}{x_2-x_3}\right]h_2+\left[\frac{x-x_2}{x_3-x_2}\right]h_3 \tag{10.42}$$

Combining Equations (10.32) and (10.41)

$$h^*(x)=\left\{\frac{x-x_{i+1}}{x_i-x_{i+1}}\right\}h_i+\left\{\frac{x-x_i}{x_{i+1}-x_i}\right\}h_{i+1}=\sum_{i=1}^{T}\phi_i(x)C_i \tag{10.43}$$

Thus $\phi_i(x)$ can now be written in terms of $x_i, x_{i+1},\ldots,$ and C_i as h_i. Usually $\phi_i(x)$ is denoted as $N_i(x)$ and termed the shape or interpolation function. For higher order approximations of $h(x)$ the number of shape functions will increase: a quadratic variation in $h^*(x)$ leads to three shape functions, N_i. This will necessitate the inversion of a 3 by 3 matrix of coefficients, typically by employing Lagrange interpolation functions (a method explained in most numerical analysis textbooks). This results in an algebraic system of equations. In matrix form Equation (10.42) ($i=2$) becomes

$$h^*=[N_1, N_2]\begin{bmatrix}h_2\\h_3\end{bmatrix} \tag{10.44}$$

and Equation (10.36) becomes

$$\int_{x_2}^{x_3}\begin{bmatrix}N_1\\N_2\end{bmatrix}k\frac{\partial^2}{\partial x^2}[N_1, N_2]\begin{bmatrix}h_2\\h_3\end{bmatrix}\mathrm{d}x+\int_{x_2}^{x_3}\begin{bmatrix}N_1\\N_2\end{bmatrix}q\,\mathrm{d}x=0 \tag{10.45}$$

which involves a double integration of the shape functions N_i. Applying Green's theorem[37] gives

$$\int_{x_2}^{x_3}N_i\frac{\partial^2 N_j}{\partial x^2}\,\mathrm{d}x=-\int_{x_2}^{x_3}\frac{\partial N_i}{\partial x}\frac{\partial N_j}{\partial x}\,\mathrm{d}x+\left[N_i\frac{\partial N_j}{\partial x}\right]_{x_2}^{x_3} \tag{10.46}$$

The advantage of Green's theorem is that it implicitly imposes the natural boundary conditions, while for even order operators the resulting matrices are symmetrical. The terms representing inter-elemental boundary conditions generally cancel, as shown below by considering the boundaries of elements 2 and 3.

$$N_i\frac{\partial N_j}{\partial x}\bigg|_{x_4}-N_i\frac{\partial N_j}{\partial x}\bigg|_{x_3}+N_i\frac{\partial N_j}{\partial x}\bigg|_{x_3}-N_i\frac{\partial N_j}{\partial x}\bigg|_{x_2} \tag{10.47}$$

This leaves only the boundary terms on the periphery of the domain. Hence for element 2, Equation (10.45) becomes

$$k\int_{x_2}^{x_3}\begin{bmatrix}\dfrac{\partial N_1}{\partial x}\dfrac{\partial N_1}{\partial x} & \dfrac{\partial N_1}{\partial x}\dfrac{\partial N_2}{\partial x}\\ \dfrac{\partial N_2}{\partial x}\dfrac{\partial N_1}{\partial x} & \dfrac{\partial N_2}{\partial x}\dfrac{\partial N_2}{\partial x}\end{bmatrix}\mathrm{d}x\begin{bmatrix}h_2\\h_3\end{bmatrix}-q\int_{x_2}^{x_3}\begin{bmatrix}N_1\\N_2\end{bmatrix}\mathrm{d}x=0 \tag{10.48}$$

If $L=x_3-x_2$ then

$$k\begin{bmatrix}-1/L & -1/L\\-1/L & 1/L\end{bmatrix}\begin{bmatrix}h_2\\h_3\end{bmatrix}-q\begin{bmatrix}L/2\\L/2\end{bmatrix}=\begin{bmatrix}0\\0\end{bmatrix} \tag{10.49}$$

$$\underset{\rightarrow}{K^e}\quad\underset{\rightarrow}{h}\quad-\quad\underset{\rightarrow}{F^e}\quad=0 \tag{10.50}$$

where K^e is the 'element stiffness matrix' and F^e is the 'element forcing matrix'.

