Groundwater

Soils consist of mineral particles in contact surrounded by voids or pores. The voids contain fluid which may be liquid, gas or a mixture of the two.

The relative volumes can be described by the void ratio e, specific volume v, or porosity n.

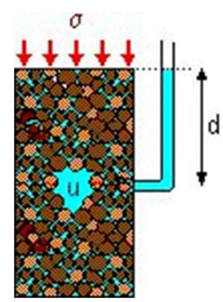
In dry soils (S_r = 0) the single pore fluid is air.
In saturated soils (S_r = 1) the single pore fluid is water.

Discussion of groundwater is usually concerned with saturated soils.

- Pore water pressure
- Permeability
- Analytical solutions
- Flow nets
- Quick condition and piping
- Measurement
- Groundwater control
- Some case histories

Pore water pressure

- Water table
- Elevation, pressure and total head
- Hydraulic gradient
- Effective stress



In general, the water in the voids of an element of saturated soil will be under pressure, either due to the physical location of the soil or as a result of external forces. This pressure is the pore water pressure or **pore pressure** u. It is measured relative to atmospheric pressure.

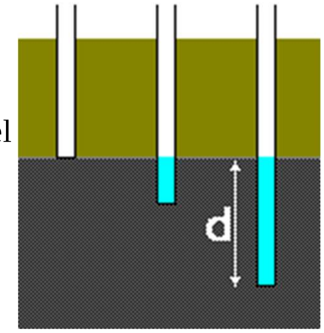
When there is no flow, the pore pressure at depth **d** below the water surface is:
$$\mathbf{u = \gamma_w\, d}$$

Pore water pressure

Water table

- Fine-grained soils
- Coarse-grained soils
- Perched water table

The level in the ground at which the pore pressure is zero (equal to atmospheric) is defined as the **water table** or **phreatic surface**.

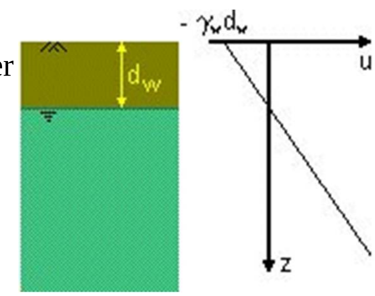


When there is no flow, the water surface will be at exactly the same level in any stand pipe placed in the ground below the water table. This is called a **hydrostatic** pressure condition.

The pore pressure at depth **d** below the water table is: : $\mathbf{u = \gamma_w\, d}$

Water table

Fine-grained soils



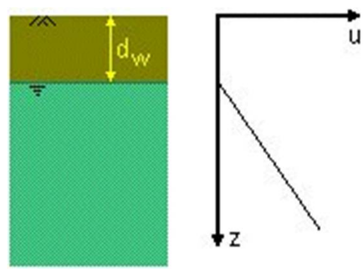
In fine grained soils, surface tension effects can cause capillary water to rise above the water table. It is reasonable to assume that the pore pressure varies linearly with depth, so the pore pressure above the water table will be negative.

If the water table is at depth d_w then the pore pressure at the ground surface is $\mathbf{u_o = -\gamma_w . d_w}$
and the pore pressure at depth z is $\mathbf{u = \gamma_w (z - d_w)}$

Where the water table is deeper, or where evaporation is taking place from the surface, saturation with capillary water may not occur. The height to which the soil remains saturated with negative pore pressures above the water table is called the capillary rise.

Water table

Coarse-grained soils



Below the water table the soil can be considered to be saturated. In coarse-grained soils, water will drain from the pores and air will therefore be present in the soil between the ground surface and the water table.

Consequently, pore pressures above the water table can usually be ignored. Below the water table, hydrostatic water pressure increases linearly with depth.

With the water table at depth d_w

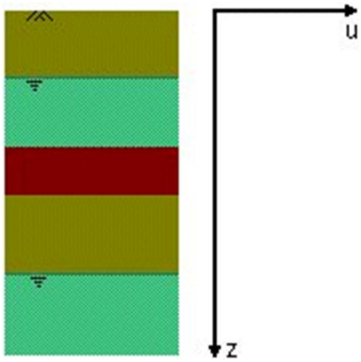
$u = 0$ for $z < d_w$

$u = \gamma_w(z - d_w)$ for $z > d_w$

Water table

Perched water table

Where the ground contains layers of permeable soil (e.g. sands) interspersed with layers of much lower permeability (e.g. clays) one or more **perched water tables** may develop and the overall distribution of pore pressure with depth may not be exclusivelyly linear.



Detection of perched water tables during site investigation is important, otherwise erroneous estimates of *in-situ* pore pressure distributions can arise.

Pore pressure conditions below perched water tables may be affected by local infiltration of rainwater or localised seepage and therefore may not be in hydrostatic equilibrium.

Show the graph of water pressure

Click on a soil layer to display a description

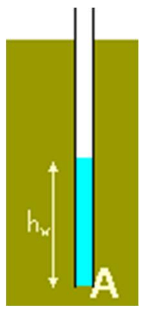
The description appears in the status bar when the mouse is held over a layer.

In Internet Explorer is also appears as a popup label

Pore water pressure

Elevation, pressure and total head

Pore pressure at a given point (e.g. point A in the diagram) can be measured by the height of water in a standpipe located at that point.

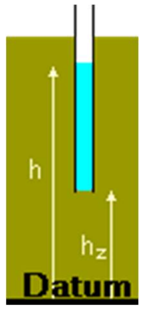


Pore pressures are often indicated in this way on diagrams.

The height of the water column is the **pressure head** (h_w)

$$h_w = \frac{u}{\gamma_w}$$

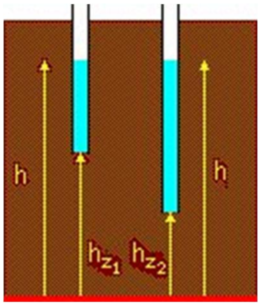
To identify significant differences in pore pressure at different points, we need to eliminate the effect of the points' position. A height datum is required from which locations are measured. The **elevation head** (h_z) of a point is its height above the datum line. The height above the datum of the water level in the standpipe is the **total head** (h).



$$h = h_z + h_w$$

Pore water pressure

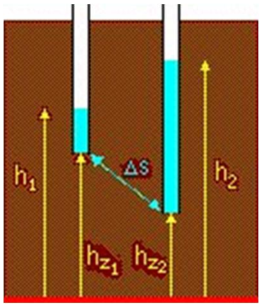
Hydraulic gradient



Flow of pore water in soils is driven from positions of higher total head towards positions of lower total head. The level of the datum is arbitrary. It is **differences** in total head that are important. The hydraulic gradient is the rate of change of total head along the direction of flow.

$$\mathbf{i = \Delta h / \Delta s}$$

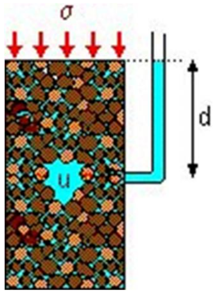
In each diagram there are two points, a small distance Δs apart, h_{z1} and h_{z2} above datum.
In the first diagram, the total heads are **equal**. The difference in pore pressure is entirely due to the difference in altitude of the two points and the pore water has no tendency to flow.



