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Chapter 16
Groundwater flow

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This chapter describes the fundamentals of steady-state groundwater flow in a saturated soil, based on Darcy's Law and the concept of soil permeability or hydraulic conductivity. Simple flow regimes including radial flow to single wells are described, and the use of plane flownets in plan and cross-section to calculate flowrates and pore water pressures is discussed. Flow in strata of anisotropic permeability and across a boundary between two soils having different permeabilities is briefly considered. The importance of controlling pore water pressures in the vicinity of an excavation is illustrated with reference to a case study, and some common methods of groundwater control are described. Finally, the concepts of transient flow and consolidation are summarised.

CONTENTS

16.1 Darcy's Law

For almost all practical purposes, the flow of groundwater through a soil is governed by Darcy's Law,

$$q = Aki \tag{16.1}$$

where q is the volumetric flowrate in m^3/s; A is the gross area through which flow occurs (m^2); k is a parameter known as the *hydraulic conductivity* or the *permeability* (m/s), which is a measure of the ease with which water can flow through the soil; and i is the hydraulic gradient in the direction of flow being considered, which is defined as:

$$i = -\frac{\mathrm{d}h}{\mathrm{d}x}. \tag{16.2}$$

i.e. the rate of loss of total head, h (m), with distance x (m) along the direction of flow. The hydraulic gradient is therefore dimensionless. The head used is always the total head, measured above an arbitrary datum which, once chosen, remains fixed. In **Figure 16.1**, the total head for the point at the centre of the left-hand cross-section, A, above the datum indicated, is h_A. This is the level above datum to which water will rise in a standpipe piezometer with its tip at the point of interest A. Other types of head, which are not relevant for the purposes of Darcy's Law, are:

- *the elevational head*, i.e. the elevation of the point above the chosen datum, z_A;
- *the pressure head*, which is defined as the pore water pressure at the point of measurement divided by the unit weight of water, and is indicated by the height to which water in the piezometer rises above the measurement point – in the case of **Figure 16.1**, this would be $h_A - z_A$;
- *the velocity head*, defined as $v^2/2g$ where v is the velocity of groundwater flow, which is usually negligibly small in groundwater problems, and g is acceleration due to gravity.

Darcy's Law is illustrated schematically in **Figure 16.1**. This figure also shows the basis of the simplest form of hydraulic conductivity measurement, which involves subjecting a specimen of known cross-sectional area A to a known head drop h (from which the hydraulic gradient h/x may be calculated), and measuring the corresponding volumetric flowrate.

The main subtleties or potential pitfalls of Darcy's Law are that:

1. It applies only if the flow is laminar. However, unless the ground is very highly permeable, this will almost always be the case in groundwater applications.
2. The flowrate q may be divided by the cross-sectional area A to obtain an apparent, superficial or Darcy flow velocity $v_D = q/A$. This is not, however, the true average fluid flow velocity, because in reality flow takes place only through the soil pores. The pores occupy a proportion of the total area of nA, where n is the soil porosity defined as the volume of pores ÷

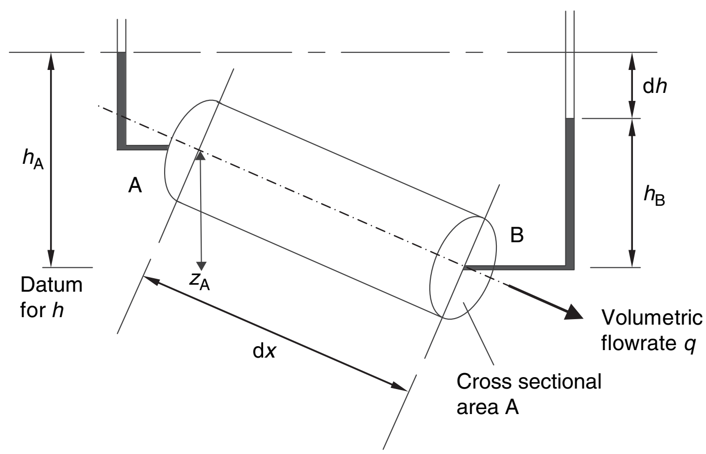

Figure 16.1 Schematic representation of hydraulic gradient and Darcy's Law

the total volume, or $n = e/(1 + e)$ where e is the void ratio. Thus the true average fluid flow velocity v_{true} is $(1 + e)/e$ times the superficial or Darcy flow velocity v_D,

