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THE INFLUENCE OF VADOSE ZONE CONDITIONS IN GROUNDWATER POLLUTION
PART II: FLUID MOVEMENT

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Summary

This is the concluding portion of a two-part review illustrating the influence of the vadose zone (i.e., soil materials located above a water table) on groundwater pollution. Seepage from landfills, lagoons, storage areas and land treatment sites invariably must pass through this region before entering an aquifer. In general, contaminant concentrations are highest near the source. Much of this zone is unsaturated, so that leachate and wastewater may have access to oxygen and be in close contact with solid particle surfaces. Consequently, conditions in the vadose zone affect both the seepage rate and the environment for attenuation of contaminant species.

The focus of this paper is on the application of Darcy's formula to describe the flow of water, leachate and other fluids under both saturated and unsaturated conditions. The discussion is presented in a state-of-the-art format. Individual topics include the material properties that influence flow and the relationship between seepage rate, fluid pressure and the degree of saturation.

To illustrate these concepts, five generalized examples are presented. They describe a wide range of practical situations, including:
- steady vertical seepage
- flow in the vadose zone parallel to a water table
- development of groundwater mounds under liquid filled impoundments
- wetting front advance through homogeneous soil
- wetting front advance under lined impoundments

These examples, and the principles from which they are derived, can be used both in analysis of existing situations and in design and operation of new facilities.

This paper is a companion to one previously published which discussed static conditions in the vadose zone.

Introduction

This is the second and final part of an overview of the influence of the vadose zone on groundwater pollution. The vadose zone is that section of a geologic profile that lies between the water table and the ground surface. Water saturation of the pores in this zone ranges from 0% to 100% and it is under a tensile stress condition. There are obviously many interfaces between air and water in the soil pores. Therefore, the principles of capillarity, or differences in the pressure between two or more fluids apply.

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The vadose zone is critical to contaminant movement when seepage must pass through it before entering an aquifer. Such a situation is illustrated on Fig. 1 and is often encountered at both existing and prospective landfills. Hence, flow in the vadose zone is an integral part of the fluid seepage and contaminant movement, and must be quantified for proper analysis and decision making.

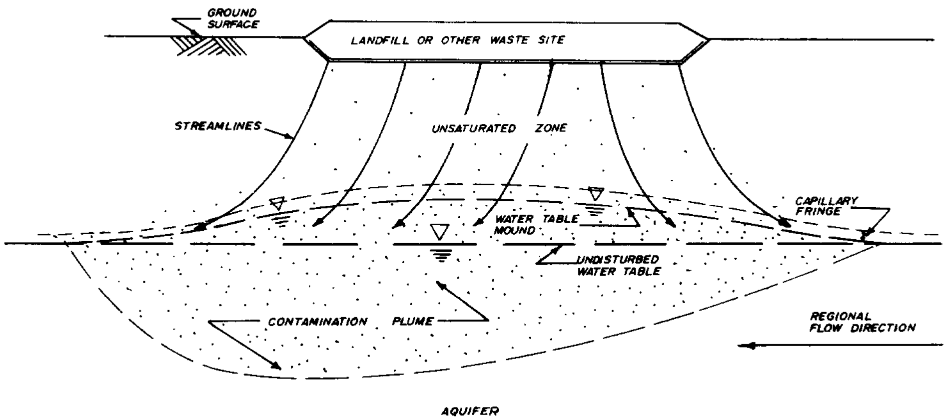

Fig. 1. Schematic of seepage from a landfill to a water table.

Similar conditions exist in many other situations of contaminated water seepage. Examples include leakage from lagoons and tailing ponds, percolation from land treatment sites and septic tank leaching fields, and infiltration due to precipitation on outside storage areas. With some modifications, the discussion presented herein also applies to spills of hydrocarbons and other immiscible fluids.

In all waste handling and disposal situations that may impact groundwater, the rate and quality of the seepage actually contacting the underlying water table are of paramount importance. Conditions in the vadose zone control the rate of liquid and gas movement, including the temporary or permanent immobilization of liquid by retention in the porous soil. Applying Darcy's equation and capillary principles, good and useful estimates of the discharge rate and degree of saturation in the vadose zone can be obtained.

Various physical, biological and chemical mechanisms take place in soil that either dilute, retard, immobilize or transform contaminant species, and otherwise mitigate the impact of the seepage. Some of these processes, notably biological decomposition, filtration and vaporization, are strongly influenced by the degree of saturation in the soil profile.

Consequently, analyses of seepage retention and movement, and solute alteration in the vadose zone will yield a more accurate estimate of the source parameters that are used in the rapidly evolving field of aquifer contamination analysis. The concentration in this two-part overview is on the hydraulics of the vadose zone. References [2—5] contain descriptions of contaminant attenuation processes in soil.