In the second diagram, the total heads are **different**. The hydraulic gradient is
$i = (h_2 - h_1) / \Delta s$
and the pore water tends to flow.

Pore water pressure

Effective stress



All strength and stress:strain characteristics of soils can be linked to changes in effective stress

Effective stress (σ') = total stress (σ) - pore water pressure (u)
$\sigma' = \sigma - u$

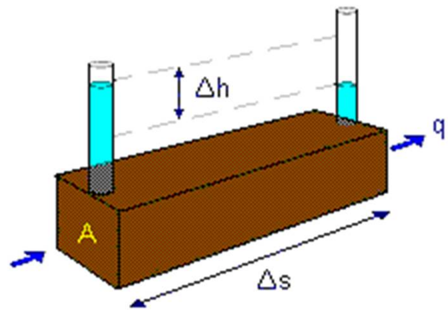
Groundwater

Permeability

- Void ratio
- Stratified soil
- Seepage velocity
- Temperature

Darcy's law

The rate of flow of water q (volume/time) through cross-sectional area A is found to be proportional to hydraulic gradient i according to Darcy's law:



$$v = \frac{q}{A} = k.i \quad i = \frac{\Delta h}{\Delta s}$$

*where **v** is flow velocity and **k** is coefficient of permeability with dimensions of velocity (length/time).*

The coefficient of permeability of a soil is a measure of the conductance (i.e. the reciprocal of the resistance) that it provides to the flow of water through its pores.

The value of the coefficient of permeability k depends on the average size of the pores and is related to the distribution of particle sizes, particle shape and soil structure. The ratio of permeabilities of typical sands/gravels to those of typical clays is of the order of 10^6. A small proportion of fine material in a coarse-grained soil can lead to a significant reduction in permeability.

Permeability

Void ratio and permeability

Permeability of all soils is strongly influenced by the density of packing of the soil particles which can be simply desrcibed through void ratio e or porosity n.

Sands

For filter sands it is found that $k \approx 0.01 (d_{10})^2$ m/s where d_{10} is the effective particle size in mm. This relationship was proposed by Hazen.

The Kozeny-Carman equation suggests that, for laminar flow in saturated soils:

$$k = \frac{1}{k_0 k_T S_S^2} \cdot \frac{e^3}{1+e} \cdot \frac{\gamma_w}{\eta}$$

where k_o and k_T are factors depending on the shape and tortuosity of the pores respectively, S_s is the surface area of the solid particles per unit volume of solid material, and γ_w and η are unit weight and viscosity of the pore water. The equation can be written simply as

$$k = C.\frac{e^3}{1+e} \approx C.e^2$$

Clays

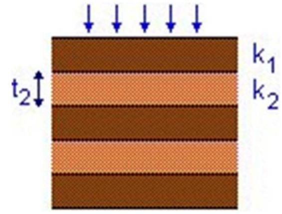
The Kozeny-Carman equation does not work well for silts and clays. For clays it is typically found that

$$\log_{10} k = \frac{e - e_k}{C_k}$$

where C_k is the permeability change index and ek is a reference void ratio.
For many natural clays C_k is approximately equal to half the natural void ratio.

Permeability

Stratified soil and permeability



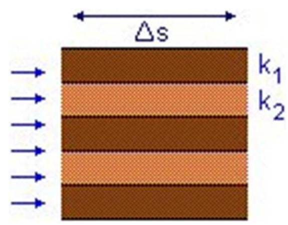
Consider a stratified soil having horizontal layers of thickness t_1, t_2, t_3, etc. with coefficients of permeability k_1, k_2 k_3, etc.

For **vertical flow**, the flow rate q through area A of each layer is the same. Hence the head drop across a series of layers is

$$\Delta h = \frac{qt_1}{Ak_1} + \frac{qt_2}{Ak_2} + \frac{qt_3}{Ak_3}$$

The average coefficient of permeability is

$$k_v = \frac{t_1 + t_2 + t_3}{\frac{t_1}{k_1} + \frac{t_2}{k_2} + \frac{t_3}{k_3}}$$



For **horizontal flow**, the head drop Δh over the same flow path length Δs will be the same for each layer. So $i_1 = i_2 = i_3$ etc. The flow rate through a layered block of soil of breadth B is therefore

$$q = Bt_1k_1i_1 + Bt_2k_2i_2 + Bt_3k_3i_3$$

The average coefficient of permeability is

$$k_h = \frac{t_1k_1 + t_2k_2 + t_3k_3}{t_1 + t_2 + t_3}$$

Permeability

Seepage velocity

Darcy's Law relates flow velocity (v) to hydraulic gradient (i). The volume flow rate q is calculated as the product of flow velocity v and total cross sectional area:
q = v.A

At the particulate level the water follows a tortuous path through the pores. The average velocity at which the water flows through the pores is the ratio of volume flow rate to the average area of voids A_v on a cross section normal to the macroscopic direction of flow. This is the **seepage velocity** v_s

$$v_s = \frac{q}{A_v}$$

Porosity of soil is related to the volume fraction of voids

$$n = \frac{V_v}{V} \approx \frac{A_v}{A} v_s \approx \frac{v}{n}$$

Seepage velocity can be measured in laboratory models by injecting dye into the seeping pore water and timing its progress through the soil.

Permeability

Temperature and permeability

The flow of water through confined spaces is controlled by its viscosity η and the viscosity is controlled by temperature.

An alternative permeability K (dimensions: length^2) is sometimes used as a more absolute coefficient depending only on the characteristics of the soil skeleton.

$$K = \frac{\eta k}{\gamma_w}$$

The values of k at 0°C and 10°C are 56% and 77% respectively of the value measured at 20°C.

Groundwater

Analytical solutions

- Steady one-dimensional flow
- Quasi-one-dimensional and radial flow
- Two-dimensional flow, Laplace
- Transient flow, consolidation

In **steady-state** flow, the pressures and flow rates remain constant over time. In **transient** flow, the pressures and flow rates are time-dependent.

