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Groundwater Engineering

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34.1 Fundamentals

Introduction

This chapter on groundwater engineering is concerned with the occurrence, movement, use and quality of water below ground. The section on fundamentals deals with the definitions, the properties of the unsaturated and saturated zones, and the physics of the movement of subsurface water. Specific engineering applications such as well hydraulics, well construction, contaminant transport, containment of contaminants, landfills, and geostatistics are discussed in the following sections.

Subsurface Water

The **water table** is the level at which the groundwater is at atmospheric pressure. The zone between the ground surface and the water table is called the **vadose zone**. It contains some water that is held between the soil particles by capillary forces. Immediately above the water table is the capillary fringe where the water fills the pores. The zone above the capillary fringe is often called the *unsaturated zone*. Below the capillary fringe is the *saturated zone*. The **saturation ratio** is the fraction of the volume of voids occupied by water. The water above the water table is below atmospheric pressure while the water below the water table is above atmospheric pressure. Only the water below the water table, the *groundwater*, is available to supply wells and springs. Recharge of the groundwater occurs primarily by percolation through the unsaturated zone. The geologic formations that yield water in usable quantities, to a well or a spring, are called **aquifers**. If the upper surface of the saturated zone in the aquifer is free to rise or to decline the aquifer is said to be an **unconfined aquifer**. The upper boundary at atmospheric pressure is the water table, also called the **phreatic surface**. If the water completely fills the formation the aquifer is **confined** and the saturated zone is the thickness of the aquifer. If the confining material is impermeable it is called an **aquiclude**. If the confining layer is somewhat permeable in the vertical direction, thus permitting slow recharge, it is called an **aquitard**. When a layer restricts downward infiltration towards the main water table, a *perched* aquifer with a separate **perched water** table may be formed. A perched aquifer is, in general, of limited areal extent, and if used as a water supply, extreme caution should be exerted because of its ephemeral nature. If the water in a well in a confined aquifer rises above the top of the aquifer, the water in the aquifer is under pressure, the well is called an **artesian well**, and the aquifer is in artesian condition. The *potentiometric surface,* also called the *piezometric surface* is defined as, the surface connecting the levels to which water will rise in several wells. If the piezometric surface is above the ground surface then a flowing well results.

Physical Properties

The *porosity*, n, is the ratio of the volume of voids, V_v, to the total volume, V_t, of the rock or soil:

$$n = V_v / V_t = \left[V_t - V_s\right] / V_t$$

where V_s is the volume of solids.

The *void ratio*, e, used in soil mechanics, is defined as $e = V_v/V_s$ so that $1/n = 1 + 1/e$. The fraction of void space between grains of soil or of unconsolidated rock is referred to as *primary porosity*. Porosity due to fracturing of the rock or chemical dissolution is called *secondary porosity*. Typical values of the porosity are given in the following Table 34.1. The *effective porosity*, n_e, is the pore fraction that actually contributes to the flow, isolated and dead end pores are excluded. In unconsolidated sediments coarser than 50 μm, n_e is of the order of 0.95 n to 0.98 n. When all the voids are occupied by water the soil is

TABLE 34.1 Values of Porosity, Permeability, and Hydraulic Conductivity

Material	Porosity n (%)	Permeability k cm^2	Hydraulic conductivity K cm/s
Unconsolidated deposit			
Gravel	25–40	10^{-3}–10^{-6}	10^{2}–10^{-1}
Sand	25–50	10^{-5}–10^{-9}	1–10^{-4}
Silt	35–50	10^{-8}–10^{-12}	10^{-4}–10^{-7}
Clay	40–70	10^{-12}–10^{-15}	10^{-7}–10^{-10}
Rocks			
Fractured basalt	5–50	10^{7}–10^{-11}	10^{-2}–10^{-6}
Karst limestone	5–50	10^{-5}–10^{-9}	1–10^{-4}
Sandstone	5–30	10^{-9}–10^{-13}	10^{-4}–10^{-8}
Limestone, dolomite	0–20	10^{-9}–10^{-12}	10^{-4}–10^{-7}
Shale	0–10	10^{-12}–10^{-16}	10^{-7}–10^{-11}
Fractured crystalline rock	0–10	10^{-7}–10^{-11}	10^{-2}–10^{-6}
Dense crystalline rock	0–5	10^{-13}–10^{-17}	10^{-8}–10^{-12}

saturated. Otherwise the fraction of the voids occupied by water is the volumetric *water content* designated by θ, which is dimensionless. When the soil is saturated, the soil moisture content is $\theta_s = n$. After the soil has been drained the remaining soil moisture is the *residual moisture content* θ_r. In unsaturated soils the *effective porosity* is $\theta_e = n - \theta_r$ and the *effective saturation* is defined as

$$s_e = (\theta - \theta_r)/(n - \theta_r)$$

The *hydraulic conductivity*, K, is a measure of the ability of water to flow through a porous medium. It is the volume rate of flow, Q, per unit gross area, A, of soil or rock under a hydraulic gradient $\partial h/\partial s$:

$$K = -(Q/A)(\partial h/\partial s)^{-1}$$

For *saturated flows* the hydraulic conductivity, K, depends on the porous medium through the *intrinsic permeability, k*, and on the fluid properties through the density, ρ, and the viscosity, μ. These properties are related by the following equation

$$K = k\rho g/\mu$$

so that a usual expression for k is

$$k = -(Q/A)(\mu/\rho g)(\partial h/\partial s)^{-1}$$

For spheres $k = C\,d^2$, where C is a constant, k has the dimension of L^2 and K has the units of L/T.$^{-1}$ Ranges of values of the permeability and hydraulic conductivity are given in Table 34.1. Several formulas exist in the literature that estimate the hydraulic conductivity of granular noncohesive materials. Most are of the form

$$K = (g/\nu)C\,\phi(n)\,d_e^2$$

where g = the acceleration of gravity
ν = the kinematic viscosity
C = a coefficient
$\phi(n)$ = a function of the porosity
d_e = the effective grain diameter, with the variables in a consistent set of units.

Vukovic and Soro (1992) list 10 formulas of this type. Two of the simplest formulas are the *Hazen formula* with $C = 6 \times 10^{-4}$, $\phi(n) = [1 + 10(n - 0.26)]$, $d_e = d_{10}$ which is applicable for 0.1 mm $< d_e <$ 3 mm and $d_{60}/d_{10} < 5$ and the *USBR formula* with $C = 4.8 \times 10^{-4}\, d_{20}^{0.3}$, $\phi(n) = 1$, $d_e = d_{20}$ and is applicable to medium sand grains with $d_{60}/d_{10} < 5$ where d_{10} is the particle size such that 10% are finer.

When the flow occurs horizontally through a series of n equally thick layers in parallel, of hydraulic conductivities $K_1, K_2, \ldots, K_n$, the equivalent hydraulic conductivity of the system of layers is the arithmetic average of the conductivities. When the flows occurs vertically through a stack of n equal layers in series, the equivalent hydraulic conductivity of the system of layers is the harmonic mean of the conductivities

$$K = \frac{n}{\dfrac{1}{K_1} + \dfrac{1}{K_2} + \cdots + \dfrac{1}{K_n}}$$

For an anisotropic material with horizontal and vertical hydraulic conductivities K_x and K_y, respectively, the hydraulic conductivity at an angle α with the horizontal, K_α, is obtained from

$$\frac{1}{K_\alpha} = \frac{\cos^2\alpha}{K_x} + \frac{\sin^2\alpha}{K_y}$$

For an *unsaturated condition* the hydraulic conductivity is a function of the moisture content of the soil and is designated by $K(\theta)$. When the soil is saturated the *saturated hydraulic conductivity* is designated by K_s. The ratio of the hydraulic conductivity for a given moisture content to the saturated conductivity is the *relative conductivity*, K_r. Brooks and Corey (1964) gave the following formula for the hydraulic conductivity of unsaturated porous materials

$$K(\theta) = K_s\left[(\theta - \theta_r)/(n - \theta_r)\right]^\lambda$$

where λ is an experimentally obtained coefficient.

Other formulas have been given by Campbell (1974) and van Genuchten (1980).

The *transmissivity*, T, is the product of the hydraulic conductivity and the thickness, b, of the aquifer: $T = K\,b$. It has the units of L^2T^{-1}. The *storage coefficient*, or *storativity*, S, is the volume of water yielded per unit area per unit drop of the piezometric surface. For unconfined aquifers the drop of the water table corresponds to a drainage of the pore space and the storage coefficient is also called the **specific yield**. In an unconfined aquifer the amount of water that can be stored per unit rise of the water table per unit area is called the *fillable porosity*, f, where $f = \theta_s - \theta$. In a confined aquifer, when the water pressure decreases the fluid expands and the fraction of the weight to be carried by the solid matrix increases, resulting in a decrease of the pore space. Since the compressibility of the water is very small its decompression contributes only a small fraction to the storage coefficient. The leakage of an overlying unconfined aquifer through an aquitard can also contribute to the yield of a semi-confined aquifer. Values of S typically vary between 5×10^{-2} and 10^{-5} for confined aquifers.

Darcy's Law

The *volumetric flow rate* Q $[L^3T^{-1}]$ across a gross area A of a formation with a hydraulic conductivity K $[LT^{-1}]$ under a hydraulic gradient $\partial h/\partial s$ in the s direction is given by Darcy's law

$$Q = -K\,A\,\partial h/\partial s = q\,A$$

where q is the *specific discharge* or flow rate per unit area $[LT^{-1}]$ also called *Darcy velocity*. The *hydraulic head* h is the sum of the elevation head z and the pressure head p/γ. The minus sign in the above equation indicates that the flow takes place from high to low head, namely in the direction of the decreasing head. The *pore velocity*, $v = q/n_e$, is the average flow velocity in the pores or the average velocity of transport of solutes that are non reactive.