$$v_{true} = \frac{(1+e)}{e} v_D . \tag{16.3}$$

3. The head loss h used in calculating the hydraulic gradient is in terms of total head, determined as the rise in water level above an arbitrary datum in an imaginary standpipe driven into the soil, with its tip at the point at which the head is to be measured.
4. The hydraulic conductivity depends on both the nature of the soil and the physical properties of the permeant fluid, in this case water.
5. Even in a uniform soil, the hydraulic conductivity depends hugely on the particle size ($k \propto D_{10}^2$) and to a lesser extent on the void ratio. In the ground it can vary massively from point to point because of natural inhomogeneities, and be strongly anisotropic (with horizontal k_h being much greater than vertical k_v) owing to soil fabric and structure. In finer grained and compressible materials such as clays and landfilled wastes, the hydraulic conductivity is especially likely to vary – often significantly – with stress and stress history, and hence with depth (Chapter 15 *Groundwater profiles and effective stresses*). In construction, what is usually sought is a reliable estimate of the effective bulk hydraulic conductivity of the volume of ground contributing to the flow of interest. This can be very difficult to estimate, even to within an order of magnitude, especially from small scale or laboratory element tests.
6. Darcy's Law forms the basis of nearly all groundwater flow calculations, whether they are carried out by hand or numerically. Darcy's Law can generally be applied to fissure flow in rocks (e.g. chalk) – in which case the hydraulic conductivity used is that representative of a volume of rock whose linear dimensions are large in relation to the typical fissure spacing.

Items (4), (5) and (6) are discussed in section 16.2.

16.2 Hydraulic conductivity (permeability)

The hydraulic conductivity, k, used in Darcy's Law depends on both the soil and the properties of the permeant fluid – in this case, water – according to the Equation (16.4):

$$k = \frac{K\gamma_f}{\eta_f} \tag{16.4}$$

where K (m^2) is the intrinsic permeability of the soil matrix, γ_f is the unit weight of the permeant fluid (kN/m^3), and η_f its dynamic viscosity (kNs/m^2). In geotechnical engineering, the hydraulic conductivity is often known simply as the permeability.

The practical implications of this are for the most part fairly limited. Obviously, the permeability will change if the permeant fluid is not water, but the dependence of the dynamic viscosity on temperature could also be significant in some cases (η_f reduces, and hence k increases by a factor of about 2 over a temperature rise from 20 to 60°C).

The hydraulic conductivity of a granular material is governed by the smallest 10% of the particles, and varies roughly with the square of the D_{10} particle size. Hazen's empirical formula:

$$k(\mathrm{m/s}) \approx 0.01[D_{10}(\mathrm{mm})]^2 \tag{16.5}$$

is often used to obtain a rough estimate of the hydraulic conductivity, even though it was originally intended for use only with clean filter sands. Thus the potential range of hydraulic conductivity even for uniform soils is very wide indeed, varying over perhaps ten orders of magnitude from 1 m/s for a clean, open gravel to 10^{-10} m/s for an unfissured clay (**Table 16.1**). This degree of variation is far greater than that of almost any other engineering parameter for natural materials – for example, the shear strength of a soft clay is a factor of about 10^5 less than that of high tensile steel.

Natural inhomogeneities such as lenses or zones of different materials (e.g. an infilled river channel) will cause the hydraulic conductivity in the ground to vary from point to point. Even in uniform soils, the *in situ* hydraulic conductivity may be anisotropic owing to the natural fabric or structure of the soil. In clays with no obvious layered structure, the hydraulic conductivity in the horizontal direction might be expected to be up to around ten times that in the vertical. If the soil displays an obvious layering, e.g. interlaminated bands of clay/silt and silt/sand, the ratio of horizontal to vertical hydraulic conductivity could easily be in the order of 10^2 or 10^3.

Some typical values of hydraulic conductivity are given in **Table 16.1**.