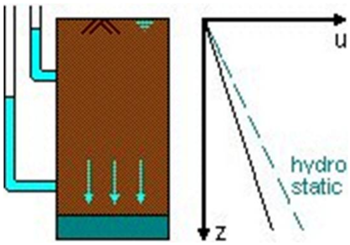
Steady one-dimensional flow is the simplest case, to which Darcy's law can be applied. This can be extended to cases of variable aquifer thickness and radial flow. The analysis of steady two-dimensional flow is more complex and results in flow nets.

Analytical solutions

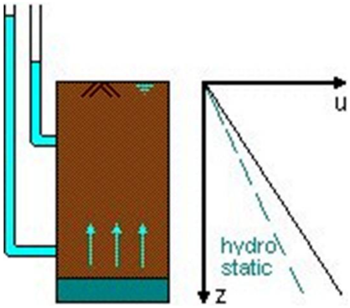
Steady one-dimensional flow

Darcy's Law indicates the link between flow rate and hydraulic gradient. For one-dimensional flow, constant flow rate implies constant hydraulic gradient.

Steady downward flow occurs when water is pumped from an underground aquifer. Pore pressures are then lower than hydrostatic pressures.



Steady upward flow occurs as a result of artesian pressure when a less permeable layer is underlain by a permeable layer which is connected through the ground to a water source providing pressures higher than local hydrostatic pressures.
The fountains of London were originally driven by artesian pressure in the aquifers trapped beneath the London clay. Pumping from aquifers over the centuries has lowered the water pressures below artesian levels.

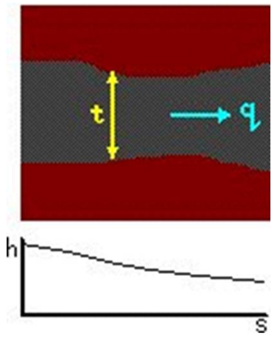


Analytical solutions

Quasi-one-dimensional and radial flow

- Cylindrical flow: confined aquifer
- Cylindrical flow: groundwater lowering
- Spherical flow

Where flow occurs in a confined aquifer whose thickness varies gently with position the flow can be treated as being essentially one-dimensional.
The horizontal flow rate q is constant. For an aquifer of width B and varying thickness t, the discharge velocity



$$v = \frac{q}{Bt}$$

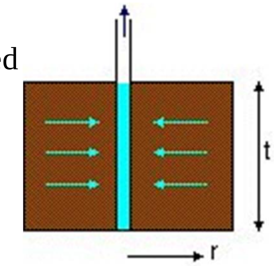
and Darcy's Law indicates that

$$i = \frac{dh}{ds} = \frac{q}{Btk}$$

Hydraulic gradient varies inversely with aquifer thickness.

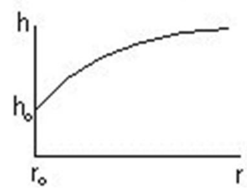
[Quasi-one-dimensional and radial flow]

Cylindrical flow: confined aquifer



Steady-state pumping to a well which extends the full thickness of a confined aquifer is a one-dimensional problem which can be analysed in cylindrical coordinates: pore pressure or head varies only with radius r.

Darcy's Law still applies, with hydraulic gradient dh/dr and area A varying with radius: $A = 2\pi r.t$



$$\frac{q}{A} = ki = \frac{dh}{dr}$$

$$dh = \frac{q}{2\pi tk}\frac{dr}{r} \qquad h - h_o = \frac{q}{2\pi tk}\ln(r/r_o)$$

where r_o is the radius of the borehole and h_0 the constant head in the borehole.

[Quasi-one-dimensional and radial flow]

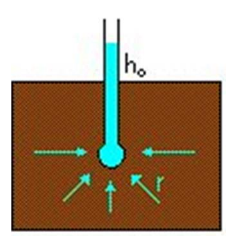
Cylindrical flow: groundwater lowering

Pumping from a borehole can be used for deliberate groundwater lowering in order to facilitate excavation. This is an example of quasi-one-dimensional radial flow with flow thickness t=h. Then $A=2\pi r.h$ and

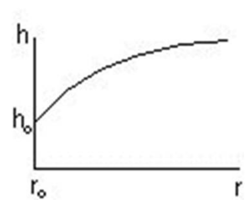
$$h.dh = \frac{q}{2\pi k}\frac{dr}{r} \qquad h^2 - h_o^2 = \frac{q}{\pi k}\ln(r/r_o)$$

[Quasi-one-dimensional and radial flow]

Spherical flow



Variation of pore pressure around a point source or side (for example, a piezometer being used for *in-situ* determination of permeability) is a one-dimensional problem which can be analysed in spherical coordinates: pore pressure or head varies only with radius r.



Darcy's Law still applies, with hydraulic gradient dh/dr and area A varying with radius: $A=4\pi r^2$

$$\frac{q}{A} = ki = \frac{dh}{dr}$$

$$dh = \frac{q}{4\pi k}\frac{dr}{r^2} \quad h - h_o = \frac{q}{4\pi k}\left(\frac{1}{r_o} - \frac{1}{r}\right)$$

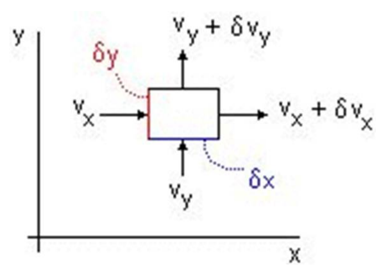
where r_0 is the radius of the piezometer and h_0 the constant head in the piezometer.

Analytical solutions

Two-dimensional flow, Laplace

- Anisotropic soil

Two-dimensional steady flow of the incompressible pore fluid is governed by Laplace's equation which indicates simply that any imbalance in flows into and out of an element in the x direction must be compensated by a corresponding opposite imbalance in the y direction. Laplace's equation can be solved graphically, analytically, numerically, or analogically.