The one-dimensional form of Darcy's law in a homogeneous medium of conductivity K is

$$q = K\left[(p_1/\gamma + z_1) - (p_2/\gamma + z_2)\right]/L$$

where the subscripts 1 and 2 refer to the two points at which the pressure head and the elevation head are considered and L is the distance between these points.

Darcy's law implies that the flow is laminar as is generally the case. However in some cases, as in karstic limestone and in rocks with large fractures, the flow may be turbulent. In such cases the flow rate is not proportional to the hydraulic gradient but to a power of the hydraulic gradient. Darcy's law as given above applies to *isotropic* media, that is, where the hydraulic conductivity is independent of direction. It also applies to flows where the direction of the hydraulic conductivity corresponds to the direction of the hydraulic gradient. In *non isotropic* media the hydraulic conductivity depends upon the direction. Then a hydraulic conductivity *tensor* is used and Darcy's law is expressed as a *tensor equation* (de Marsily, 1986).

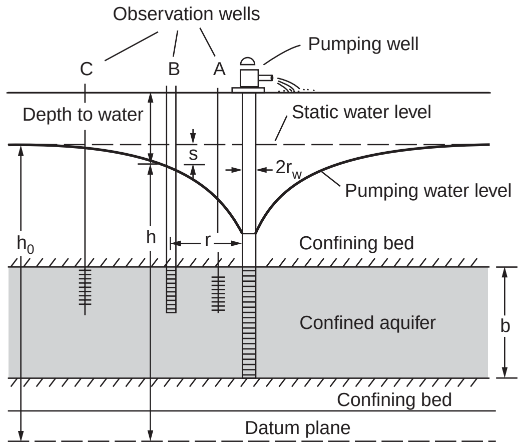

FIGURE 34.1 Well in a confined aquifer. (*Source*: Heath, R.C. 1998. *Basic Ground-Water Hydrology.* U.S. Geological Survey Water Supply Paper 2220, U.S. Government Printing Office.)

Dupuit Assumption

The one-dimensional form of Darcy's law applies to simple flow problems in the vertical or horizontal direction. In some cases with both horizontal and vertical components, the horizontal component dominates and the vertical component can be neglected. The flow can then be approximated as a horizontal flow uniform across the depth. This is the Dupuit assumption also sometimes referred to as the *Dupuit-Forchheimer* assumption.

34.2 Hydraulics of Wells

Steady Flow to a Well

The steady flow to a well of radius r_w fully penetrating a *confined* aquifer (Fig. 34.1) with a transmissivity T is given by the *Thiem equation*

$$Q = 2\pi T\left[h - h_w\right] / \ln\left(r/r_w\right)$$

where h = the hydraulic head at a distance r
h_w = the hydraulic head at the well

For a well fully penetrating an unconfined aquifer (Fig. 34.2) the equation for the flow rate obtained using the Dupuit assumption and neglecting the seepage face is:

$$Q = \pi K\left[h^2 - h_w^2\right] / \ln\left(r/r_w\right)$$

When solved for the head at the well, this equation does not yield accurate results because of the neglect of the vertical flow component.

Transient Flow to a Well

Pumping a well causes a *cone of depression*, or *drawdown*, of the water table of an unconfined aquifer or of the piezometric surface for a confined aquifer. The drawdown $s(r,t)$ at a distance r from a fully penetrating well at time t after the beginning of pumping at a constant rate Q from a confined aquifer with transmissivity T and storage constant S is given by the *Theis equation*:

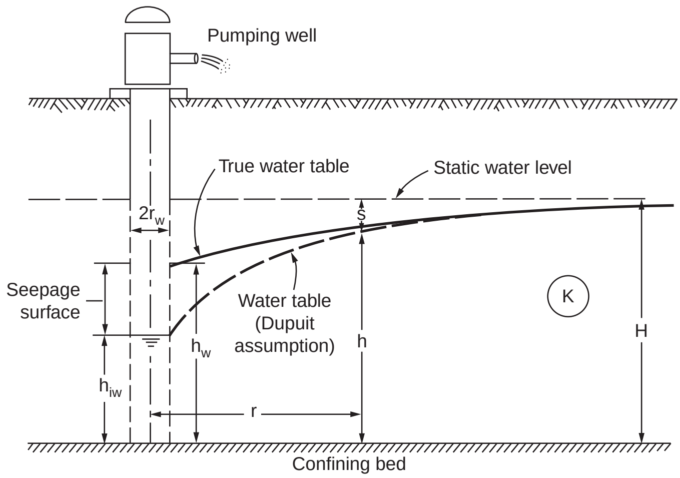

FIGURE 34.2 Well in a unconfined aquifer. (*Source:* Bouwer, H., 1978. *Ground Water Hydrology.* McGraw-Hill, New York.)

$$s(r,t) = \left[Q/(4\pi T)\right] \int_u^\infty \left(e^{-z}/z\right) dz = \left[Q/(4\pi T)\right] W(u)$$

where $u = r^2 S/(4Tt)$.

The integral in this equation is the exponential integral also known as the *well function* $W(u)$. This function can be expanded as

$$W(u) = -0.577216 - \ln u + u - \frac{u^2}{2 \times 2!} + \frac{u^3}{3 \times 3!} - \cdots$$

For $u < 0.01$ only the first two terms need to be considered and the drawdown is approximated by Jacob's equation

$$s(r,t) = \left[2.30\, Q/(4\pi T)\right] \log_{10} \left[(2.25 T t)/\left(r^2 S\right)\right]$$

The drawdown from a unconfined aquifer with horizontal and vertical hydraulic conductivities K_r and K_z, respectively, has been given by Neuman (1975). A simplified form, given by Freeze and Cherry (1979), is

$$s(r,t) = \left[Q/(4\pi T)\right] W(u_a, u_b, \eta)$$

where

$$u_a = r^2 S/(4Tt), \quad u_b = r^2 S_y/(4Tt) \quad \eta = r^2 K_z/\left(b^2 K_r\right),$$

where b = the initial saturated thickness
S_y = the specific yield
S = the elastic storage coefficient

This solution is valid for $S_y \gg S$. Freeze and Cherry (1979) give a plot of the well function $W(u_a, u_b, \eta)$.

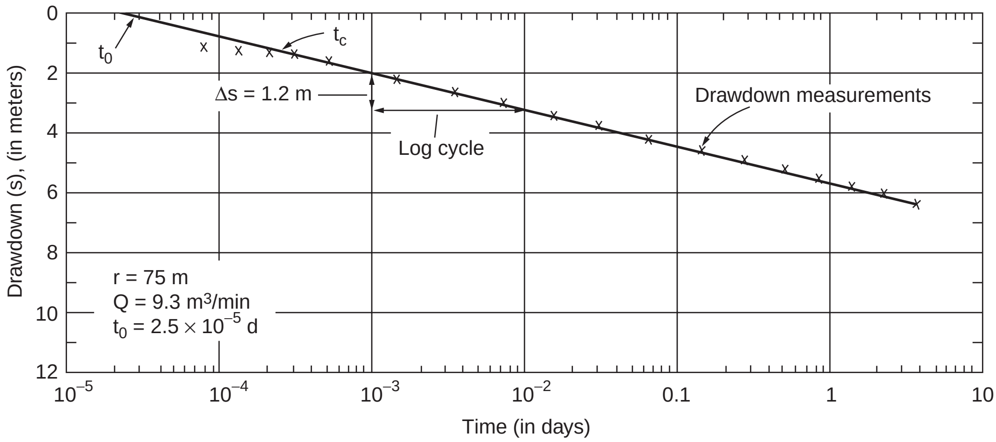

FIGURE 34.3 Time drawdown analysis (*Source*: Heath, R.C., 1998. *Basic Ground-Water Hydrology.*U.S. Geological Survey, Water Supply Paper 2220, U.S. Government Printing Office.)

For a pumped leaky confined aquifer with constants T and S, separated from an unpumped upper aquifer by an aquitard of thickness b' and constants K' and S', Hantush and Jacob (1955) obtained a relationship for the drawdown which can be written as

$$s(r,t) = \left[Q/(4T)\right] W\left(u, r/B\right)$$

where

$$u = \left(r^2 S\right)/\left(4Tt\right) \quad \text{and} \quad r/B = r\left[K'/\left(Tb'\right)\right]^{1/2}$$

Values of W (u, r/B) can be found in Bouwer (1978), Freeze and Cherry (1979) and Fetter (2001).

Pumping Tests

The hydraulic properties of aquifers can be determined by pumping a well at constant discharge and observing the drawdown at one or more observation wells for a period of time. For confined aquifers the Thiem *steady state* equation yields only the transmissivity

$$T = \left[Q \ln\left(r_2/r_1\right)\right]/\left[2\pi\left(s_1 - s_2\right)\right]$$

from the observed drawdowns s_1 and s_2 at distances r_1 and r_2 from the pumped well.

For confined aquifers the transient state Jacobs equation yields both the transmissivity and the storage constant based on a semi-log straight line plot (Fig. 34.3) of the observed drawdown (arithmetic scale) versus the time since pumping began (logarithmic scale) as

$$T = 2.3\, Q/\left(4\pi\Delta s\right) \qquad S = 2.25\, T t_o/r^2$$

where Δs = the increase in drawdown per log cycle of t
t_o = the time intercept of the straight line fitted through the drawdowns at the several times.