In Part I of this paper [1], static conditions in the vadose zone were discussed. Major elements described therein included the distribution of moisture in the vadose zone, applications of capillary principles to porous media, and the implications of unsaturated conditions on contaminant attenuation. In this part, the discussion continues with a description of Darcy's formula as applied to both saturated and unsaturated soil and the soil properties that influence seepage rates. These concepts are illustrated with the use of several examples which are of interest both for analysis of current situations of contaminated seepage and in design or operation of new facilities.

Essential features of fluid movement

Darcy's formula

Darcy's formula is the empirical relationship that describes the macroscopic flow of fluids in porous media. It applies to the flow of relatively incompressible liquids such as water and oil, and also to the bulk flow of gases such as air or natural gas. Darcy's formula has been used in both unsaturated and saturated conditions.

The basic formulation relates the fluid flow rate to the total head or hydraulic gradient, and to the viscous resistance to flow. The latter parameter is treated as a soil property, the coefficient of permeability or hydraulic conductivity. The flow rate of incompressible fluids is customarily expressed by Darcy's formula in volumetric terms:

$$Q = -K\,i\,A \tag{1}$$

where Q = volume discharge (L^3/T); i = hydraulic or total head gradient (L/L); K = hydraulic conductivity (L/T) — usually called simply the "permeability"; and A = gross cross-sectional area normal to the flow (L^2).

Each of these terms will be discussed below. In general, it can be stated that Darcy's formula implies laminar flow where the energy or head loss is directly proportional to the flow rate. Therefore, the permeability is customarily treated as a fixed and characteristic property of the medium. However, the form of Darcy's formula can be adapted to situations where the permeability is a function of the gradient [6], the fluid properties [7] or the degree of saturation [7, 8]. Coincident flow of more than one fluid can also be examined [7].

Darcy's formula is used directly to describe the seepage rate of a leachate, which is usually a water-based solution containing contaminants. Advection and dispersion of dissolved or suspended pollutant mass itself is a function of the solvent discharge and velocity [4]. Contaminant transport also occurs by molecular diffusion in response to concentration gradients. This is independent of the fluid flow rate and is customarily described by Fick's law. However, advection, dispersion and diffusion of dissolved pollutants can be combined in a single mass flux expression [4].

Dividing the discharge of the cross-section area yields a velocity term V, the Darcy (or superficial) velocity:

$$V = Q/A = -K\,i \tag{2}$$

The Darcy velocity is numerically, but not dimensionally, equal to the volume flux or discharge through a unit cross-section area of a porous medium. However, this cross-section intercepts both solids and pore space. Some pores may not contain water. Contaminant dispersion and transient fluid movement depend upon the seepage velocity, the average velocity within the water-filled portion of the cross section:

$$V_s = \frac{V}{S\,n} \tag{3}$$

where V_s = seepage velocity (L/T); n = porosity; and S = degree of saturation.

Total head gradient

The total head gradient, also called the hydraulic gradient, is the rate of loss in mechanical energy that would occur with flow in a given direction. At any point in space, flow will actually occur in the direction of the maximum total head gradient. In most situations of flow in porous media the total head gradient is computed with elevation and pressure head components only, as the velocity (kinetic) head is usually negligible.

$$i = \frac{dh}{dl} = \frac{d(h_z + h_p)}{dl} \tag{4}$$

where h = total head (L); h_z = elevation head (L); h_p = pressure head (L); and l = length measure along the flow path (L).

Water pressure, u_w, is converted to units of length, h_p, by division by the unit weight of the fluid:

$$h_p = \frac{u_w}{\gamma_w} \tag{5}$$

where γ_w = unit weight of water or leachate (F/L^3).

Below the water table, which is at atmospheric pressure, water is under a compressive stress. This is designated positive. Hence, in the vadose zone of water under tension, both the water pressure and the pressure head carry a negative sign.

The elevation head represents the gravity potential, and is numerically equal to the height of the point of interest above a datum. It is usually convenient to select a datum such that h_z has positive values throughout a region of interest. It can be seen from eqn. (4) that the gradient depends upon the difference in total head between two points, not on the individual value at any one location.

In a hydrostatic pressure distribution, water pressure increases linearly with depth as shown on Fig. 2. In this case, the variations in elevation and total head in a direction normal to the water table are equal and opposite. Consequently, the total head gradient is zero along this path, and there is no flow along it. Situations of vertical flow will be discussed later. Along a sloping water table, as shown on Fig. 1, there is no change in the pressure head, because the water table is defined as the locus of points with zero (gage) water pressure. However, there is a variation in elevation head. Consequently, groundwater flows downslope in both the vadose and phreatic zones.