Indicative soil type	Degree of permeability	Typical hydraulic conductivity range m/s
Clean gravels	High	$>1\times10^{-3}$
Sand and gravel mixtures	Medium	1×10^{-3} to 1×10^{-5}
Very fine sands, silty sands	Low	1×10^{-4} to 1×10^{-7}
Silt and interlaminated silt/sand/clays	Very low	1×10^{-6} to 1×10^{-9}
Intact clays	Practically impermeable	$<1\times10^{-9}$

Table 16.1 Range of hydraulic conductivity for different soil and ground types
Data taken from Preene *et al.* (2000) (CIRIA C515)

In ground engineering, it is usual to use a fairly simple flow model based on a single value of hydraulic conductivity, at least in an initial calculation. It therefore becomes important that the value used is reasonably representative of the whole volume within which flow is taking place. Large scale, *in situ* tests that involve pumping on one or more wells, with an array of piezometers to monitor the resulting drawdown, are likely to give the most realistic estimates. Building a reliable picture from smaller scale tests can be difficult, not only because of the small volume of soil involved in the flow, but also because of the potential for disturbance during sample retrieval (for laboratory tests) or borehole installation (for rising or falling head tests in single boreholes). A review of the various methods of determining hydraulic conductivity for the purpose of large-scale groundwater control system design is given by Preene *et al.* (2000).

16.3 Calculation of simple flow regimes

Hand calculations based on Darcy's Law, combined with the condition of continuity of flow (i.e. the volumetric flow of water into an element of soil must be equal to the volumetric flow of water out, over a given increment of time, if the soil element is not changing in volume), can be carried out for simple flow regimes that are essentially two-dimensional in nature. Common situations include the following:

(i) Radial flow to a fully penetrating well in a confined aquifer (illustrated, with the terms defined, in **Figure 16.2**):

$$q = \frac{2\pi Dk\left(H-h\right)}{\ln\left(R_{o}/r_{w}\right)}. \tag{16.6}$$

(ii) Radial flow to a fully penetrating well in an unconfined aquifer (illustrated in **Figure 16.3**):

$$q = \frac{\pi k\left(H^2-h^2\right)}{\ln\left(R_{o}/r_{w}\right)}. \tag{16.7}$$

Full details of the analyses leading to Equations (16.6) and (16.7) are given in Powrie (2004).

A large excavation of plan dimensions a (length) × b (breadth) surrounded by a ring of dewatering wells can be analysed approximately as a single well of equivalent radius r_e, provided that the aspect ratio of the excavation a/b is between 1 and 5, and the distance from the edge of the excavation to the recharge boundary, L_o, is greater than $3a$. The equivalent radius r_e is given by $(a + b)/\pi$, and the equivalent radius of influence R_o is $(L_o + a/2) \approx L_o$. Further details are given by Preene and Powrie (1992).

(iii) Plane flow: for excavations with close recharge boundaries, flowrates can be calculated by means of a flownet sketched within one half of the cross-sectional plane. The plane flownet is used to calculate the flowrate per metre run: this is then multiplied by the perimeter of the excavation to give the overall flowrate. If the excavation is long ($a/b > 10$) with close recharge boundaries ($L_o/a < 0.1$), plane flow to the long sides will dominate and end effects may be neglected. If the excavation is rectangular ($1 < a/b < 5$) with close recharge boundaries ($L_o/a < 0.3$), plane flow to the four sides dominates and corner effects may be neglected.

Plane flownet sketching is a useful and highly effective graphical technique for solving the Laplace Equation – governing the plane flow of groundwater through a soil of uniform or

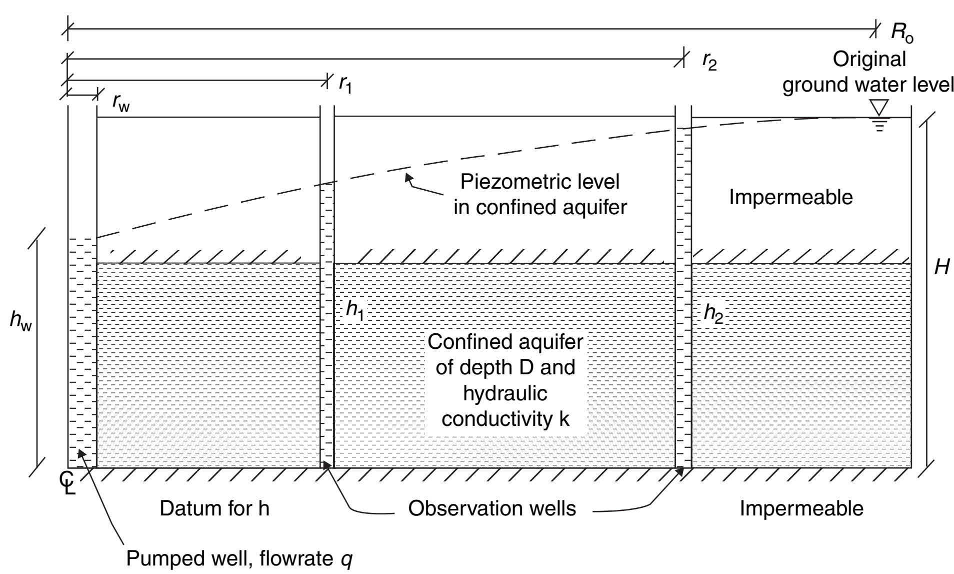

Figure 16.2 Radial flow to a fully penetrating well in a confined aquifer
Reproduced from Powrie (2004)