Only the observations corresponding to very small times violate Jacobs assumption that $u = r^2S/(4Tt) < 0.01$ and do not fall on the straight line. This approach is known as *time-drawdown* analysis. If simultaneous drawdown observations are taken at different distances, then the *distance drawdown* analysis can

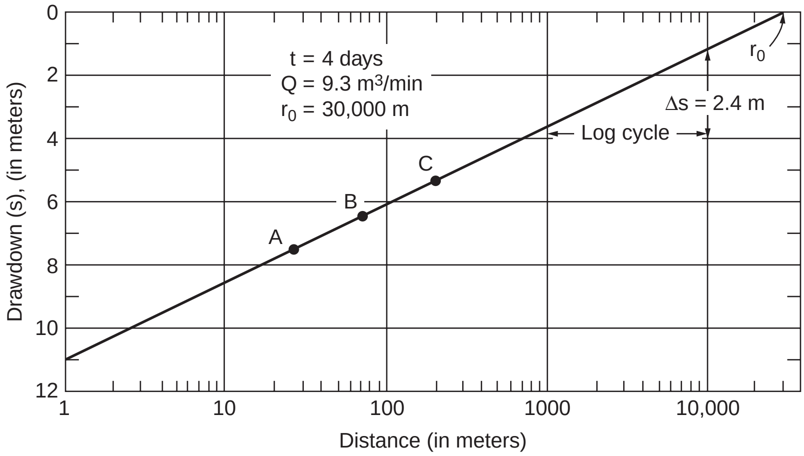

FIGURE 34.4 Distance-drawdown analysis. (*Source*: Heath, R.C. 1998. *Basic Ground-Water Hydrology.* U.S. Geological Survey, Water Supply Paper 2220, U.S. Government Printing Office.)

be used. In this latter approach a semi-logarithmic plot of the drawdown (arithmetic scale) vs. the distance r from the well (logarithmic scale) is used (Fig. 34.4) and the aquifer constants are given by

$$T = 2.3\,Q/(2\pi\Delta s) \qquad S = 2.25\,T\,t/r_0^2$$

where Δs = the drawdown across one log cycle of r
t = the time at which the drawdowns were measured
r_o = the distance intercept of the straight line fitted through the drawdowns at several distances.

Application of the Theis equation and solutions for unconfined aquifers require more elaborate graphic solutions (Bouwer, 1978; Fetter, 2001; Freeze and Cherry, 1979) or computer solutions (Boonstra, 1989, Kasenow and Pare, 1996).

The well test can also be performed using only the drawdown measurements at the pumped well without observation wells. This type of test is called the *single well test*. The previous equations assume laminar flow and a linear relationship between drawdown through the geologic formation and discharge. As the flow reaches the gravel pack around the well screen the velocity increases and the flow becomes turbulent except for very small pumping rates. The total drawdown at the well s_t is thus the sum of the formation drawdown s and the *well loss* s_w (Walton, 1962).

$$s_t = s + s_w = BQ + CQ^2$$

where the constant C is related to well characteristics.

If the well is pumped at different rates for the same length of time, a plot can be prepared of the total drawdown versus discharge. A tangent at the origin will separate the formation drawdown and the well loss. The time-drawdown plot is then performed for a constant discharge and the transmissivity is determined as before. A line is plotted parallel to the straight line portion of the time-drawdown observations at a distance s_w above the observations. This line determines the time intercept t_o used in the relationship for the storage constant S, (Fig. 34.5). For more details on well hydraulics and well tests see, for example, Boonstra (1999a).

Multiple Wells and Boundaries

For multiple wells the total drawdown s_t is the sum of the drawdowns due to the individual wells. For the case of a confined aquifer the Theis formula yields:

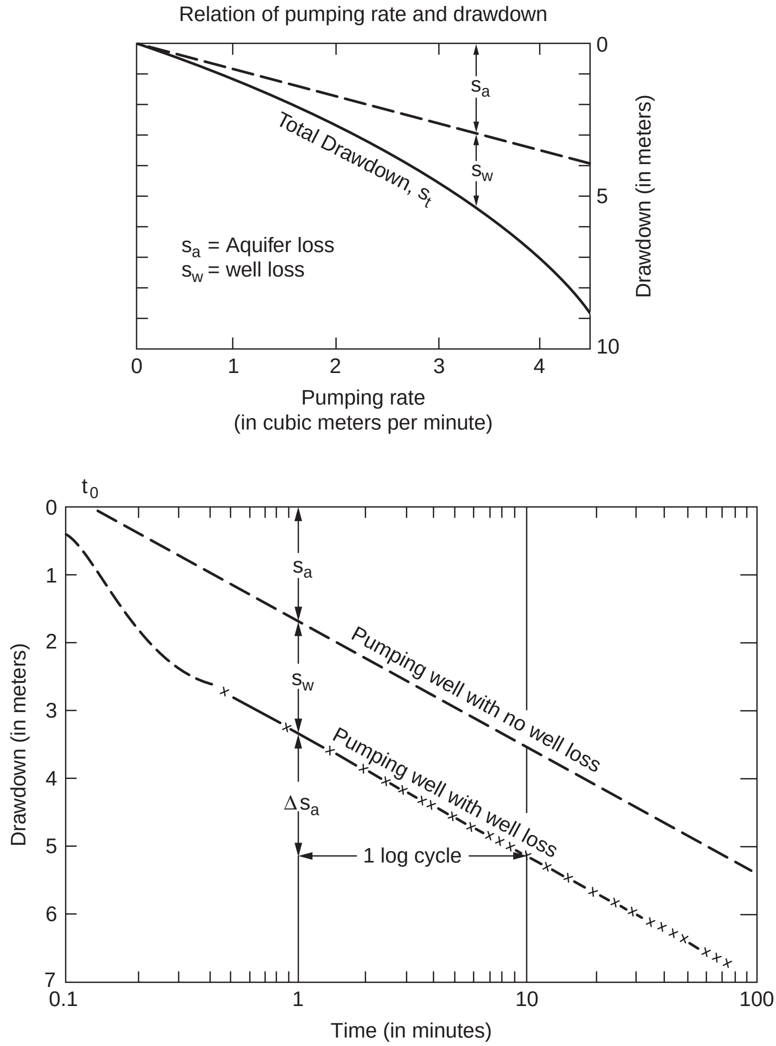

FIGURE 34.5 Single well test. (*Source*: Heath, R.C., 1998. *Basic Ground-Water Hydrology.* U.S. Geological Survey Water Supply Paper 2220, U.S. Government Printing Office.)

$$s_t = (4\pi T)^{-1}\left[Q_1 W(u_1) + Q_2 W(u_2) + \ldots\right]$$

where $u_i = r_i^2 S/(4\,Tt_i)$ in which r_i is the distance from the i th pumping well to the observation point
t_i = the time since pumping began at well i with a discharge Q_i

In the case of a pumping well and a *recharge well*, the change in the piezometric surface is the algebraic sum of the drawdown due to the pumped well and the buildup due to the recharge well. *Well interference* is an important matter in the design of well fields.

A recharge boundary or an impervious boundary within the cone of depression modifies the shape of the drawdown curve. The effect of a *perennial stream* close to a pumped well can be analyzed by considering an *image* recharge well operated at the same flow rate. The drawdown is the algebraic sum of the drawdowns due to the pumped and the image recharge wells. The resulting water level is constrained by the water surface elevation in the perennial stream. Similarly the effect of a vertical *impervious boundary* near a pumped well can be analyzed by considering an image pumping well operating at the same discharge. The resulting drawdown is the sum of the drawdowns. The resultant water level is seen to be perpendicular to the vertical impervious boundary. This horizontal surface has no gradient at the boundary thus indicating that there is no flow, which is consistent with the requirement of an impervious boundary. More elaborate boundaries can be simulated by the method of images.

34.3 Well Design and Construction

Well Design

Well design includes the selection of the well diameter, total depth of the well, screen or open hole sections, gravel pack thickness and method of construction. The pumping rate determines the pump size, which in turn determines the well diameter. Well pump manufacturers provide information on the optimum well diameter and size of pump bowls for several anticipated well yields.

Generally water enters a well through a wire screen or a louvered or shuttered perforated casing. The screen diameter is selected so that the entrance velocity of the water does not exceed 0.1 ft/s (0.03m/s). Dividing the design discharge by this velocity gives the required open area of the screen. A safety factor of 1.5 to 2.0 is applied to this area to account for the fact that part of the screen may be blocked by gravel packed material. The manufacturers supply the open areas of screen per lineal foot for different slot sizes and screen diameters. The required length of the screen is obtained by dividing the required area by the open area per lineal foot. Screens are normally installed in the middle 70% to 80% of confined aquifers and the lower 30% to 40% of unconfined aquifers.

A gravel envelope or gravel pack (Fig. 34.6) is used around the screen to prevent fine material from entering the well. Gravel packs make it possible to use larger screen slots thus reducing the well loss. They also increase the effective radius of the well. Gravel packs are used in fine textured aquifers in which D_{90} (the sieve size retaining 90% of the material) is less than 0.25 mm and has a coefficient of uniformity $D_{40}/D_{90} < 3$. Wells dug through multiple layers of sand and clay are generally constructed with a gravel pack (Fig. 34.6). A sand bridge is usually provided at the top of the gravel pack to separate it from the impermeable grout that extends to the surface. The well casing should extend somewhat higher than the ground level and a concrete slab sloping away from the well is provided to prevent surface runoff from entering into the well.

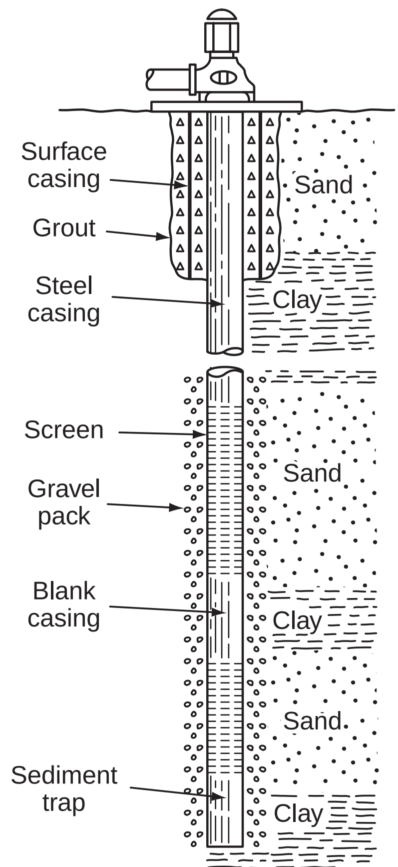

FIGURE 34.6 Supply well with multiple screen and gravel pack. (*Source*, Heath, R.C., 1998. *Basic Ground-Water Hydrology.* U.S. Geological Survey Water Supply Paper 2220, U.S. Government Printing Office.)