For a rectangular element with dimensions δ_x, δ_y and unit thickness, in the x direction the velocity of flow into the element is

$$v_x = -k\frac{\partial h}{\partial x}$$

the negative sign being required because flow occurs **down** the hydraulic gradient. The velocity of flow out of the element is

$$v_x + \delta v_x = -k\left(\frac{\partial h}{\partial x} + \frac{\partial^2 h}{\partial x^2}\delta x\right)$$

Similar expressions can be written for the y direction. Balance of flow requires that

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} = 0$$

and this is Laplace's equation. In three dimensions, Laplace's equation becomes

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial z^2} = 0$$

Two-dimensional flow, Laplace

Anisotropic soil

For a soil with permeability k_x and k_y in the x and y directions respectively, Laplace's equation for two-dimensional seepage becomes

$$k_x \frac{\partial^2 h}{\partial x^2} + k_y \frac{\partial^2 h}{\partial_y{}^2} = 0$$

This can be solved by applying a scale factor to the x dimensions so that transformed coordinates xt are used

$$x_t = x\sqrt{\frac{k_y}{k_x}}$$

In the transformed coordinates the equation regains its simple form

$$\frac{\partial^2 h}{\partial x_t{}^2} + \frac{\partial^2 h}{\partial y^2} = 0$$

and flownet generation can proceed as usual. Calculations of flow are made using an equivalent permeability

$$k_t = \sqrt{k_x k_y}$$

It may be preferable in some cases to transform the y coordinates using:

$$y_t = y\sqrt{\frac{k_x}{k_y}}$$

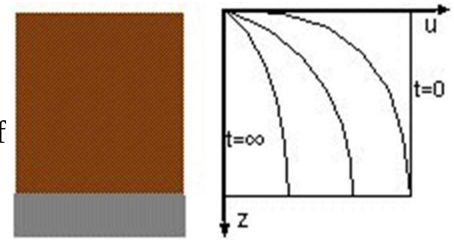
The equivalent permeability remains unchanged.

For many natural sedimentary soils seasonal variations in the depositional regime have resulted in horizontal macroscopic permeabilities significantly greater than vertical permeabilities. Transformation of coordinates lends itself to analysis of seepage in such situations.

Analytical solutions

Transient flow, consolidation

Since water may be regarded as being essentially incompressible, unsteady flow may arise when water is drawn into or expelled from pores as a result of changes in the size of pores. This can only occur as a result of changes volume associated with changes in effective stress.



The time-dependent transient change in pore pressure that occurs as a result of some perturbation, and associated change in effective stress is called **consolidation**.

One-dimensional compression tests in an oedometer define the relationship between vertical effective stress σ'_v and specific volume v or void ratio e from which a one-dimensional compliance mv can be defined

$$m_v = \frac{\delta v}{v \delta \sigma'_v}$$

Then, under conditions of constant total stress, consolidation is governed by a diffusion equation:

$$\frac{\partial u}{\partial t} = c_v \frac{\partial^2 u}{\partial z^2} \quad c_v = \frac{k}{m_v \gamma_w}$$

where cv is the coefficient of consolidation having dimensions (length²/time).

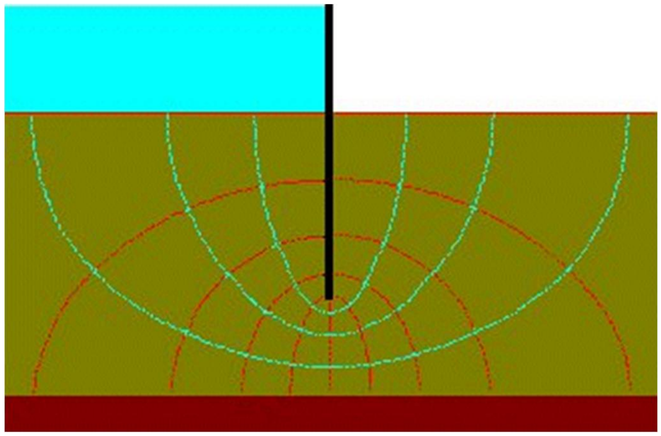
Solutions of the consolidation equation are typically presented as **isochrones**, i.e. variations of pore pressure with position at successive times, but can also be converted to curves linking settlement with time.

Groundwater

Flow nets

- Calculation of flow
- Calculation of total flow
- Boundary between layers
- Boundary conditions
- Flow through embankments

Solutions to Laplace's equation for two-dimensional seepage can be presented as flow nets. Two orthogonal sets of curves form a flow net:
equipotentials connecting points of equal total head h



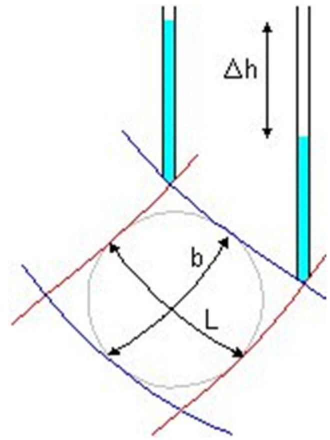
flow lines indicating the direction of seepage down a hydraulic gradient

If standpipe piezometers were inserted into the ground with their tips on a single equipotential then the water would rise to the same level in each standpipe. (The pore pressures would be different because of their different elevations.)

There can be no flow along an equipotential, because there is no hydraulic gradient, so there can be no component of flow across a flow line. The flow lines define channels along which the volume flow rate is constant.

Flow nets

Calculation of flow



Consider an element from a flow channel of length L between equipotentials which indicate a fall in total head Δh and between flow lines b apart. The average hydraulic gradient is

$$i = \frac{\Delta h}{L}$$

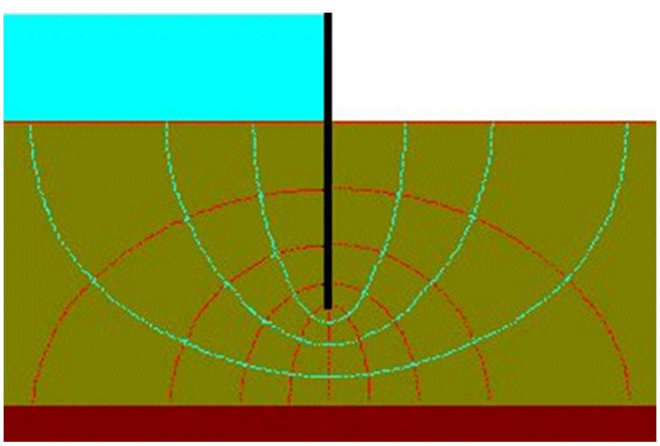
and for unit width of flow net the volume flow rate is

$$q = kb\frac{\Delta h}{L}$$

There is an advantage in displaying or sketching flownets in the form of curvilinear 'squares' so that a circle can be insrcibed within each four-sided figure bounded by two equipotentials and two flow lines. Then $b = L$ and $q = k\Delta h$
so the flow rate through the flow channel is the permeability multiplied by the uniform interval Δh between equipotentials.

Flow nets

Calculation of total flow



For a complete problem, the flownet has been drawn with the overall head drop h divided into Nd equal intervals:

$\Delta h = h / N_d$

with N_f flow channels.