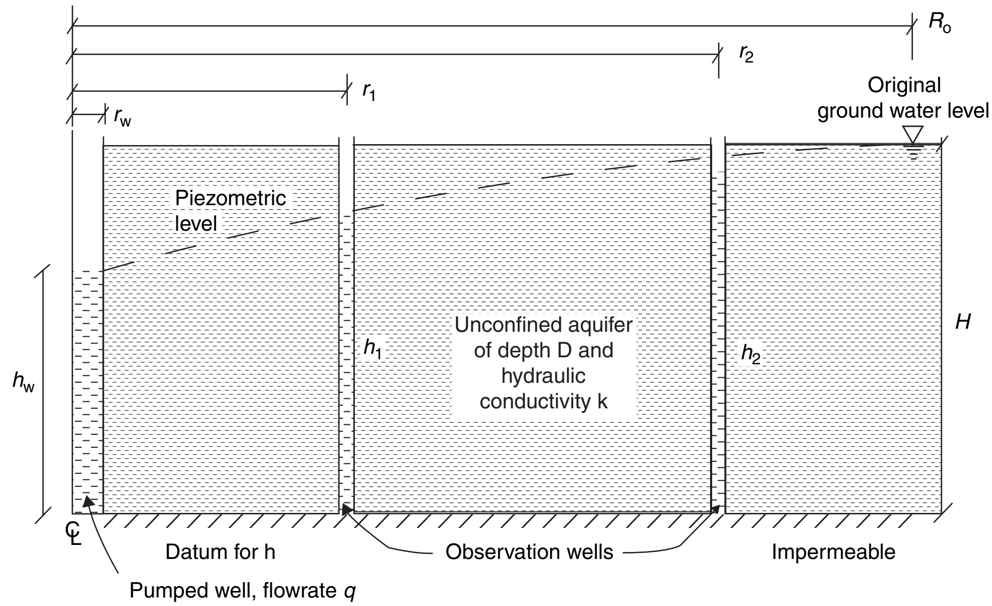

Figure 16.3 Radial flow to a fully penetrating well in an unconfined aquifer
Reproduced from Powrie (2004)

anisotropic hydraulic conductivity. The derivation of the equation, and a comprehensive introduction to flownet sketching, is given by Powrie (2004). In summary, plane flownet sketching involves determining the upper and lower equipotentials (i.e. lines of equal head) representing the source of water and the sink, and two bounding flowlines. The space so defined is then filled with a network of intermediate flowlines which intersect intermediate equipotentials at 90° to form elements known as 'curvilinear squares', whose length is equal to their breadth. (In practice, this means that a circle can be fitted inside each of them, although the circles will vary in diameter depending on the position of the element in the flownet.) In an unconfined aquifer, the need to determine the position of the top flowline (which represents the drawn down water table) as part of the general flownet sketching process represents a small additional complication. The flowrate per metre run is then calculated as:

$$q = kH \, {}^{N_F}\!/_{N_H} \tag{16.8}$$

where H is the overall head drop between the upper and lower equipotentials (the source and the sink), N_F is the number of flowtubes (which is equal to the number of flowlines minus 1) and N_H is the number of equipotential drops (i.e. the number of equipotentials minus 1). A full description of the basic technique, which progresses by trial and error, is given by Powrie (2004).

An example of a plane flownet in the vertical plane is shown in **Figure 16.4**.

In cases where there is no similarity of vertical cross-sections and flow is primarily horizontal, flownets may be sketched in the horizontal plane as shown in **Figure 16.5**. The principles are the same, but the total flowrate is calculated by multiplying the flownet result (equation (16.8)) by the depth of the aquifer. An inherent assumption is that regions of vertical flow, for example up into the excavation, are small and do not significantly affect the basic premise of substantially horizontal flow.