Equation (10.50) can be solved relatively easily using published computer programs. The difficulty lies generally in the correct assembly of the matrices. From Equation (10.49) it can be seen that these matrices are formed from the coefficient of permeability,

k, the lengths of the finite elements, L, and the inflow/outflow terms, q. The elevation of the groundwater surface, h, is, of course, the unknown variable for which a solution is being sought.

10.5.2 Finite elements applied to the two-dimensional Laplace equation

The Laplace equation in two dimensions may be written as

$$\frac{\partial^2 h}{\partial x^2}+\frac{\partial^2 h}{\partial y^2}+q=0 \tag{10.51}$$

h is now a function of two variables, x and y, and can be approximated by the bilinear expansion

$$h(x, y)=h^*(x, y)=a_0+a_1x+a_2y+a_3xy \tag{10.52}$$

Assuming a rectangular element of sides a and b with the nodal values shown below,

$$\begin{aligned} &\text{Node 1} \quad h(0,0)=h_1\\ &\text{Node 2} \quad h(0,a)=h_2\\ &\text{Node 3} \quad h(a,b)=h_3\\ &\text{Node 4} \quad h(0,b)=h_4 \end{aligned} \tag{10.53}$$

so that when these values are substituted into Equation (10.52)

$$h^*(x, y)=[N_1 \quad N_2 \quad N_3 \quad N_4]\begin{bmatrix} h_1\\ h_2\\ h_3\\ h_4 \end{bmatrix} \tag{10.54}$$

In this instance the interpolation functions N are given by

$$\begin{aligned} N_1&=(1-x/a)(1-y/b)\\ N_2&=(1-x/a)(y/b)\\ N_3&=(x/a)(y/b)\\ N_4&=(x/a)(1-y/b) \end{aligned} \tag{10.55}$$

Applying the Galerkin weighted residual method in two dimensions

$$=\int_0^a\int_0^b N_i\left\{\frac{\partial^2 h^*}{\partial x^2}+\frac{\partial^2 h^*}{\partial y^2}+q\right\}\mathrm{d}x\,\mathrm{d}y=0 \tag{10.56}$$

$$=\int_{\Gamma e} N_i\frac{\partial h^*}{\partial x}\,\mathrm{d}y-\int_0^a\int_0^b\frac{\partial N_i}{\partial x}\frac{\partial h^*}{\partial x}\,\mathrm{d}x\,\mathrm{d}y+\int_{\Gamma e} N_i\frac{\partial h^*}{\partial y}\,\mathrm{d}x-\int_0^a\int_0^b\frac{\partial N_i}{\partial y}\frac{\partial h^*}{\partial y}\,\mathrm{d}x\,\mathrm{d}y$$
$$+\int_0^a\int_0^b N_i q\,\mathrm{d}x\,\mathrm{d}y=0 \tag{10.57}$$

$$=\int_{\Gamma e} N_i\frac{\partial h^*}{\partial x}\eta_x\,\mathrm{d}l+\int_{\Gamma e} N_i\frac{\partial h^*}{\partial y}\eta_y\,\mathrm{d}l+\int_0^a\int_0^b\left\{\frac{-\partial N_i}{\partial x}\frac{\partial h^*}{\partial x}-\frac{\partial N_i}{\partial y}\frac{\partial h^*}{\partial y}+qN_i\right\}\mathrm{d}x\,\mathrm{d}y=0 \tag{10.58}$$

$$= -\int_0^a \int_0^b \left\{ \frac{\partial N_i}{\partial x} \frac{\partial h^*}{\partial x} + \frac{\partial N_i}{\partial y} \frac{\partial h^*}{\partial y} \right\} \mathrm{d}x\,\mathrm{d}y + \int_0^a \int_0^b N_i q \,\mathrm{d}x\,\mathrm{d}y$$
$$+ \oint_{\Gamma e} N_i \left\{ \frac{\partial h^*}{\partial x} \eta_x + \frac{\partial h^*}{\partial y} \eta_y \right\} \mathrm{d}l = 0 \tag{10.59}$$