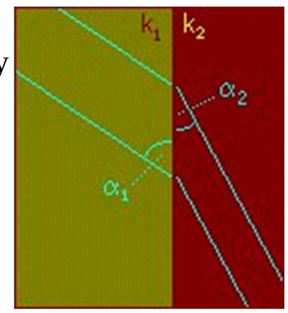
Then the total flow rate per unit width is

$$q = N_f \cdot \frac{k.h}{N_d} = (\frac{N_f}{N_d})kh$$

It is usually convenient in sketching flownets to make Nd an integer. The number of flow channels Nf will then generally not be an integer. In the example shown, of flow under a sheet pile wall
Nd ;= 10, Nf = 3.5 and q = 0.35kh per unit width.

[Flow nets](#)

Boundary between layers



Flow across a boundary between two layers of soil of different permeability produces a refraction effect.

Consideration of continuity of flow and of continuity of velocity normal to the interface shows that

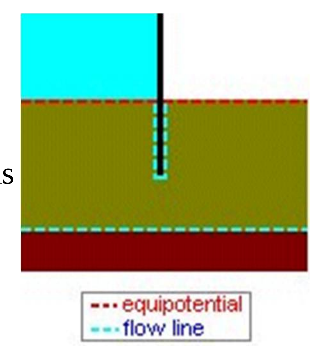
$$\frac{\tan \alpha_1}{\tan \alpha_2} = \frac{k_1}{k_2}$$

It is not possible to construct a flow net with curvilinear squares on both sides of the interface unless the head drop between equipotentials is changed in inverse proportion to the permeability ratio.

If the ratio of permeabilities is greater than about 10, e.g. at the boundary of a drainage layer then construction of the part of the flow net in the more permeable soil is unlikely to be necessary.

[Flow nets](#)

Boundary conditions



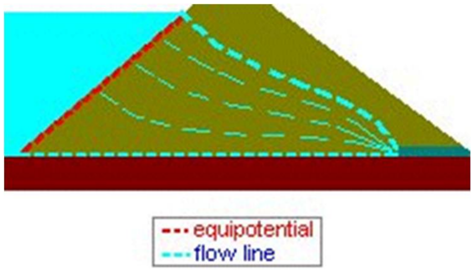
A surface on which the total head is fixed (for example, from the level of a river, pool, reservoir) is an [equipotential](#). A surface across which there is no flow (for example, an impermeable soil layer or an impermeable wall) is a [flow line](#)

For the situation shown, with flow occurring under a sheet pile wall, the axis of symmetry must also be an equipotential.

Flow nets

Flow through embankments

Seepage through an embankment dam is an example of unconfined flow bounded at the upper surface by a phreatic surface which represents the top flow line and on which the pore pressure is everywhere zero (atmospheric).



Total head changes and elevation changes thus match and for equal head intervals Δh between equipotentials there must be equal vertical intervals between the points of intersection of equipotentials with the phreatic surface.

Groundwater

Quick condition and piping

- Seepage force
- Critical hydraulic gradient

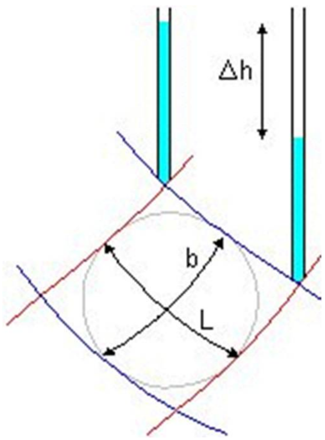
If the flow is upward then the water pressure tends to lift the soil element. If the upward water pressure is high enough the effective stresses in the soil disappear, no frictional strength can be mobilised and the soil behaves as a fluid. This is the **quick** or **quicksand** condition and is associated with piping instabilities around excavations and with liquefaction events in or following earthquakes.

Quick condition and piping

Seepage force

The viscous drag of water flowing through a soil imposes a **seepage force** on the soil in the direction of flow.

Consider the actual distribution of pore water pressure around an element length L and thickness b taken from a flownet, bounded by two equipotentials with fall in head Δh, and two flow lines.



These pore water pressures are partly supporting the weight $\gamma_w bL$ of water in the element and partly providing the seepage force. It is found that the seepage force is

$J = i \gamma_w b L$

equivalent to a seepage pressure (force per unit volume) in the direction of flow

$j = i \gamma w$

Quick condition and piping

Critical hydraulic gradient

The quick condition occurs at a critical upward hydraulic gradient i_c, when the seepage force just balances the buoyant weight of an element of soil. (Shear stresses on the sides of the element are neglected.)

$$i_c \gamma_w V = (\gamma - \gamma_w) V = \gamma' V$$

$$i_c = \frac{\gamma'}{\gamma_w} = \frac{G_s - 1}{V}$$

The critical hydraulic gradient is typically around 1.0 for many soils. Fluidised beds in chemical engineering systems rely on deliberate generation of quick conditions to ensure that the chemical process can occur most efficiently.

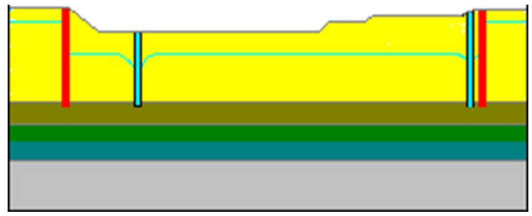
Case studies

The following brief project descriptions illustrate some issues associated with groundwater flow. The list is not exhaustive. In each case, a reference is given to the source material.

- Diaphragm cut-off wall Sizewell B power station
- Ground freezing Rheinberg, West Germany
- Deep dewatering wells Blackwater Valley
- Complex groundwater control system Munich airport
- Buoyancy dependent foundation Lecco bypass, near Milan, Italy
- Wellpoint pumping Sevenoaks, Kent
- Artificial island Trans Tokyo Bay Highway, Japan
- Flooded tunnel and ground freeze Thames Water ring main, London
- Rising groundwater levels The deep aquifer beneath London

- Simulation of rising groundwater Canary Warf, London
- Flooded cofferdam Ennerdale Link Road, River Hull

Case studies



Diaphragm cut-off wall

The Sizewell B power station

Due to the groundwater conditions, construction of the Sizewell B station could not begin until the UK's deepest ever diaphragm wall was installed to isolate the site. The foundations would need excavations nearly 18m below the water table, and dewatering consultants were appointed in 1984. Conventional dewatering techniques were rejected for several reasons. The next idea was to construct a diaphragm wall, extending into the London Clay, and linking with a cofferdam to form a 1260m-long all-encompasing cut-off wall around the whole site. The established techniques for diaphragm walling had only been used down to 30m at that time, but trenching with new reverse circulation rigs could go down to more than 100m very accurately. This solution had several advantages, and had been selected by July 1985. The wall had to have a controlled maximum permeability and virtually leak proof construction joints. Performance was monitored via a network of observation wells and piezometers. After more than 4000000m³ of water had been pumped away, the excavation was dry until the pumps were switched off in the spring of 1992. The water table had been kept at least 2m below the deepest excavation.