Although the technique could be viewed as having been superseded by computer-based numerical methods, flownet sketching is quick and simple and has the great advantage of giving strong insights into the physicality of, and the main factors governing, the flow regime.

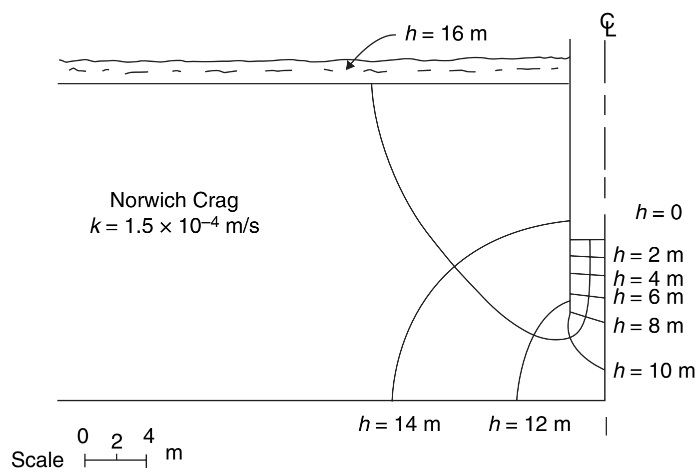

Figure 16.4 Example of a plane flownet in one half of the vertical cross-sectional plane for flow into a trench excavation
Reproduced from Powrie (2004)

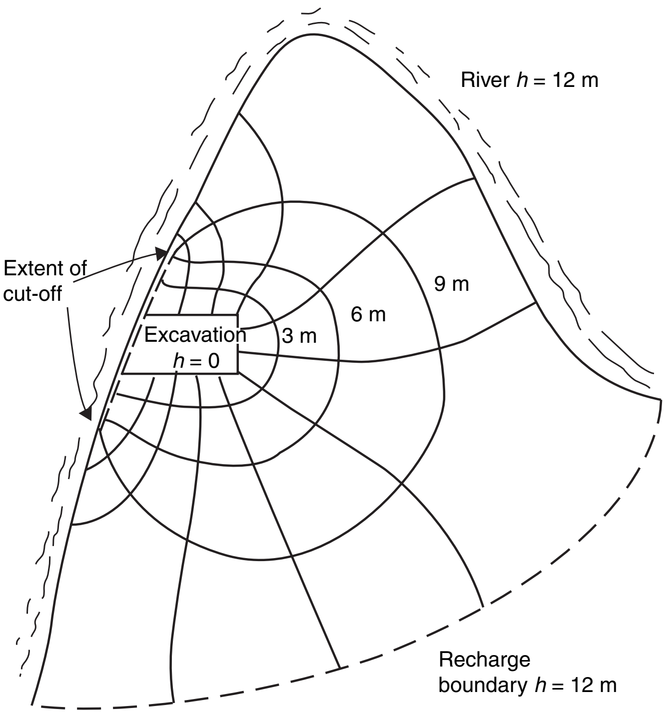

Figure 16.5 **Example of a horizontal plane flownet, for flow to an excavation in a deep chalk aquifer on the Medway peninsula**
Reproduced from Powrie (2004)

A finite difference solution to a plane flow problem can be obtained by establishing a grid of nodes and using Darcy's Law to write an expression for the flowrate into each node – in terms of the head difference between it and the adjacent nodes, the implied area of flow, and the soil hydraulic conductivity. At the steady state, the net flowrate into or out of each node is zero (except of course for the nodes on the constant head or equipotential boundaries). To obtain values of head that satisfy the condition of zero net flow into or out of each internal node, and give a sum of flowrates *into* the boundary nodes on the upper equipotential (the source) equal to the sum of the flowrates *out* of the boundary nodes on the lower equipotential (the sink), solution is by trial and error and is greatly facilitated by the use of a spreadsheet.

16.4 More complex flow regimes

Flownets can be sketched for anisotropic soils in the usual way, provided that the real geometry of the cross-section has first been 'transformed' to account for the difference between the vertical and the horizontal hydraulic conductivities. The true horizontal dimensions, x, must be multiplied by $\sqrt{(k_v/k_h)}$, where k_v and k_h are the hydraulic conductivities in the vertical and horizontal directions respectively. As k_v is usually less than k_h, this involves a shrinking of the horizontal distances on the transformed section. Equation (16.8) for the calculation of flowrates holds, except that the effective hydraulic conductivity of the transformed section, k_t, that must be used for this purpose is the geometric mean of k_v and k_h – in other words $\sqrt{(k_v k_h)}$. Derivations and further details are given in Powrie (2004).