where η_x and η_y are the x and y components of the unit normal to the boundary of the element Γe, $\oint$ is Green's integral and dl is the elemental length along Γe. As in the previous one-dimensional example, the integrals over the inter-elemental boundaries will cancel leaving Equation (10.59) in the form

$$\underset{\rightarrow}{K^e} \; \underset{\rightarrow}{h} - \underset{\rightarrow}{Q^e} = 0 \tag{10.60}$$

where

$$K_{ij}^e = \int_0^a \int_0^b \left\{ \frac{\partial N_i}{\partial x} \frac{\partial N_j}{\partial x} + \frac{\partial N_i}{\partial y} \frac{\partial N_j}{\partial y} \right\} \mathrm{d}x\,\mathrm{d}y \tag{10.61}$$

and

$$Q_i^e = \int_0^a \int_0^b q N_i \,\mathrm{d}x\,\mathrm{d}y \tag{10.62}$$

Q^e is the 'element flow matrix' representing the flow within the element.

For the case of steady groundwater flow in an anisotropic aquifer with a coefficient of permeability k_x and k_y in the x and y directions, respectively

$$k_x \frac{\partial^2 h}{\partial x^2} + k_y \frac{\partial^2 h}{\partial y^2} + q = 0 \tag{10.63}$$

where q represents any inflow to the aquifer and h denotes the fluid potential. The Galerkin process leads to the equation

$$\iint \left[k_x \frac{\partial N_i}{\partial x} \frac{\partial N_j}{\partial x} + k_y \frac{\partial N_i}{\partial y} \frac{\partial N_j}{\partial y} \right] \mathrm{d}x\,\mathrm{d}y\, h = \iint q N_i \,\mathrm{d}x\,\mathrm{d}y$$

giving as before $K^e h = Q^e$.

10.5.3 Comments on finite element models

Zienkiewicz *et al.*[36] described the solution of two-dimensional anisotropic seepage using finite elements, while Neuman and Witherspoon[55] described a finite element method of analysing steady seepage with a free surface. In 1970 the introduction of iso-parametric elements allowed a more speedy and efficient solution to many problems. Later work, by for instance Aalto[56], made greater use of the finite element technique by extending it to solve more complicated boundary conditions.

Pinder and Gray[53] discussed the relative merits of the finite difference and finite element methods and showed that the finite element approximation of the convective diffusion transport equation can be expressed as established finite difference equations. It was shown also that while finite difference methods approximate the differential equation at a point, the finite element method can be interpreted as an approximation of the integrated form of the differential equation. Aberra[38] compared the two methods of analysis as applied to square aquifers containing pumped wells. Using discrete time steps, good agreement was found between the two methods when the length of the steps was small, but for larger steps the discrepancies in the analytical methods increased, as expected.

10.6 Other considerations and useful bibliography

There are several papers which have not previously been referred to in the text that might prove useful to anyone undertaking the construction of a groundwater model. These are mentioned briefly below. Clarke[57] reviewed some of the mathematical models used in hydrology and offered observations on their calibration and use. Some of the terms which are frequently used (and misused) in connection with mathematical models are defined.

Cooley and Sinclair[58] considered the uniqueness of a model of steady state groundwater flow and concluded that there was no unique set of parameters which produced a distinct minimum error (defined as the sums of the squares of the differences between computed and observed hydraulic heads). Some parameters could be changed over a large range without significantly affecting the accuracy of the model. It may sometimes be possible to achieve an 'accurate' model despite the fact that some of the parameters are incorrect or at variance with the field data. It was argued that field data collection and the use of reliable parameter estimation methods are complimentary. In fact, the latter could indicate where additional field data should be collected. Techniques which can be used for the purpose of parameter estimation have been described by Stallman[59], Kleinecke[60], Yeh and Tauxe[61], Sagar *et al.*[62], Garay *et al.*[63] and Birtles and Morel[64], among others.

Murray and Johnson[65] described the mathematical modelling of a region of unconfined groundwater flow. Frequently, unconfined aquifers are difficult to model since interconnection with surface water sources is possible. Indeed, at times it may be desirable to combine surface and groundwater resources and use the two conjunctively. Downing *et al.*[66] considered the regional development of groundwater resources in combination with surface water and discussed the value of mathematical models in the design and management stages of such schemes. Miles and Rushton[67] gave details of a model which represented the flow of both groundwater and surface water in a catchment area in central England. The model was used to investigate the long-term water resources of the region.