Reference

groundwater conditions
50m of dense sands and silts known as the Norwich Crag deposits overlay London Clay and form a natural aquifer. The site is also uncomfortably close to the North sea.

Reasons
Preliminary calculations showed that even with 52 wells (rather than 6 used for the A station), it would still only be possible to lower the water by 16m. The excessive draw down below adjacent bird reserves, settlement beneath Sizewell A, heavy encrustation on the pipework due to high iron content in the groundwater, and a cost of at least £16M, all made this option unacceptable.

advantages
Only nine dewatering wells would be needed, and the construction period would be halved to six months with a saving of £2M.

Case studies

Ground freezing

Rheinberg, West Germany

A record 15GJ/h refrigeration plant formed a tube of ice-stabilised strata reaching 528m down to a dry zechstein formation overlying the Ruhr coalfield: the world's largest ground freeze. The artificially stabilised section of ground, with its steel and concrete lining structure, is the start of a 7.5m diameter shaft continuing down to 1.3km. Its prime purpose is ventilation of the nearby mines. Proximity of the Rhine demanded that a 4m platform should be built to lift the working area above any flood level. But the rise and fall of groundwater levels in the 14m of terrace sands and gravels is insignificant compared with conditions deeper down. Tertiary deposits of sands, silts and clays go down to 100m where there is about 30m chalk. Then there is Bunter sandstone, which is very soft and charged with water right down to 511m where the dry and stable zechstein occurs.

The 44 freeze pipe holes, with 127mm diameter casing, were drilled on a 22m diameter circle, and the freeze was computer-monitored. A central drill hole formed a pressure relief drain for groundwater expelled within the constricting ice front.

Reference

lining structure
When the freeze is switched off, the 30000t of steel and concrete composite tube lining the shaft will retain its structural integrity independent of long term subsidence of the the surrounding aquifer.

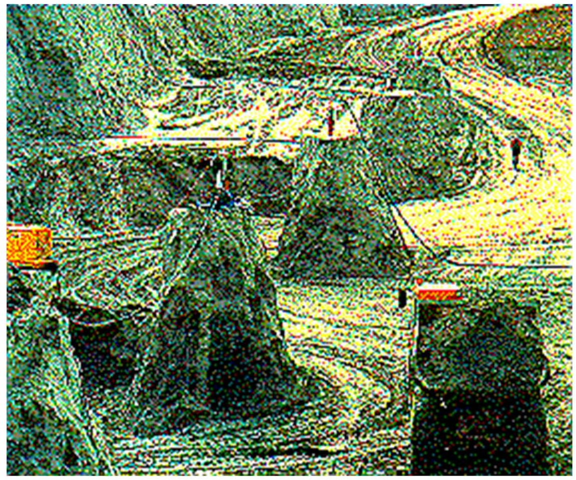
refrigeration plant
Refrigeration comes via nearly 400m³ of calcium chloride brine being delivered at -33ºC and returned to the plant at -28ºC.

Monitoring
Three additional casings were set 120º apart and at 2m, 4m and 6m outside the freeze circle for the computer-monitored thermometer installation assessing the progress of the freeze.

Case studies

Deep dewatering wells

Blackwater Valley



Installation of deep dewatering wells by hole punching is proving a cost-effective and efficient way of dewatering a borrow pit on the northern contract of the Blackwater Valley Route project. A scheme was proposed to extract sand horizons from a pit in the underlying Bracklesham Beds, creating a home for surplus material.

The groundwater lies only a metre below ground level, and the Beds are hydrostatically pressurised, so excavation without reducing the pressure would lead to rupturing. Following a series of drawdown and falling head tests, a two stage scheme was proposed. The conventional first stage produced a drawdown sufficient for excavation of the terrace gravels, reducing ground level by 3m. Working from the reduced level at the top of the Bracklesham Beds, the deep wells were installed. Pumping of these wells produced a drawdown sufficient for the pit to be stepped down a further 5m. This was followed by a second series of deep wells, finally allowing excavation to the required 15m.

Reference

Blackwater Valley Route
Surrey and Hampshire county councils' joint proposed dual carriageway bypass of Farnborough and Aldershot.

borrow pit
The borrow pit is needed because the 5km section requires some 600000m³ of class one fill for construction of the embankments. Importing this fill and carting away 150000m³ of unsuitable surplus material would be less efficient.

hole punching
The deep wells were installed with a hole puncher. High pressure water and air are injected through a central jetting pipe. This erodes the ground at the probe's base and flushes debris up the annulus between the jetting pipe and the outer casing. At the same time, a top-acting 3t drop weight drives the outer casing into the ground.

The pit needed to be 15m deep. Trials resulted in the sand boiling when the excavation extended below 6m. The situation was complicated by the permeability in the overlying gravel being ~0.001m/s, approximately 100 times greater than that in the Bracklesham Beds.

First stage
Well points were installed for pumping at the base of the gravels, around the perimeter of the proposed pit. A garland drain was constructed around the base of the excavation to stop overbleed from the gravels recharging the top of the Brackleshams.

The 300mm diameter deep wells were installed to a depth of 24m at 30m centres.

Case studies

Complex water control system
Munich airport, West Germany

Elaborate groundwater control measures are a key feature of Munich's new airport - one of the largest greenfield construction sites in Europe. Groundwater within 1m of the surface meant a risk of regular flooding and vulnerability to frost damage of the runways and taxiways. A system of ditches and drains has dropped the level, but environmental constraints dictated the water table should be restored outside the airport boundary. Restoration of diverted streams, and a multiple level system of dirty, contaminated and clean run off drainage is required. The solution inside the airport was to lower the groundwater table by around 2m. Two longitudinal ditches between the runways are drained by an underground pipe link to the main boundary interceptor. On the northern downstream boundary, the water is put back into the ground by a series of well point injectors. Control centre computers monitor the flow distribution and any contamination with spilt aircraft fuel etc. Glycol*(used as a de-icer on the taxiways* will be intercepted by 20m wide ridged aprons of impermeable geotextile buried under sand and gravel filters.

Reference

Geology
The subsoil is a 10m depth of Quaternary gravel overlying an impervious layer capping a 30m thick stratum of Tertiary gravels. The gravels each have separate groundwater movements of about 1m to 2m a day from south to north. They are valuable aquifers, and all flows need to be maintained.