Flownet sketching can also accommodate the presence of two or more strata of different hydraulic conductivities. If the ratio of hydraulic conductivities is in the order of hundreds, one will act as a reservoir compared with the other (i.e. any head drop within it will be negligible compared with that in the less permeable stratum), and this is the appropriate approximation to make. If the ratio of hydraulic conductivities is in the order of one to 100, the flownet should be sketched so that the flowlines deflect through an angle ($\beta_1 - \beta_2$) as they pass from a soil with hydraulic conductivity k_1 into a soil with smaller hydraulic conductivity k_2, such that

$$\frac{\tan\beta_1}{\tan\beta_2} = \frac{k_1}{k_2}. \tag{16.9}$$

This is analagous to the refraction of light as it passes between media of different optical densities. The derivation is given by Powrie (2004).

A comprehensive guide to flownet sketching for these more complex cases and even transient flow is given by Cedergren (1989), but given the ready availability of spreadsheets and low-cost personal computing it is probably easier to solve such problems using the finite difference approach and a spreadsheet, or one of the commercial computer programs available for modelling groundwater flow. The underlying principles, however, remain Darcy's Law and continuity of flow, and success still depends on understanding the physical constraints to the flow domain and the judicious selection of numerical values of soil hydraulic conductivity.

16.5 Groundwater control for stability of excavations

Whenever an excavation is made below the natural water table, at least in a non-clay soil, groundwater and pore water pressures must be controlled to prevent flooding and potential instability of the excavation. This can be achieved by physical means, e.g. grouting or a cut-off wall, pumping from dewatering wells placed in or around the excavation, or most usually a combination of both. Even when the sides of the excavation are supported by retaining walls, pumping may still be necessary to prevent instability of the base. Pumping will also often reduce the pore water pressures acting on the outside of the retaining walls, enabling them to be constructed more economically. In built-up areas, consideration should be given to mitigating the potential for settlement damage to buildings resulting from ground settlements associated with reducing the pore water pressures outside the excavation. Powrie and Roberts (1995) provide a case history on this subject.

As outlined earlier in this chapter, flow calculations are required (i) to estimate the likely flowrate that a dewatering system will have to pump, and (ii) to calculate the reductions in pore water pressure that a dewatering system will achieve. The pore water pressures will feed in turn into calculations relating to the stability of sheet pile and other retaining walls, excavation side slopes and the excavation base. Most of these calculations are discussed elsewhere in this manual, but the importance of maintaining stability of the base will be discussed briefly here.

The base of an excavation will become unstable if the pore water pressures within a certain depth of the excavated surface become equal to the vertical total stress due to the weight of soil. The 'certain depth' will depend to some extent on the width of the excavation. If the sidewalls are of the embedded type, they will offer some additional frictional resistance to uplift, but the effect will diminish towards the centre of the excavation especially if the excavation is wide.. Ignoring any benefits of sidewall friction, it can easily be shown that instability or uplift will occur when the hydraulic gradient associated with upward flow to the base of the excavation is approximately 1, see Powrie (2004); or, when the pore water pressure on the base of a plug of low permeability soil exceeds the vertical total stress due to its weight.

The perils of ignoring the potential for unrelieved pore water pressures at depth to cause major damage, and possibly even loss of life, should not be underestimated and are illustrated by the case study summarised in **Figure 16.6**. This relates to an excavation for a pumping station near Weston-super-Mare which the contractor attempted to make without taking steps to reduce the pore water pressures in the underlying silty sand. As formation level was approached, the base became unstable and the excavation rapidly filled with water. This was quite predictable: at full depth, the pore water pressure at the point A on the base of the 6 m depth of soft clayey silt that remains below the excavation floor is 13 m × 10 kN/m^3 = 130 kPa, which is substantially in excess of the total vertical stress due to the weight of the clayey silt of 6 m × 18 kN/m^3 = 108 kPa. The remedial works necessitated by the failure were substantially more expensive than the cost of a pre-emptive groundwater control scheme, quite apart from the damage to plant and the potential for personal injury or even loss of life.