Construction Methods

The principal methods of well construction include digging, boring, driving, jetting, percussion drilling, hydraulic rotary drilling, and air rotary drilling. Table 34.2 indicates the suitability of the several well construction methods according to geologic conditions.

After the construction is completed the well is developed, stimulated, and sterilized. The removal of fine sand and construction mud is called *well development*. This can be accomplished by water or air

TABLE 34.2 Suitability of Different Well Construction Methods to Geologic Conditions

					Drilled		
					Percussion	Rotary	
Characteristics	Dug	Bored	Driven	Jetted	(cable tool)	Hydraulic	Air
Maximum practical depth, in m (ft)	15 (50)	30 (100)	15 (50)	30 (100)	300 (1000)	300 (1000)	250 (800)
Range in diameter, in cm (in.)	1–6m (3–20 ft)	5–75 (2–30)	3–6 (1–2)	5–30 (2–12)	10–46 (4–18)	10–61 (4–24)	10–25 (4–10)
Unconsolidated material:							
Silt	X	X	X	X	X	X	
Sand	X	X	X	X	X	X	
Gravel	X	X			X	X	
Glacial till	X	X			X	X	
Shell and limestone	X	X		X	X	X	
Consolidated material:							
Cemented gravel	X				X	X	X
Sandstone					X	X	X
Limestone					X	X	X
Shale					X	X	X
Igneous and metamorphic rocks					X	X	X

Source: Heath, R.C., 1998. *Basic Ground-Water Hydrology.*U.S. Geological Survey Water Supply Paper 2220, U.S. Government Printing Office.

surging. *Stimulation* is a technique to increase the well production by loosening consolidated material around the well. For example, high-pressure liquid can be injected into the well to increase the size of the fractures in the rock surrounding the well. Finally, water supply wells should be *sterilized*. This can be done with chlorine or other disinfectant.

Centrifugal pumps are normally used for water supply wells. If the water level in the well is *below* the center line of the pump it is necessary to check that the available positive suction head exceeds the required positive suction head specified by the manufacturer in order to avoid cavitation. For high heads *multiple stage pumps* are used. *Submersible pumps* avoid the need of long shafts and are used for very deep wells. Drillers keep **well logs** or records of the geological formation encountered.

For further details about well drilling methods, well design, well screens, well pumps and their maintenance the reader is referred to the works of Driscoll (1986) and Boonstra (1999 b).

34.4 Land Subsidence

Introduction

Groundwater pumping causes a downward movement of the water table or of the piezometric surface which in turn can cause a downward movement of the land surface called subsidence or consolidation. This movement can be a few centimeters to several meters. If the subsidence is not uniform, the differential settlement can produce severe damage to structures.

Calculation of Subsidence

Consider a unit area of a horizontal plane at a depth Z below the ground surface. The total downward pressure P_t due to the weight of the overburden on the plane is resisted partly by the upward hydrostatic pressure P_h and partly by the *intergranular pressure* P_i exerted between the grains of the material:

$$P_t = P_h + P_i \quad \text{or} \quad P_i = P_t - P_h$$

A lowering of the water table results in a decrease of the hydrostatic pressure and a corresponding increase in the intergranular pressure. If P_{i1} and P_{i2} denote the intergranular pressures before and after a drop in the water table or piezometric surface, the subsidence S_u can be calculated as

$$S_u = Z\left[P_{i1} - P_{i2}\right]/E$$

where E is the modulus of elasticity of the soil.

If there are layers of different soil types, the subsidences are calculated for each layer and added to yield the total subsidence. As the modulus of elasticity of clayey materials is much less than that of sands and gravel, most of the settlement takes place in the clayey layers.

The previous equation can also be used to calculate the rebound when the intergranular pressure decreases. Caution must be exercised because the modulus of elasticity usually is not the same for decompression as for compression. This is particularly the case for clays. For Boston blue clay the rebound modulus of elasticity is only about 50% of that for compression (Bouwer,1978, p 323). If subsidence has occurred for a long time, complete rebound is unlikely to occur.

If there is an upward vertical flow, the head loss due to friction as the water flows in the pores results in an increase in the hydrostatic pressure. This in turn results in a decrease in the intergranular pressure. A condition known as *quicksand* is reached when the intergranular pressure vanishes and the sand loses its bearing capacity. Horizontal flow of the ground water can cause a lateral displacement that can result in damage to wells.

34.5 Contaminant Transport

Introduction

This section describes the transport and fate of constituents in groundwater. *Constituent* is a general term that does not necessarily imply a polluting substance. The *fate* of a constituent depends on its transport through the groundwater and includes possible decay or reactions that may occur. The nature, behavior and physicochemical characteristics of important groundwater contaminants have been described by Blatchley and Thompson (1999). Contaminant transport can occur by **advection, diffusion,** and dispersion. Transport of solutes can be accompanied by chemical processes such as precipitation, dissolution, **sorption**, radioactive decay and biochemical processes such as biodegradation.

Advection

Advection is the movement of a constituent as a result of the flow of the groundwater. It is the most important mechanism of solute transport. The average flow velocity in the pores v is obtained by dividing the Darcy or gross velocity by the effective porosity n_e or $v_x = -[K/n_e]/(\partial h/\partial x)$, where K is the hydraulic conductivity and $\partial h/\partial x$ is the hydraulic gradient. The one-dimensional differential equation for the advection transport in the x direction is given by:

$$\partial C/\partial t = -v_x\, \partial C/\partial x$$

where C is the solute concentration (M/L^3).

Diffusion and Dispersion

Diffusion is the spreading of the solute due to molecular activity. The mass flux of solute per unit area per unit time F is given by Fick's law; $F = -D_d\ (\partial C/\partial x)$, where C is the solute concentration (M/L^3) and D_d is the diffusion coefficient (L^2/T). For solutes in an infinite medium this coefficient is of the order of 10^{-9} to 2×10^{-9} m^2/s at 20° C and varies slightly with temperature. The diffusion coefficient in a porous medium is reduced by a factor of 0.1 to 0.7 for clays and sands, respectively (de Marsily, 1986, p 233) because of the *tortuosity* of the flow paths, and is designated by D^*. The variability of the pore sizes, the multiplicity of flow paths of different lengths and the variation of the velocity distribution in the pores of different sizes result in a mechanical spreading known as dispersion. The *longitudinal* dispersion in the flow direction is larger than the *transverse* dispersion perpendicular to the flow direction. When advection and dispersion are the dominant transport mechanisms diffusion is a second order effect.

Molecular diffusion and **mechanical dispersion** are grouped under the term **hydrodynamic dispersion**. The longitudinal and transverse dispersion coefficients D_L and D_T, respectively, are given by

$$D_L = \alpha_L v_x + D^*$$

$$D_T = \alpha_T v_x + D^*.$$

where α_L and α_T = the longitudinal and transverse dynamic dispersivity
D^* = the molecular diffusion coefficient in the porous medium

The relative importance of the dynamic dispersivity and the molecular diffusion can be determined from the value of the *Peclet* number. It is defined as $P_e = v_x L/D_d$ where L is a characteristic length of the porous medium, generally taken as the mean diameter of the grains or the pores. The longitudinal advective dispersion dominates over the molecular diffusion when $P_e > 10$ and the transverse advective dispersion dominates when $P_e > 100$. The dispersion coefficients α_L and α_T are known to vary with the scale at which they are measured. Fetter (1999, p 80–86), as a first approximation, suggested the regression equation $\alpha_L = 0.1\ x$ where x is the flow distance. Other expressions can be found in the literature. For example, for the dispersivities measured in the field, called apparent dispersivities and designated by α_m, Neuman (1990) proposed $\alpha_m = 0.0175 L_s^{1.46}$ (both α_m and L_s are in meters) for travel distances L_s less than 3500 m and Xu and Eckstein (1995) proposed $\alpha_m = 0.83(\log L_s)^{2.414}$ (both α_m and L_s are in meters). This latter equation does not have the distance restriction that the Newman equation has.

The two-dimensional diffusion-dispersion in a flow in the x direction in an homogeneous aquifer is governed by the equation:

$$\partial C/\partial t = D_L\ \partial^2 C/\partial x^2 + D_T\ \partial^2 C/\partial y^2 - v_x\ \partial C/\partial x$$

Sorption

This discussion is limited to the cases of *adsorption* when the solute in the groundwater becomes attached to the surface of the porous medium and cation exchange when positively charged ions in the solute are attracted by negatively charged clay particles. The relationships relating the solute concentration C of a substance to the amount of that substance per unit mass in the solid phase, F, are called *isotherms* because they are determined at constant temperature. The simplest is the linear isotherm $F = K_d\ C$ where K_d is the distribution coefficient. Nonlinear isotherms have been proposed.

The effect of the adsorption is to retard the transport of the substance. The resulting one-dimensional advection-diffusion-dispersion equation for a flow in the x direction in an homogeneous aquifer is:

$$R\ \partial C/\partial t = D_L\ \partial^2 C/\partial x^2 - v_x\ \partial C/\partial x$$

where $R = 1 + K_d\ \rho_b/n_e$ is the *retardation factor* in which n_e is the effective porosity
ρ_b = the bulk density of the porous medium

Figure 34.7 shows a chemical spill with several constituents at similar concentrations and different retardation factors. Constituent 3 with $R = 3$ has more affinity for the soil matrix than constituents 2 or 1 with R values of 2 or 1, respectively. Thus constituent 3 will spend more time in association with the soil matrix than constituents 2 or 1. Contaminants with lower retardation factors are transported over greater distances over a given time period than contaminant with larger retardation factors. As a result, a monitoring well network has a greater chance of encountering contaminants with low retardation factors because they occupy a greater volume of the aquifer.