The Thames Groundwater Scheme provides a well-known example of a situation where both surface and groundwater flows are important. The aim of the scheme is to regulate the River Thames using water from the Chalk. In order to gain experience of this type of flow augmentation, the Lambourne Valley Pilot Scheme was undertaken between 1967 and 1969 and a model study was commissioned. The first model constructed was of the resistance–capacitance electrical analog variety, although this has now been superseded by digital computer models[68,69,70]. The Thames Groundwater Scheme provides a very clear and interesting illustration of the effects and significance of induced infiltration.

The modelling of confined aquifers is relatively straightforward, except when the piezometric head is small. Under these circumstances the aquifer may become unconfined if there is a reduction in water level. The problem of aquifers changing between the confined and unconfined state has been considered by Rushton[71], Rushton and Tomlinson[72], and Birtles and Reeves[73]. Additionally, Birtles and Wilkinson[74] described the mathematical simulation of groundwater abstraction from a confined aquifer for river regulation purposes. They concluded that high net gains could only be sustained if unconfined storage is developed around the abstraction site. An aquifer with a large response time is also desirable.

A rather unusual model was developed by Corapcioglu and Brutsaert[75]. This was a viscoelastic aquifer model that was designed to analyse and predict the subsidence due to pumping in the San Joaquin Valley in California. However, the model was unable to represent satisfactorily the mechanical properties of the aquifer as related to surface subsidence. This is one of the few failures to be found in the literature. Most models appear to perform well and are of great value in the investigation and management of water resources. Many examples of such models can be found by referring to the appropriate journals. Additionally, Remson *et al.*[34] included a useful appendix which lists published applications of the numerical techniques used in subsurface hydrology.

10.7 References

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Appendix I

Values of the Correlation Coefficient for Different Levels of Significance

For a total correlation, v is 2 less than the number of pairs in the sample; for a partial correlation, the number of eliminated variates should also be subtracted. The probabilities at the head of the columns refer to the two-tail test of significance and give the chance that $|r|$ will be greater than the tabulated values. For a single-tail test the probabilities should be halved.

	.1	.05	.02	.01	.001
$v=1$	.98769	.99692	.999507	.999877	.9999988
2	.90000	.95000	.98000	.990000	.99900
3	.8054	.8783	.93433	.95873	.99116
4	.7293	.8114	.8822	.91720	.97406
5	.6694	.7545	.8329	.8745	.95074
6	.6215	.7067	.7887	.8343	.92493
7	.5822	.6664	.7498	.7977	.8982
8	.5494	.6319	.7155	.7646	.8721
9	.5214	.6021	.6851	.7348	.8471
10	.4973	.5760	.6581	.7079	.8233
11	.4762	.5529	.6339	.6835	.8010
12	.4575	.5324	.6120	.6614	.7800
13	.4409	.5139	.5923	.6411	.7603
14	.4259	.4973	.5742	.6226	.7420
15	.4124	.4821	.5577	.6055	.7246
16	.4000	.4683	.5425	.5897	.7084
17	.3887	.4555	.5285	.5751	.6932
18	.3783	.4438	.5155	.5614	.6787
19	.3687	.4329	.5034	.5487	.6652
20	.3598	.4227	.4921	.5368	.6524
25	.3233	.3809	.4451	.4869	.5974
30	.2960	.3494	.4093	.4487	.5541
35	.2746	.3246	.3810	.4182	.5189
40	.2573	.3044	.3578	.3932	.4896
45	.2428	.2875	.3384	.3721	.4648
50	.2306	.2732	.3218	.3541	.4433
60	.2108	.2500	.2948	.3248	.4078
70	.1954	.2319	.2737	.3017	.3799
80	.1829	.2172	.2565	.2830	.3568
90	.1726	.2050	.2422	.2673	.3375
100	.1638	.1946	.2301	.2540	.3211

From Fisher and Yates: Statistical Tables for Biological, Agricultural and Medical Research. Oliver & Boyd, Edinburgh.