Methods of groundwater control by pumping are summarised in **Table 16.2**. Further details are given in Powrie (2004), Cashman and Preene (2001), Preene *et al.* (2000) and Powers (1992). The suitability of each method depends on the pumped flowrate, and hence the soil hydraulic conductivity and the drawdown, as shown schematically in **Figure 16.7**.

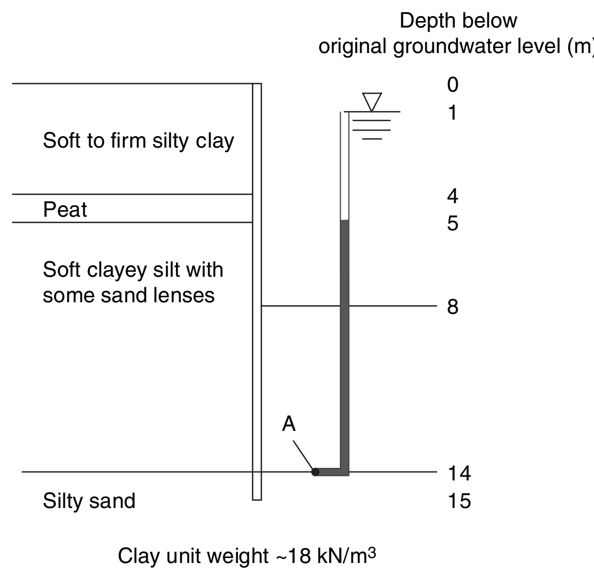

Figure 16.6 Case study demonstrating the importance of controlling pore water pressures to an adequate depth below the excavation floor

Method	Thumbnail description	Comments and case studies
Open sump	Literally an open sump into which groundwater is allowed to flow under gravity, and pumped away by a stand-alone diesel or electric site pump	Subject to limitations on space and accessibility, this method is generally only suitable for modest drawdowns (up to about 2 m) in well interlocked, open gravels
Vacuum wellpoints	Closely spaced small wells (typically in a line 1–3 m apart) approximately 50 mm diameter connected by risers to a common header main, pumped by a surface vacuum pump	Maximum drawdown (owing to limitations of vacuum lift) about 6 m, but can be used in multiple stages. Comfortable with flowrates typical of sandy soils
Deep wells	Large diameter wells (e.g. 200 mm diameter screens in 350 mm diameter bores) pumped individually by slimline electrical submersible pumps. Widely spaced so recovery of water table between wells is significant and well depth must allow for this	Can deal with higher flowrates and drawdowns than wellpoints. Restriction on depth of drawdown likely to be economic rather than physical. Vacuum can be applied via a separate system in lower permeability ground. Powrie and Roberts (1995); Bevan *et al.* (2010)
Ejectors (sometimes called eductors)	Nozzle and venturi devices installed at depth in deep but small diameter boreholes (minimum 50 mm), driven by high pressure water pumped from the surface	Will draw air as well as water so will create a vacuum automatically if the well is sealed. Particularly useful for lower permeability soils. Powrie and Roberts (1990); Roberts *et al.* (2007)

Table 16.2 Methods of groundwater control by pumping

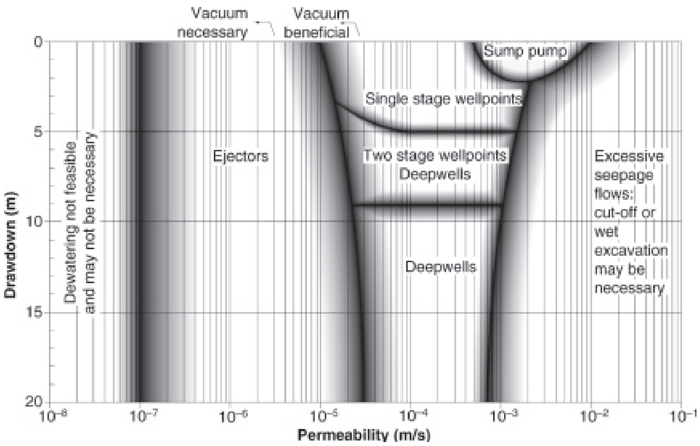

Figure 16.7 Range of application of dewatering techniques
Reproduced with permission from CIRIA C515, Preene *et al*. (2000), www.ciria.org

16.6 Transient flow

So far, we have considered steady-state flow, in which the volume of water flowing into a soil element in a given time is the same as the volume of water flowing out, i.e. there is no change in the volume of the soil element. This is not always the case; if the volume of the soil element is changing over time (for example in response to a change in the external loading regime), the flow into the element will be greater than the flow out if the soil volume is increasing (swelling), or less than the flow out if the soil volume is decreasing (compression). Non steady-state flow is known as *transient flow*, and the time-dependent process of volume change associated with it is known as *consolidation* (or possibly *swelling* if it is a volume increase).