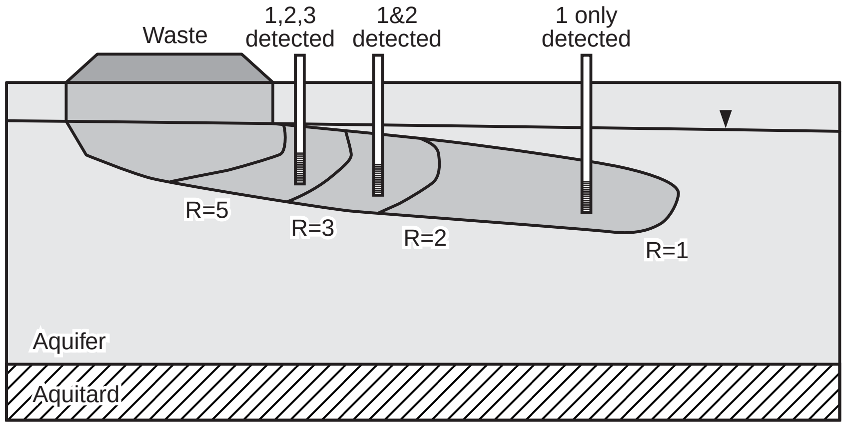

FIGURE 34.7 Transport of contaminants with several retardation factors at a waste site. (*Source:* U.S. Environmental protection Agency. 1989. *Transport and Fate of Contaminants in the Subsurface,* EPA/625/4–89/019.)

In the case of organic compounds the partition coefficient is $K_d = K_{oc} f_{oc}$ where K_{oc} is the partition coefficient with respect to organic carbon and f_{oc} is the fraction of organic carbon. A number of regression equations have been obtained that relate K_{oc} to the octanol-water partition coefficient and to the aqueous solubility (de Marsily, 1986, Fetter, 1999).

Multiphase Flow

Liquids that are not miscible with water are called **nonaqueous phase liquids** (**NAPL**). In the unsaturated zone four phases may be present: soil, water, air and NAPL. Many contaminant problems are associated with the movement of NAPL. The NAPL can have densities that are less than that of water and are called *light nonaqueous phase liquids* (**LNAPL**) or they can have densities that are larger than that of water and are called dense nonaqueous phase liquids (**DNAPL**). In an unconfined aquifer a LNAPL will tend to float near the water table whereas a DNAPL will tend to sink to the bottom of the aquifer. Figure 34.8 shows a schematic illustration of the behavior of LNAPL compounds. Under some conditions (a), the mass of an LNAPL spill is insufficient to allow penetration to the capillary fringe. With additional compound introduction (b), the LNAPL product will reach the water table and begin to spread, though the compound will not penetrate far beyond the phreatic surface. If the source of LNAPL is eliminated (c), removal of the NAPL will allow "rebound" of the water table. Figure 34.9 shows a schematic illustration of the behavior of DNAPL compounds. Under some conditions (A), the mass of DNAPL spill is insufficient to allow penetration to the capillary fringe; vertical movement of the DNAPL is by viscous fingering. With additional compound introduction (B,C), the DNAPL product will reach the water table and continue to move vertically until it reaches an impermeable boundary.

When two liquids compete for the pore space one will preferentially spread over the grain surface and wet it. The **wettability** depends upon the interfacial tension between the two fluids. In the case of oil-and-water systems, water is the wetting fluid in the saturated zone but in the unsaturated zone oil is the wetting fluid if the soil is very dry. The *relative permeability* is the ratio of the permeability of a fluid at a given saturation to the intrinsic permeability of the rock k. The relative permeability of the wetting fluid is designated by k_{rw} and that of the nonwetting fluid is k_{rnw}. For two phase flow Darcy's laws for the wetting and nonwetting liquids are respectively

$$Q_w = -\left[k_{rw}\, k \rho_w / \mu_w\right] A\, \partial h_w / \partial s$$

$$Q_{nw} = -\left[k_{rnw}\, k \rho_{nw} / \mu_{nw}\right] A\, \partial h_{nw} / \partial s$$

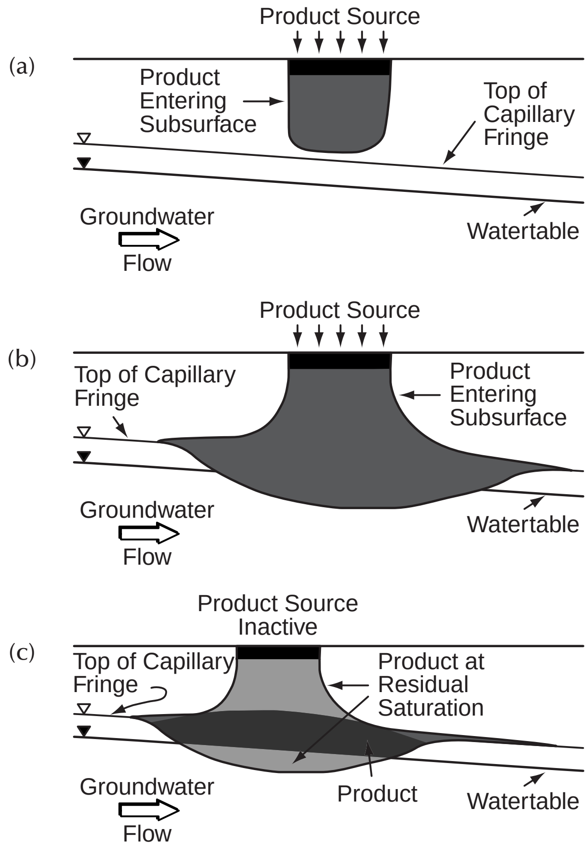

FIGURE 34.8 Movement of LNAPL into the subsurface (a) distribution of LNAPL after a small volume has been spilled; (b) depression of the capillary fringe and water table; (c) rebound of the water table as the LNAPL drains from overlying pore space. (*Source*: U.S. Environmental protection Agency. 1989. *Transport and Fate of Contaminants in the Subsurface,* EPA/625/4–89/019.)

where the subscripts w and nw refer to the wetting and nonwetting fluids, respectively, μ is the viscosity and A is the cross sectional area of the flow.

If an LNAPL (e.g., oil) is spilled on the ground it will infiltrate, move vertically in the vadose zone and, if sufficient quantity is available, eventually it will reach the top of the capillary fringe. Some *residual* NAPL remains in the vadose zone. As the NAPL (oil) accumulates over the capillary zone an *oil table* (oil surface at atmospheric pressure) forms and the water capillary fringe becomes thinner and eventually completely disappears. The oil table then rests on the water table (Abdul, 1988). The mobile oil below the oil table moves downward along the slope of the water capillary fringe. Soluble constituents of the LNAPL are dissolved in the ground water and are transported by advection and diffusion close to the water table. The residual NAPL left behind in the vadose zone partitions into vapor and liquid phases depending upon the degree of volatility and of water solubility. The thickness of LNAPL in a monitoring well is larger than that of free LNAPL in the subsurface.

If a DNAPL (e.g., chlorinated hydrocarbon) spills on the ground surface, under the force of gravity, it migrates through the vadose zone and through the saturated zone, eventually reaching an impervious layer. A layer of DNAPL then accumulates over the impervious layer. The mobile DNAPL then migrates along the slope of the impervious surface, which does not necessarily coincide with the slope of the water table and the direction of the ground flow. Monitoring wells placed just at the top of the impervious layer will show the presence of the DNAPL at the bottom of the well. If the well extends into the impervious

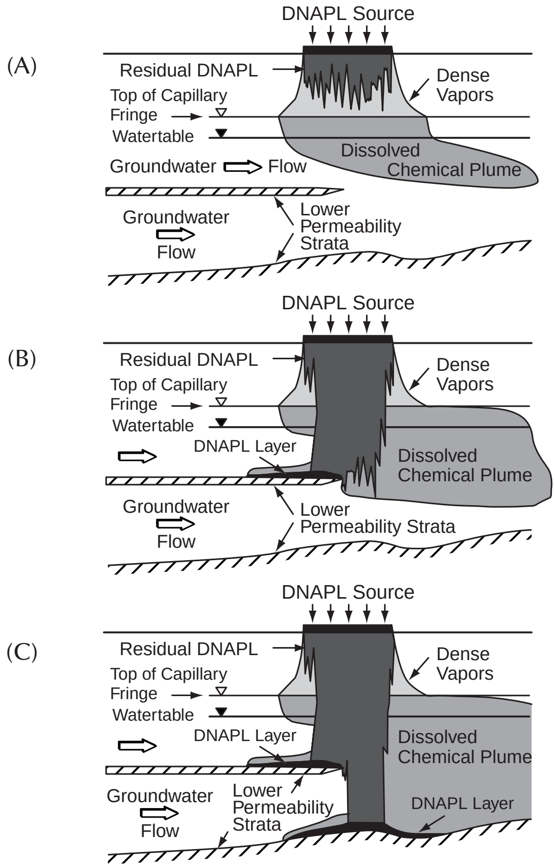

FIGURE 34.9 Movement of DNAPL into the subsurface : distributions of DNAPL after a small (A), moderate (B), and large (C) volumes have been spilled. (*Source*: U.S. Environmental Protection Agency. 1989. *Transport and Fate of Contaminants in the Subsurface,* EPA/625/4–89/019.)

layer the DNAPL will also fill that portion of the monitoring well below the impervious surface that acts as a sump.

34.6 Remediation

Monitoring Wells

Before any site remediation work is undertaken it is necessary to explore the aquifer and the extent of the ground water contamination. Monitoring wells are used principally for measuring the elevation of the water table or of the piezometric level, to collect water samples for chemical analysis and eventually observe the presence of nonaqueous phase liquids (lighter or denser than water) and to collect samples of these nonaqueous phase liquids. The equipment and supplies must be decontaminated before they are used in a water quality monitoring well.