Appendix II

Groundwater treatment processes and cost estimates

Groundwater treatment commonly consists of nothing more than chlorination. Chlorination aims to eliminate completely, or reduce to negligible concentrations, bacteria and other organisms. Additionally, chlorination safeguards the water while it is in the distribution system by imparting a residual resistance to reinfection. Ozonation can be an alternative to chlorination, although not widely used in the United Kingdom. Sometimes other processes such as softening, the removal of manganese, iron, and nitrate, or activated carbon treatment to control tastes and odours, may also be employed.

The costs quoted below are estimates, and generally only include the cost of the chemical plant required. Other expenses which may not have been considered include civil and mechanical work, operating, maintenance and contract costs. These may significantly increase the cost of the entire works so that the total cost is substantially greater than the sum of the components. Some examples of total cost estimates are given by Anon, 1977[1].

The cost estimates given below are for individual processes[1]. These prices relate to 1976, Quarter 3 (1976, Q3), and should be updated by using a suitable index (see Chapter 1). To obtain 1985 cost equivalents the 1976 Q3 prices should be multiplied by a factor of about 2.5.

Chlorination

A typical chlorine dose for groundwater is 0.2 mg/l. The cost of the equipment required is given by

$$\text{Cost} = 45.1 \times \text{CHLCAP}^{0.46}$$

where Cost is the chlorination equipment cost (£000s) and CHLCAP is the total chlorine capacity (000s kg/d). Bulk storage of chlorine can increase the cost by around 50 per cent. Since groundwaters are generally of relatively good quality, chlorination costs could be expected to be about or below average, perhaps between £4000 and £30 000.

Ozonation

Ozone treatment is not used in the United Kingdom, so cost estimates are difficult to establish. Nevertheless, for the control of tastes and colour it has been estimated that the cost of the plant required is likely to be between £2500 and £4700 per thousand m^3/d according to dose rate and plant capacity. For colour reduction requiring higher dose rates (in excess of 7 kg/hr) the plant costs may be between £2000 and £8000 per thousand m^3/d of throughput. Civil work may double the costs given above.

Aeration and desorption

Aeration can be used for the oxidation of iron and manganese or for the desorption of carbon dioxide, hydrogen sulphide or ammonia. Costs vary appreciably according to the method used and the degree of aeration required. However, as an approximate guide the 1976 Q3 capital costs lie in the range £2000 to £12 000 per thousand m^3/d of capacity.

Activated carbon treatment

Activated carbon treatment is used for the control of tastes and odours, or for the reduction of the organic content. This process consists of adding powdered or granular carbon during chemical clarification. Cost is difficult to assess, although Burley and Short[2] have made some predictions.

Softening

There are two principal methods of softening groundwater. The first is precipitation softening, and the second is ion exchange. Precipitation softening requires the use of a sedimentation tank, so cost estimates must allow for this and the removal of the sludge solids. The cost depends upon the quality of the water to be treated and the type of installation adopted. It is not possible to quote a single cost function for this process, but a proprietary tank system cost around £400 000 in 1976 (range approximately £0.068M to £1.12M).

The cost of ion exchange treatment depends upon the system adopted and the degree of hardness reduction required. In 1976 the range of plant costs was from about £7000 to £20 000 per thousand m^3/d of output after blending with unsoftened water.

Ammonia and nitrate removal

Groundwaters are generally free from ammonia, but high nitrate concentrations are becoming relatively common (see Section 8.7). In 1976 there were no established operational plants in the United Kingdom for the removal of nitrate. Ion exchange treatment can be used, but the capital and operating costs may be high[3]. The approximate operating cost of removing 10 to 12 mg/l of nitrogen from 45 000 m^3/d of groundwater are £18 per thousand m^3 for low sulphate waters (20 mg/l SO_4), and £36 per thousand m^3 for high sulphate waters (120 mg/l SO_4). However, the cost could be doubled by chemical handling charges, while civil and mechanical expenses are additional. Similarly, the operating cost of removing 10 mg/l of nitrogen from water containing 10 mg/l of dissolved oxygen using a biological suspended growth technique is around £2.5 to £3.0 per thousand m^3 (1976, Q3). Although the removal of nitrate from groundwaters is expensive at present, it is possible that the cost may fall when more operational plants are established and when the results of recent research have been assimilated.

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