The injectors are 10m deep steel diffuser pipes 200mm in diameter
<PBiological action should break down the glycol before it reaches the edge of the membrane and trickles down.

Case studies

Buoyancy-dependent foundations

Lecco bypass, near Milan, Italy

The Italian road authority ANAS' bypass includes twin bore 3.2km long tunnels passing underneath Monte Barro and approach viaducts of 200m and 950m at either end. At the longer of the two, the underlying limestone bedrock dips in a 100m deep glacial valley, now infilled with soft and loose lake deposits. A brave design concept was proposed, in which hydrostatic upthrust supports the viaduct's load.

While floating roads built on lightweight polystyrene blocks have been constructed over marshland, at Lecco the viaduct is to be built on 20 huge, jet grouted cylindrical cells, bedded 25m below the ground surface. Soil within each cell is excavated from the surface using a backhoe. The cells are lined with concrete and PVC, and a 1m thick cap completes the hollow space.

The foundation design is dependent almost entirely upon buoyancy, provided by the cells being water tight and remaining air filled. Yet water has leaked into some of the completed cells and permanent pumping may be necessary to keep some of the cells dry.

Reference

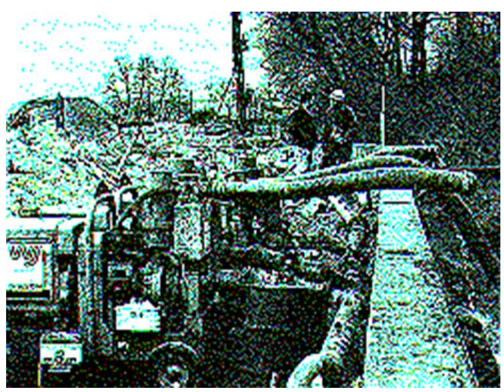
jet grouted cylindrical cells
The cells are formed by vertically drilling a 10m diameter ring of 31 overlapping 1.5m diameter jet grouted columns. Each column is reinforced by a steel pipe inserted into the soil-grout mix before it sets. A second outer ring of jet grouting is then constructed over the bottom 13m of the cell while a third phase of jet grouting forms a consolidated 5m deep plug between 20m and 25m depth.

A 300mm thick reinforced concrete wall is cast in place after every 2m of excavation until the top of the plug section has been reached. The base is then strengthened with a 2.5m thick reinforced concrete slab and a PVC geomembrane is installed to waterproof the sides of the cell. Then a second internal concrete wall is cast.

Case studies

Wellpoint pumping

Sevenoaks, Kent



In preparation for the construction of the piling mattress for a new superstore, subcontractors commenced dewatering to lower the water table within the fine silty sands of the Lower Greensand. The Gault clay dips down towards the northern end of the site, making dewatering unnecessary in this area, so the wellpoints were installed along the bank of the stream at the east boundary, and around the southern boundary. Groundwater in that area is almost at surface level.

120 wellpoints were installed, the aim being to dewater the Greensand down to 4m, allowing excavation for the construction of the pile mat. The wellpoints were connected to a pair of six-inch pumps which discharge directly into the stream. Initially, the pumps were discharging at a rate of 40 litre/sec, but after four days this had dropped to 10-15litre/sec. The Gault clay contained a number of perched water tables, and these were sump pumped during the first week of dewatering in order to drain them.

Reference

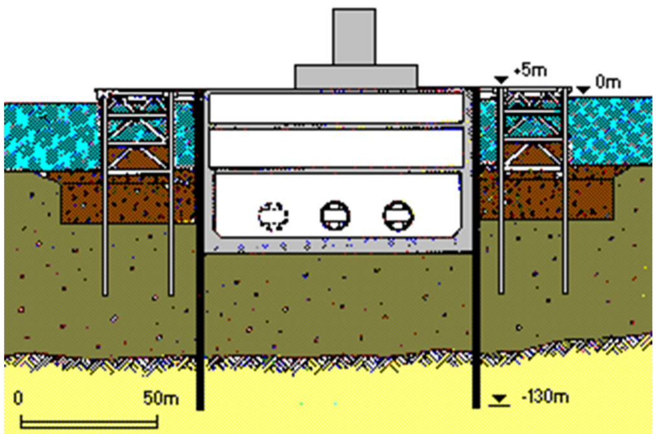
GEOLOGY
The geology consists of a layer of made ground overlying alluvium with some peat deriving from the time when the area was marshland. This overlies Gault clay and an extremely fine sand which represents the top of the Lower Greensand.

Site investigation
Site investigation consisted of eight boreholes and 16 trial pits, two of which were prevented from going to full depth because of the high groundwater inflows.

Case studies

Artificial island

Trans Tokyo Bay Highway, Japan



Record breaking diaphragm walls has been trenched 135m under Tokyo bay at the artificial island of Kawasaki - a key feature of the £5000M Trans Tokyo Bay Highway.(Note 1) The island will serve a dual role as the main pit for tunnel driving during construction, and later as a ventilator opening into the highway tunnel.

Loose marine sands have been stabilised by compaction from a barge before construction of the working platforms. Note 2 The support framework functions as part of a retaining wall when fill is placed to form a mounded island to sea level. The wall is supported and protected from erosion by rockfill armour on the outside.

A rig worked from the platform to trench out the 98m diameter, 135m deep ring of diaphragm wall panels, slicing down through the placed fill, then marine material, to form a cut off in hard rock. Once the wall was watertight, the interior was excavated to form a pit Note 3 reaching 70m below sea level. Tunnel drives then started from the dry chamber.

Reference

(Note 1)
The Yokohama Bay bridge carries the new road 860m across the water from Kisarazu. The rest of the distance to a second artificial island is spanned by a 4.4km low level viaduct. From there the road plunges beneath the sea carried through twin 9.1km tunnels to the mainland.

(Note 2)
A 200m diameter ring of steel trestle working platforms, founded on tubular steel piles driven into the treated material.

(Note 3)
Over 0.5M.m³ of fill and soft sediment will have to be dug out while in situ reinforced concrete supports are erected inside.

Case studies

Flooded tunnel and ground freeze

Thames Water ring main, London

More than twice the depth of London's deepest tube station, the tunnel was being driven for the Streatham to Brixton section of Thames Water's ring main. Groundwater pressures approaching 400kN/m² meant that compressed air fed into the workings to stem the flow broke through the legal limit of 345kN/m² for man working. The 2.5m diameter drive was started in dry London Clay before breaking into the underlying Woolwich & Reading beds. Saturated sand beds up to 0.5m thick needed more than the 100kN/m² air pressures predicted to keep the tunnel dry. Even raising the air to over 200kN/m² did not prevent water ingress.