If the purpose of the well is for observation of the water elevation only, a 1-in. casing is adequate. If water samples are required a 2-in. casing is necessary. Screens are used to allow the water into the well. In unconfined aquifers the screens must be placed so that they extend approximately from 5 ft. above the expected high water table to 5 ft below the expected low water table level. **Piezometer** screens for confined aquifers are shorter and generally have a length of 2 to 5 ft. The screen is surrounded by a *filter* *pack* consisting of medium to coarse silica sand. The filter pack extends about 2 ft above the screen. A seal is placed on top of the filter pack. It consists of a 2-ft layer of fine sand and an optional 2-ft layer of granular bentonite for further sealing. If there is a leachate plume several wells with different depths and screen lengths may be necessary to intercept the plume. Multilevel sampling devices that are installed in a single casing have been developed.

Monitoring of the water quality in the vadose zone can be accomplished with *lysimeters,* which are installed in a bore hole above the water table. The lysimeter consists principally of a porous cup mounted at the lower end of a tube with a stopper at the upper end. As the soil water pressure is below atmospheric, suction must be applied so that the water penetrates the porous cup. The water accumulated in the porous cup is then pumped into a flask at ground level.

For more details on groundwater monitoring, see, for example, Houlihan and Lucia (1999a).

Removal and Containment of Contaminants

Control of the source will prevent the continued addition of pollutant. The three principal methods of source control are: removal, containment and hydrodynamic isolation. Removal of the source will require transportation of the waste and its final disposition in an environmentally acceptable manner. Containment of the waste can generally be accomplished by a cutoff wall made of soil-bentonite slurry or concrete. The waste can also be isolated hydrodynamically by installing a pumping well immediately downstream of the contaminant plume so that the flow through the contaminated zone is captured by the well. The shape of the *capture zone* with a single well at the origin of the coordinate axes has been given by Javandel and Tsang (1986) for a confined aquifer as

$$y = \pm Q/(2BU) - Q/(2\pi BU)\,\tan^{-1}(y/x)$$

where Q = the pumping rate
B = the thickness of the aquifer
U = the regional Darcy velocity

Javandel and Tsang (1986) also give equations for the capture zone formed by several wells. Figure 34.10 shows the capture zones for a single well for several values of Q/BU. The curve that fully encloses the plume is selected. The required pumping rate is obtained by multiplying the value of the parameter Q/BU by the product of the aquifer thickness, B, and the regional flow velocity, U.

Wellhead Protection

The Wellhead Protection Area is usually delimited as the capture zone of the well or well field limited by lines of equal travel time or *isochrones.* For a single well pumped at a rate Q in a confined aquifer of thickness B and regional Darcy velocity U, the curves of Fig. 34.10 show the capture zones in terms of the parameter Q/BU. The capture zone can thus be found for given values of Q, U, and B. The capture zone is an elongated area that extends in the up-gradient direction from a stagnation point located slightly down-gradient of the pumping well. It is possible to draw the streamlines inside the capture zone and to mark on them points of equal travel time to the well. The points with equal travel times can be connected by curves called isochrones. The one to three years isochrones are for short travel time and 5 to 10 years for long travel time. For example, the area from which the groundwater would reach the well in three years is called the three-year capture zone. In practice computer models are used to delineate the well head protection areas (see, for example, Haitjema, 1995). After the wellhead protection area is delineated, the potential sources of contaminant within the area are identified, management approaches are developed to protect the groundwater within the wellhead protection area and contingency plans are developed. In the U.S., this type of wellhead protection program is mandated by the Environmental Protection Agency for the preservation of the quality of groundwater used for production of drinking water.

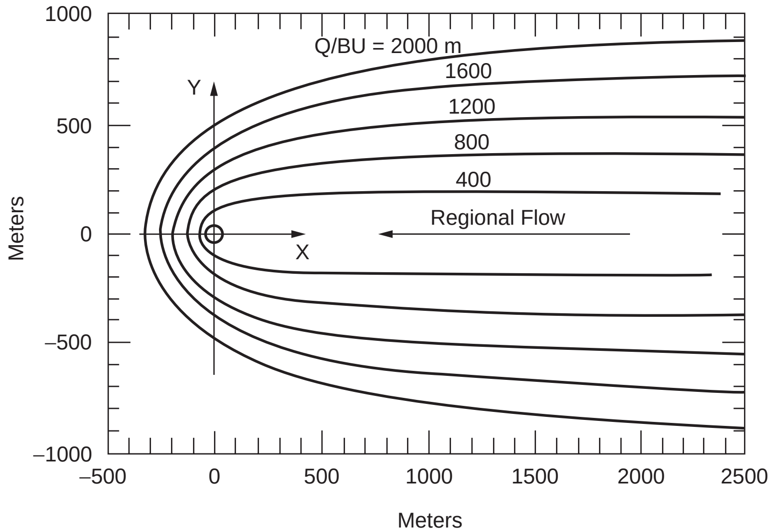

FIGURE 34.10 Capture zone for a single well in a confined aquifer. (*Source*: Javandel, I. and Tsang, C.F.,1986. Capture zone type curves: A tool for aquifer cleanup. *Ground Water* 24(5):616–625.)

Software

A modular semi-analytical model for the delineation of wellhead protection areas, WHPA, and a wellhead analytical element model, WhAEM are available from the U.S. Environmental Protection Agency (1998). The following description of WHPA is taken from the web site *http://www.epa.gov/esd/databases/whpa/abstract.htm*

"WHPA is applicable to homogeneous aquifers exhibiting two-dimensional, steady ground-water flow in an areal plane and appropriate for evaluating multiple aquifer types (i.e., confined, leaky-confined, and unconfined). The model is capable of simulating barrier or stream boundary conditions that exist over the entire depth of the aquifer. WHPA can account for multiple pumping and injection wells and can quantitatively assess the effects of uncertain input parameters on a delineated capture zone(s). Also, the program can be used as a postprocessor for two-dimensional numerical models of ground-water flow."

The following description of WhAEM is taken from the web site http://www.epa.gov/esd/databases/whaem/abstract.htm

"A computer-based tool used in the wellhead protection decision-making process to delineate ground-water capture zones and isochrones of residence times. Unlike similar programs, WhAEM can accommodate fairly realistic boundary conditions, such as streams, lakes, and aquifer recharge due to precipitation."

34.7 Landfills

A typical landfill consists of three major layers: a top, a middle and a bottom subprofile (Fig. 34.11). The purpose of the top subprofile is to cover the waste, to minimize the rainfall infiltration into the waste and to provide an exterior surface that is resistant to erosion and deterioration. The middle subprofile includes the waste layer, a lateral drainage layer with the leachate collection system underlain by a flexible membrane liner. The bottom subprofile includes an additional drainage layer, a leakage detection system and a barrier soil liner.

The design and operation of landfills are controlled by federal and local regulations. The federal regulations include Subtitle C landfill regulations of the Resource Conservation and Recovery Act (RCRA) as amended by the Hazardous and Solid Waste Amendments (HSWA) of 1984 and the Minimum Technology Guidance, (EPA 1988). The RCRA regulations mandate that below the waste layer there must

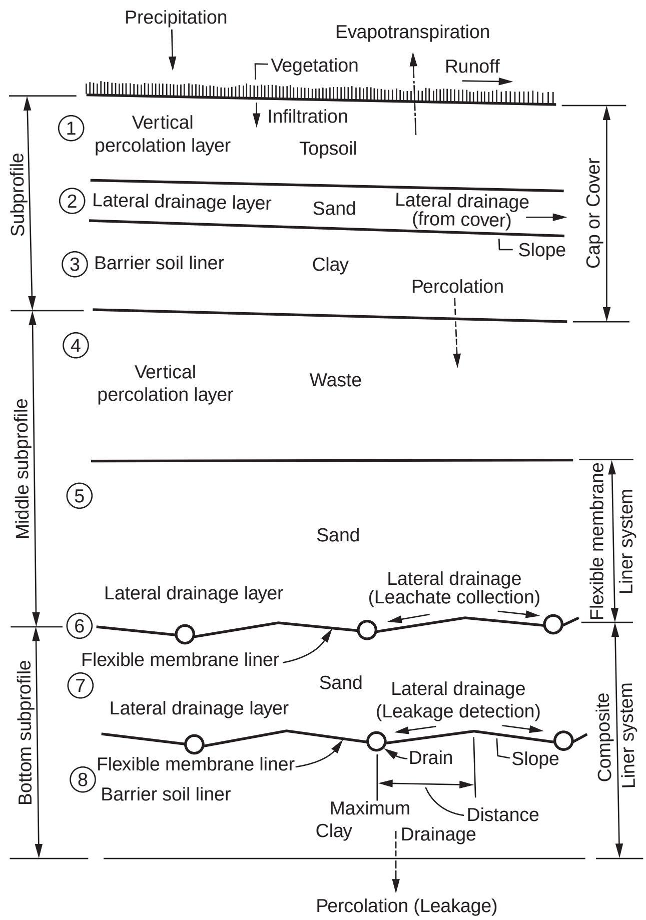

FIGURE 34.11 Landfill profile. (*Source*, Schroeder, P.R., Peyton, R.L., McEnroe, B.M., and Sjostro, J.W. 1992a, *Hydrologic Evaluation of Landfill Performance* (HELP) *Model, vol. III: User's Guide for Version* 2. Department of the Army.)

be double liners with a leak detection system. According to the guidance the double liner system includes a synthetic liner, a secondary leachate collection system and a composite liner consisting of a synthetic liner over a low permeability soil or a thick low permeability soil liner. The soil liner should have an in-place hydraulic conductivity not exceeding 1×10^{-7} cm/sec and a thickness of at least 3 ft. The primary and secondary leachate collection systems should include a drainage layer with a thickness of at least 1 ft with a saturated hydraulic conductivity of at least 1×10^{-2} cm/sec and a minimum bottom slope of 2%. The leachate depth cannot exceed 1 ft.