For the effective stress to increase, the soil skeleton must compress: this requires water to flow from the pores, which cannot occur instantaneously because the rate of flow is limited by the hydraulic conductivity of the soil. Thus an increase in total stress results initially in an increase in the pore water pressure. The increase in pore water pressure causes local hydraulic gradients, in response to which water will flow out of the soil. As water flows from the pores, the pore water pressure falls back towards its equilibrium value, the soil compresses and the effective stress increases to make up for the reduction in pore water pressure. Eventually, the pore water pressures return to the initial equilibrium values, and transient flow stops. The soil has compressed, the effective stress has increased to mirror the increase in total stress, and consolidation is complete.

The rate of consolidation increases with the soil hydraulic conductivity k. The amount of compression associated with the eventual increase in effective stress decreases as the soil stiffness in compression, E'_o, increases. The parameters k and E'_o are combined to give the consolidation coefficient, c_v,

$$c_v = \frac{kE'_0}{\gamma_w} \qquad (16.10)$$

Soil parameter	Medium sand	Fine sand	Silt	Clay
Hydraulic conductivity k (m/s)	10^{-3}	10^{-4}	10^{-6}	10^{-8}
Stiffness in one-dimensional compression E'_o (MPa)	100	50	10	2
Time t for completion of transient flow with drainage path length d = 50 m	4 min	1.4 h	1 month	40 years

Table 16.3 Indicative times for completion of transient flow with a drainage path length of 50 m
Data taken from Preene *et al*. (2000) (CIRIA C515)

where γ_w is the unit weight of water. The consolidation coefficient c_v will obviously vary with soil stiffness and hydraulic conductivity, and hence with stress history, stress state and stress path. For stiff soils subjected to small changes in stress, it might be reasonable to assume an equivalent single value of c_v in analysis. However, for problems involving softer sediments and larger changes in stress, an approach which allows c_v to vary with both depth and stress will probably be required. The consolidation time also depends on the distance to the drainage boundary, d, and the flow geometry. For simple flow geometries approximating to one-dimensional flow, consolidation is usually considered to be substantially complete after a time t such that the dimensionless group T defined in Equation (16.11) is approximately equal to one:

$$T = \frac{c_v t}{d^2} = 1 \qquad (16.11)$$

In principle, all soils will undergo transient flow in response to a change in total stress or boundary pore water pressure. However, consolidation is normally only of practical importance in saturated, low permeability, fine grained soils. The reason for this, as illustrated in **Table 16.3**, is that sandy soils are so relatively stiff and more permeable that the timescale for transient flow is very short – perhaps a matter of minutes rather than months or even years.

It is important to realise that, while much practical soil mechanics theory is relatively straightforward, huge simplifications are involved both in idealising the complex behaviour of the soil and in choosing often a single value to characterise highly variable parameters such as hydraulic conductivity, soil stiffness and consolidation coefficient. This is why experience, judgement and a respect for reality are so important in translating theory into geotechnical engineering practice.

16.7 Summary

- The flow of water through the ground is governed by Darcy's Law, and controlled by a parameter known as the *hydraulic conductivity* or *permeability*.

- Hydraulic conductivity varies widely across the range of soils and can be difficult to determine with confidence, especially on the basis of tests involving small volumes of soil, owing to scale and fabric effects.
- Simple flow regimes can be analysed mathematically or by the use of plane flownet sketching.
- Anisotropy, and the deflection of flowlines as they pass between soil strata of differing hydraulic conductivity, can be taken into account if needed.
- Steps must be taken to lower groundwater levels and pore water pressures in the vicinity of an excavation below the natural water table, to prevent instability of the excavation sides or base. This is usually achieved by pumping from wells in a process known as construction detwatering.
- Transient flow occurs as a soil is changing in volume in response to a change in boundary stress. This leads to the process known as consolidation, which is usually only significant for low-permeability (clay) soils.

16.8 References

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All chapters within Sections 1 *Context* and 2 *Fundamental principles* together provide a complete introduction to the Manual and no individual chapter should be read in isolation from the rest.