The miners were probing to see how much Woolwich & Reading bed material protected them from the tricky Thanet Sands below, when the tunnel blew, letting 400kN/m² pressurised groundwater blast in, bringing with it more than 10m³ of sand and exposing the Chalk below. The volumes and rate of tunnel inflow, coupled with the fact that there is no sign of depressurising the Thanet Sands, indicates that the water is coming from the Chalk.

Fortunately, the tunneller was stranded below open ground, so a 50m vertical shaft can be sunk for machine recovery. The base of the 7m diameter shaft will be frozen to allow a short horizontal drive to the machine.

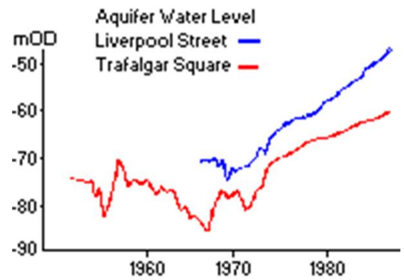
Reference

Case studies

Rising groundwater levels

- Future water levels
- Effects on buildings and tunnels

The deep aquifer beneath London



Many major cities obtain water by pumping from the ground. Changed industrial practices and water-supply systems over the years have led to much less water being taken from these sources. As a result, water levels which had been drawn down are now rising.

During the past two centuries, the pumping from the deep aquifer lowered the groundwater level by as much as 70m. The level is now rising, in many areas by about 1m/yr. If the rise continues for 20 to 30 years, the water pressures in the sands and clays above the Chalk will increase, causing ground movements in the clays. This could damage some large buildings and tunnels and increase leakage into them. These problems can be prevented by additional pumping, for a capital expenditure which is small compared with the potential cost of damage.

Reference

Deep aquifer
The water-bearing Basal Sands and Chalk below London are referred to as the deep aquifer. The aquifer is confined over much of the Basin by the overlying thick, relatively impermeable layers of clay, which separate the deep aquifer from the perched groundwater in the overlying gravels.

Future water levels
A regional groundwater computer model has been used to estimate the rates of rise likely to occur over the next 90 years, together with the likely maximum future groundwater levels. It shows that water levels could return almost to their original values within the next 30 to 40 years, unless action is taken.

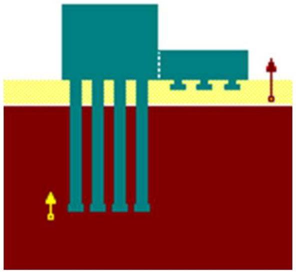
Additional pumping
Additional pumping of water from the aquifer in Central London not exceeding 30 mega-litres per day would be sufficient. This is about 5% of the normal daily consumption of water in Central London.

Capital Expenditure
The pumping would require about 30 wells at a capital cost of £10M to £30M.

Potential cost
The cumulative cost of remedial works for tunnels could exceed £10M. Repairs to large buildings could cost tens of millions of pounds. Costs of new construction would also increase.

Effects on buildings and tunnels

The potential effects of rising groundwater levels on buildings are:



- damage, perhaps instability caused by differential ground movements
- seepage into basements
- chemical attack on buried steel and concrete

Settlement or heave of a deep body of clay underlying a building would not necessarily damage it. Damage is more likely where the clay is thin and the foundations or basements are close to the aquifer. Broadly, this occurs in the lower-lying areas of Central London, east of Westminster.

Although older buildings are unlikely to be damaged, modern structural forms, such as those in the diagram, would be vulnerable. If water levels rise causing the clay to swell, the two parts of the building with differing foundation levels will heave by amounts which could differ by up to 100mm.

There are some 130km of tunnels under London which are located near the base of the London Clay or in the sandy deposits. Lengths totalling about 40km have been identified which could suffer increased seepage, loading and chemical attack.

Case studies

Simulation of rising groundwater

Canary Warf, London

Concern over rising groundwater at Canary Warf has led to the testing of a variable water pressure trial pipe: The piezometric head is being artificially increased in the Thanet sands pile founding strata by injecting water into five 150mm diameter wells. Bourdon gauges connected to three piezometers monitor changes in head during testing. The experiment models the change in pile base effective stress which is predicted if London ground water levels continue to rise, plus the effect of unloading under the 9m deep excavation for Founders Court car park. Trial excavation has been made redundant.

The total change in vertical effective stress at the pile base is equal to the sum of changes in total vertical stress and water pressure. The reduction in vertical effective stress is calculated to be about 180kPa, roughly equivalent to the 17m increase in water head induced by injection into Thanet Sands. This compares with a current vertical effective stress of 300kPa. The potential effect could be very serious if the factor of safety and design were inadequate.

Reference

[Case studies]

Flooded cofferdam

Ennerdale Link road, River Hull

The 2.5km dual carriageway road has been designed to relieve traffic congestion in eastern Hull and provide a direct link to the docks. Its planned route beneath the River Hull lies through an overall 361m long retained cutting, including a central 81m twin-celled concrete box section constructed in cut-and-cover through the river bed.

Original site investigation suggested a 4.3m deep layer of alluvium was underlain by a minimum 4m thick bank of impermeable boulder clay. This protected and confined a large aquifer in the chalk beneath it. The aim was to found the tunnel box in the clay zone without affecting the chalk. To construct the tunnel in two halves across the river, the contractor drove the first of two sheetpiled cofferdams on the eastern bank. With the cofferdam dewatered and being bottomed up, a 2m diameter sinkhole suddenly appeared in the river bed close to the cofferdam's north west corner, and a second hole formed nearby inside the structure. The excavation totally flooded overnight.

The cofferdam was quickly plugged with tremmied concrete and following a detailed three month second site investigation, urgent work started on jet grouting down the three exposed faces of the cofferdam, while the possible solutions were debated.

Reference

sheetpiled cofferdams
The 140m long, 9m deep cofferdam projected 12m into the diverted river. The 22m long piles extended down through the boulder clay to found well into the chalk beneath.

The second investigation
The second investigation revealed the boulder clay to be thinner than expected. It may contain perched water lenses and a prime concern is its integrity to protect the aquifer beneath. Water in the cofferdam has come from the river, not the aquifer, and it is possible that it has leaked through the sheetpiling rather than migrated beneath it.

Possibel solutions
Protecting the 21m wide tunnel from uplift hydrostatic pressure from the aquifer would need extensive mass concrete loading to the 1m thick tunnel floor. Ground anchors to hold the tunnel down are an option, but could puncture the aquifer.

The tunnel was eventually abandoned in favour of a lift bridge. In excess of £10m has been spent on the aborted work prior to starting on the bridge.