The simplified steady state equations governing the moisture flow through the landfill as given by Peyton and Schroeder (1990) are as follows. The lateral drainage per unit area Q_D is given by

$$Q_D = 2C_1 K_D Y h_o / L^2$$

where K_D = the saturated hydraulic conductivity of the lateral drainage layer, (Fig. 34.12)
Y = the average saturated depth over the liner (in.)
h_o = the head above the drain at the crest of the drainage layer (in.)
L = the drainage length (in.)
C_1 = 0.510 + 0.00205αL, where α is the dimensionless slope of the drainage layer(ft/ft)

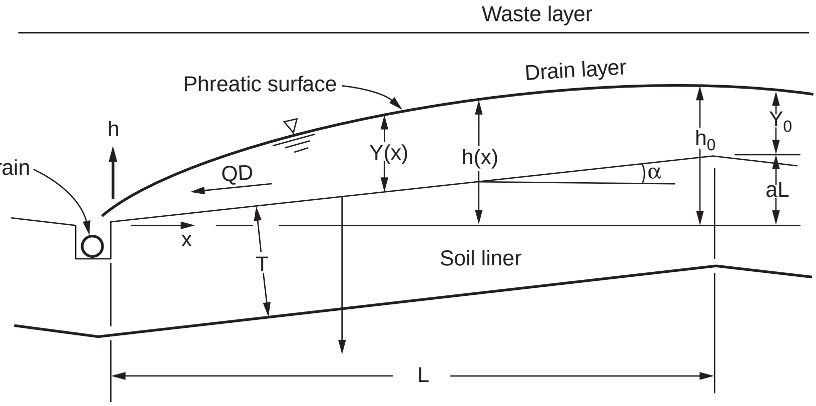

FIGURE 34.12 Landfill drainage layer. (*Source:* Peyton R.L. and Schroeder, P.R., 1990. Evaluation of landfill liner designs. *J. Environ. Eng., ASCE.* 116(3):421–437.)

The saturated depth at the crest of the drainage layer, Y_o (in.) is

$$Y_o = Y^{1.16} / [\alpha L]^{0.16}$$

and the vertical percolation rate through the soil liner Q_P is given by:

$$Q_P = L_F K_P [Y + T] / T$$

where L_F = the synthetic liner leakage factor
K_P = the saturated hydraulic conductivity of the soil liner
T = the thickness of the soil liner

Graphs for the estimation of the synthetic liner leakage factor can be found in Peyton and Schroeder (1990) and in Schroeder et al. (1992 a, b). The Hydrologic Evaluation of Landfill Performance model (HELP, Schroeder, et al. 1992 a, b) solves extended forms of the above equations for Q_P, Q_D, and Y. For more details on landfills, see, for example, Repetto (1999).

Software

The Computer program "Hydrologic Evaluation of Landfill Performance "(HELP) Version 3 can be downloaded from the U.S. Army Corps of Engineers Waterways Experiment Station web site (*http://www.wes.army.mil/el/elmodels/helpinfo.html*). The model computes estimates of water balance for municipal landfills, RCRA and CERCLA facilities and other confined facilities for dredged material disposal. Other related software is available from EPA and is listed on the Web site *http://www.epa.gov/esd/databases/datahome.htm*.

34.8 Geostatistics

Definition of Kriging

Hydrologic and hydrogeologic variables such as hydraulic conductivity, hydraulic head, storage coefficient, transmissivity, rainfall and solute concentration are functions of space. These quantities, although very variable, are not completely random and often exhibit a spatial correlation or structure. The study of such variables was developed by Matheron (1971) under the name *regionalized variables*. **Kriging** is a method of optimal estimation of the magnitude of a regionalized variable at a point or over an area, given the observations of this variable at a number of locations. Kriging also provides the variance of the estimation error. Kriging is useful for the estimation of the variable at the nodes of a network of points to develop contour maps of the variable. An estimate of the mean value of the variable on a given block or pixel is useful in Geographic Information Systems (GIS). It is also useful in the optimization of observation networks, for example by choosing the additional location that minimizes the uncertainty or by choosing the location to be removed that minimizes the error increment. Kriging provides the best (in the mean square sense of minimizing the error covariance) linear unbiased estimate.

Stationary and Intrinsic Cases

In the stationary case the mean m of the observations $Z(\boldsymbol{x})$ at $\boldsymbol{x} = (x,y,z)$ is the same everywhere and the correlation between two observations $Z(\boldsymbol{x}_1)$ and $Z(\boldsymbol{x}_2)$ depends only on their relative separation distance $\boldsymbol{h} = \boldsymbol{x}_1 - \boldsymbol{x}_2$ or

$$E\left[Z(\boldsymbol{x})\right] = m$$

$$E\left\{\left[Z(\boldsymbol{x}_1) - m\right]\left[z(\boldsymbol{x}_2) - m\right]\right\} = C(\boldsymbol{h})$$

where $C(\boldsymbol{h})$ is called the *covariance* function.

A useful generalization is the intrinsic case in which the increments of the variables are stationary. In the intrinsic case

$$E\left[Z(\boldsymbol{x}_1) - Z(\boldsymbol{x}_2)\right] = 0$$

$$E\left\{\left[Z(\boldsymbol{x}_1) - Z(\boldsymbol{x}_2)\right]^2\right\} = 2\gamma(\boldsymbol{h})$$

where $\gamma(\boldsymbol{h})$ = the semivariogram
$2\gamma(\boldsymbol{h})$ = the variogram

The variogram must satisfy some specific mathematical requirements. Examples of acceptable semivariogram functions include the following (de Marsily, 1986, p 303 or Kitinadis, 1993, p.20.6):

the power model	ωh^λ	$\lambda < 2$
the spherical model	$\omega[3/2(h/a) - ½(h/a)^3]$	$h < a$
	ω	$h > a$
the exponential model	$\omega[1 - \exp(-h/a)]$	
the gaussian model	$\omega\{1 - \exp[-(h/a)^2]\}$	

where h is the length of the vector $\boldsymbol{h}$ and the parameters ω, a and λ are selected to fit the empirical semivariogram.

Estimation

The pairs of observation points are classified according to distances, d_1, d_2, d_3,…, for example $0 < d_1 <$ 1 km, $1 < d_2 < 2$ km, $2 < d_3 < 4$ km, etc.. For each class the average distance and the average of $½(Z_i - Z_j)^2$ are calculated. The plot of these quantities (the mean distances against the half mean squares of the observation differences) is the *empirical semivariogram*. An acceptable semivariogram is fitted to the empirical one. The estimate Z_o^* of the variable Z at the desired point $\boldsymbol{x}_o$ based on the *observations* Z_i at the points $\boldsymbol{x}_i$, $i = 1…..n$ is

$$Z_o^* = \Sigma_i \lambda_o^j Z_i$$

The kriging coefficients λ_o^j are obtained from the kriging equations (de Marsily, 1986, p. 296)

$$\sum_j \lambda_o^j \gamma(\boldsymbol{x}_i - \boldsymbol{x}_j) + \mu = \gamma(\boldsymbol{x}_i - \boldsymbol{x}_o), \quad i = 1,\dots,n$$

and the unbiasedness constraint is

$$\sum_i \lambda_o^i = 1$$

where λ_o^j and μ = the unknowns

$\gamma(\boldsymbol{x}_i - \boldsymbol{x}_i)$ = the fitted semivariogram values for the distance between the observation points $\boldsymbol{x}_i$ and $\boldsymbol{x}_j$

$\gamma(\boldsymbol{x}_i - \boldsymbol{x}_o)$ = fitted semivariogram values for the distances between the observation points $\boldsymbol{x}_i$ and the point $\boldsymbol{x}_o$ at which the estimate is being obtained

The variance of the error of estimation, σ^2, is

$$\sigma^2 = \sum_i \lambda_o^i \gamma(\boldsymbol{x}_i - \boldsymbol{x}_o) + \mu$$

and the 95% confidence interval of the estimate is approximately

$$Z_o^* = \sum_i \lambda_o^i Z_i \pm 2\sigma$$

Extension

The methodology can be extended to the cases of two or more observation variables that are correlated. This extended technique is known as *cokriging* and its estimate is superior to that obtained by kriging each variable independently without considering their correlation.

For more details on geostatistics, see, for example, Kitinadis (1999)

Software

Two public domain software packages have been developed by EPA: GEOEAS, which is a geostatistical environmental assessment package (Englund and Sparks, 1988), can be downloaded from the EPA web site *http://www.epa.gov/esd/databases/geo-eas/abstract.htm*) and GEOPACK, which is a geostatistical package (Yates and Yates, 1989), can be downloaded from the EPA web site *http://www.epa.gov/esd/databases/geo-pack/abstract.htm*).

The following desription of GEOEAS is taken from the web site *http://www.epa.gov/esd/databases/geo-eas/ abstract.htm.*

"A collection of interactive software tools for performing two-dimensional geostatistical analyses of spatially distributed data. The principal functions of the package are the production of grids and contour maps of interpolated (kriged) estimates from sample data. Geo-EAS can produce data maps, univariate statistics, scatter plots/linear regression, and variogram computation and model fitting."

The following description of GEOPACK is taken from the web site *http://www.epa.gov/esd/databases/geo-pack/ abstract.htm.*

"A comprehensive geostatistical software package that allows both novice and advanced users to undertake geostatistical analyses of spatially correlated data. The program generates graphics (i.e., linear or logarithmic line plots, contour and block diagrams); computes basic statistics (i.e., mean, median, variance, standard deviation, skew, and kurtosis); runs programs for linear regression, polynomial regression, and Kolomogorov-Smirnov tests; performs linear and nonlinear estimations; and determines sample semivariograms and cross-semivariograms. GEOPACK allows users to incorporate additional programs at a later date without having to alter previous programs or recompile the entire system."

34.9 Groundwater Modeling

The management of groundwater requires the capability of predicting subsurface flow and transport of solutes either under natural conditions or in response to human activities. These models are based on the equations governing the flow of water and solutes. These equations are the conservation of mass, Darcy's equation and the contaminant transport equation. When written in two or three dimensions, these are partial differential equations.

The continuity equation is (De Smedt, 1999)

$$\rho\left[\theta(\alpha+\beta)\frac{\partial p}{\partial t}+\frac{\partial\theta}{\partial t}\right]=-\nabla\cdot(\rho\boldsymbol{q})$$

where θ = the water content
α = the elastic compressibility coefficient of the porous formation
β = the compressibility coefficient of the water, both with dimensions $[L^2/F]$
p = the groundwater pressure
$\boldsymbol{q}$ = the Darcian flux vector, namely the volumetric discharge which has components in the three directions x, y, z
ρ = the water density
$\nabla = (\partial/\partial x, \partial/\partial y, \partial/\partial z)$ is the del operator and the dot represents the scalar product

The saturated groundwater flow equation is (De Smedt, 1999)

$$\frac{\partial}{\partial x}\left(K_h\frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(K_h\frac{\partial h}{\partial y}\right)+\frac{\partial}{\partial z}\left(K_v\frac{\partial h}{\partial z}\right)=S_o\frac{\partial h}{\partial t}$$

where K_h and K_v = the horizontal and vertical hydraulic conductivities
h = the hydraulic head
S_o = the specific storage coefficient, namely the volume of water released per unit bulk volume of the saturated porous medium and per unit decline of the piezometric surface

If the flow is unsteady there is an initial condition at time $t = 0$ of the form $h(x,y,z,0) = h_o(x,y,z)$. There are three types of boundary conditions. In the first type the value of h is known at the boundary $h(x_b, y_b,z_b,t) = h_b(t)$ where the subscript b refers to the flow boundary. The second type is a flux boundary condition when the amount of groundwater exchange at the boundary is known. It is of the form $q_b(x_b,y_b,z_b,t) = q_b(t)$. The third type is a mixture of the two pervious types.

Generally these partial differential equations are solved numerically along with the boundary conditions specific to the problem. Both finite difference and finite element methods can be used.

The solute transport equation is (Konikow and Reilly, 1999) is

$$\frac{\partial(n_eC)}{\partial t}=\frac{\partial}{\partial x_i}\left(n_eD_{ij}\frac{\partial}{\partial x_i}\right)-\frac{\partial}{\partial x_i}(n_eCv_i)-C'W^*-\rho_b\frac{\partial\bar{C}}{\partial t}$$

where C = the solute concentration
n_e = the effective porosity of the porous medium
D_{ij} = the coefficient of hydrodynamic dispersion (a second order tensor)
v_i = the seepage velocity (q_i/n_e)
C' = the concentration of the solute in the source or sink of fluid
W^* = the volumetric flux (positive for outflow, negative for inflow)
$\bar{C}$ = the concentration of the species adsorbed on the solid (mass of solute/mass of solid)
ρ_b = the bulk density of the porous medium.

This equation is written for the case of linear equilibrium controlled sorption or ion-exchange reactions. The first term on the right hand side represents the change in concentration due to hydrodynamic dispersion. The last term changes in case of chemical rate control reactions or decay.

The complete solute transport model requires at least two equations: one equation for the flow and one for the solute transport. The velocities are obtained from the flow equation. For advectively dominated transport problems, the equations are hyperbolic partial differential equations. In this case the method of characteristics can be used.

For a more complete treatment of groundwater modeling, see for example, Konikow and Reilly (1999).

Software

One of the most comprehensive deterministic groundwater flow model available in the public domain is MODFOW (Harbaugh et al. 2000). The following description is taken from the web site *http://water.usgs.gov/software/modflow-*2000*.html*

"MODFLOW-2000 simulates steady and nonsteady flow in an irregularly shaped flow system in which aquifer layers can be confined, unconfined, or a combination of confined and unconfined. Flow from external stresses, such as flow to wells, areal recharge, evapotranspiration, flow to drains, and flow through river beds, can be simulated. Hydraulic conductivities or transmissivities for any layer may differ spatially and be anisotropic (restricted to having the principal directions aligned with the grid axes), and the storage coefficient may be heterogeneous. Specified head and specified flux boundaries can be simulated as can a head dependent flux across the model's outer boundary, which allows water to be supplied to a boundary block in the modeled area at a rate proportional to the current head difference between a "source" of water outside the modeled area and the boundary block. Currently MODFLOW is the most used numerical model in the U.S. Geological Survey for groundwater flow problems. In addition to simulating groundwater flow, the scope of MODFLOW-2000 has been expanded to incorporate related capabilities such as solute transport and parameter estimation.

The ground-water flow equation is solved using the finite-difference approximation. The flow region is subdivided into blocks in which the medium properties are assumed to be uniform. In plan view the blocks are made from a grid of mutually perpendicular lines that may be variably spaced. Model layers can have varying thickness. A flow equation is written for each block, called a cell. Several solvers are provided for solving the resulting matrix problem; the user can choose the best solver for the particular problem. Flow-rate and cumulative-volume balances from each type of inflow and outflow are computed for each time step."

The method of Characteristics Model (MOC3D) developed by Konikow, et al.(1996) simulates solute transport in flowing groundwater in three dimensions. The following summary is taken from the Web site *http://water.usgs.gov/cgi-bin/man_wrdapp?moc*3*d.*

> This model simulates three-dimensional solute transport in flowing ground water. The model computes changes in concentration of a single dissolved chemical constituent over time that are caused by advective transport, hydrodynamic dispersion (including both mechanical dispersion and diffusion), mixing (or dilution) from fluid sources, and mathematically simple chemical reactions (including linear sorption, which is represented by a retardation factor, and decay). The model can also simulate groundwater age transport and the effects of double porosity and zero-order growth/loss.
>
> The transport model is integrated with MODFLOW, a three-dimensional ground-water flow model that uses implicit finite-difference methods to solve the transient flow equation. MOC3D uses the method-of-characteristics to solve the transport equation on the basis of the hydraulic gradients computed with MODFLOW for a given time step. Particle tracking is used to represent advective transport and explicit finite-difference methods are used to calculate the effects of other processes.

Other groundwater models available from the U.S. Geological Survey are listed on the Web site *http://water.usgs.gov/software/ground_water.html*

Another approach to groundwater modeling consists in combining elementary analytic solutions. This *Analytic Element* method was developed by Strack (1989) and expanded by Haitjema (1995).

For more details on groundwater modeling, see, for example, Konikow and Reilly (1999).

Defining Terms

Advection — Transport of a solute due to mass movement of ground water and not dispersion or diffusion.

Artesian well — A well in which the water rises above top of the upper confining layer of an aquifer under pressure condition. If the piezometric level is above the ground the well is free flowing. Named after Artois, a former province of northern France.

Aquiclude — A geologic formation that is essentially impermeable.

Aquifer — A geologic formation that is water saturated and sufficiently permeable to yield economically important amounts of water to wells and springs.

Aquifuge — A geologic formations that does not contain nor transmit water.

Aquitard — A geologic formation that is less permeable than an aquifer and that partially restricts the flow of water.

Confined aquifer — An aquifer which is confined between two layers of impervious material.

Diffusion — Spreading of a solute in the ground water due to molecular diffusion.

Hydrodynamic dispersion — Spreading of a solute in ground water due to the combined effect of diffusion and mechanical dispersion.

Kriging — A statistical method to obtain the best (in the mean square sense) linear unbiased estimate of a hydrologic variable (such as hydraulic conductivity) at a point or over an area given values of the same variable at other locations.

Mechanical dispersion — Spreading of a solute in ground water due to heterogeneous permeability in the porous medium.

DNAPL — Denser than water non-aqueous-phase-liquid, such as chlorinated solvents.

LNAPL — Lighter than water non-aqueous-phase-liquid, such as gasoline.

NAPL — Non-aqueous-phase liquid in a multiliquid flow.

Perched water — A saturated zone of a limited extent located above the water table due to a local impermeable layer.

Phreatic surface — The water table or free surface in an unconfined aquifer.

Piezometer — A tube with a small opening penetrating an aquifer for the purpose of observing the hydraulic head.

Saturation ratio — Fraction of the volume of voids occupied by water.

Sorption — Chemical reaction between a solute in the groundwater and the solid particles which results in a bonding of part of the solute and the porous medium.

Specific yield — Volume of water drained per unit area of an unconfined aquifer due to a unit drop of the water table.

Unconfined aquifer — An aquifer without a covering confining layer in which the water surface or water table is free to move up or down.

Vadose zone — The zone between the ground surface and the top of the capillary fringe immediately over the water table.

Water Table — The water free surface at atmospheric pressure at the top of an unconfined aquifer.

Well log — A description of the types and depths of the geologic materials encountered during the drilling of a well.

Wettability — Preferential spreading of one liquid over a solid surface in a two liquid flow. In the saturated zone water is the wetting fluid. In the vadose zone the non-aqueous-phase liquid usually is the wetting fluid.

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Further Information

Heath (1998) provides an excellent, well-illustrated introduction to groundwater flow, wells, and pollution.

NRC (1984) provides a well-documented non-mathematical introduction to groundwater contamination including case studies.

Driscoll (1988) provides a wealth of practical information on well hydraulics, well drilling, well design, well pumps, well maintenance and rehabilitation and ground water monitoring.

Freeze and Cherry (1979), provides a textbook with a detailed treatment of groundwater flow and transport processes.

Bear(1979) and Bear and Verrujt (1987) provide an in depth study of ground water flow and contaminant transport, respectively, from a mathematical perspective.

Other useful textbooks on groundwater flow and contaminant transport include Charbeneau (2000) and Bedient et al.(1994).

Young et al. (1992) discuss the basic principles of contaminant transport in the unsaturated zone.

Deutsch and Journel (1992) provide a didactic review of kriging as well as a collection of geostatistical routines and Fortran source code for PC computers.

Delleur (1999) provides an extensive handbook on practical and theoretical aspects of groundwater engineering.