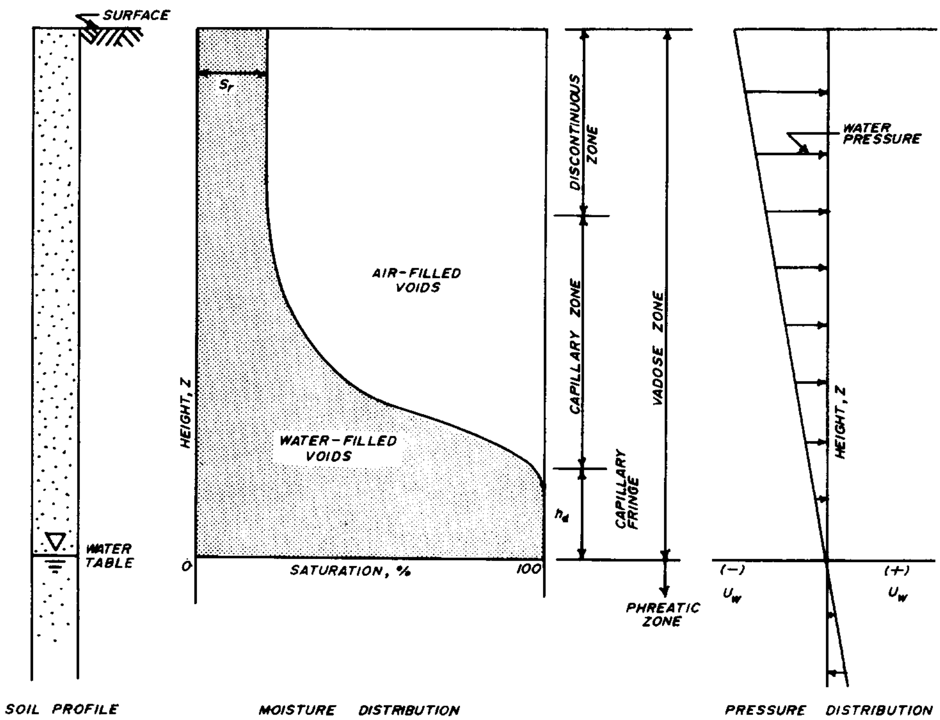

Fig. 2. Hydrostatic pressure and zones of moisture retention above a water table.

Much of the development of unsaturated flow analyses has been done in the agricultural and petroleum fields. In that literature, h_p may be replaced with h_c, the capillary pressure head. In the air–water situations of interest herein, h_c is always positive and is defined as follows:

$$h_c = \frac{(u_a - u_w)}{\gamma_w} = \frac{p_c}{\gamma_w} \tag{6}$$

where u_a = gauge air pressure (F/L^2); and p_c = capillary pressure (F/L^2).

Other components of total head may be considered in special circumstances, such as adsorptive and osmotic heads [9].

Saturated permeability

The four basic controls on permeability are the properties of the porous medium, those of the fluid, interactions between the soil and the fluid, and the degree of saturation. The influence of fluid properties and the degree of saturation will be discussed in following sections. The balance of this discussion deals with the saturated permeability of soils and rocks to water.

Saturated permeability has one of the widest ranges of a material property that is routinely used in engineering practice. The influence of permeability on discharge is evident from eqn. (1). Hence, emphasis is given in many projects on determining the permeability and its variations in space and direction. The main determinant of permeability is the gradation. Typical values are given in Table 1 [10].

TABLE 1

Typical values of saturated permeability

Typical formation	K (cm/sec)
Coarse gravel, open jointed rock	1×10^{-1}
Sand	1×10^{-2}—1×10^{-3}
Silty sand	1×10^{-3}—1×10^{-5}
Silt, fine sandstone	1×10^{-5}—1×10^{-7}
Clay, shale, mudstone	less than 10^{-7}

Soils with a saturated permeability below 10^{-7} cm/sec are often referred to as "impermeable".

Other physical properties that significantly affect permeability include porosity, particle arrangement or fabric, plasticity and stratification. Variations in these properties may influence the permeability of a given soil up to an order of magnitude or more, i.e., a loose sand of high porosity will obviously be more permeable than the same material after compaction.

The influence of many of these properties on permeability can be understood in terms of their effect on the average pore hydraulic radius, channel roughness, and tortuosity, which is the ratio of the microscopic to the macroscopic flow path lengths. Various theoretical equations such as the Kozeny—Carman formulation have been derived [11] to predict saturated permeability in terms of the factors discussed above. Such equations work fairly well for coarse, inert materials such as gravels and sands, but are not suitable for fine-grained, surface-active materials such as clays.

The measurable soil property of plasticity distinguishes clays from inert, granular soils. The negative charges on the surfaces of clay minerals attract water and its dissolved ions to form a bound water layer, often referred to as the diffuse ionic double layer [12]. Consequently, some of the porosity is unavailable for fluid convection, and the permeability is less than that which would be expected due to porosity and grain size alone.

Clay particles tend to be plate-shaped, and the detailed particle arrangement, or fabric, significantly affects the shape of the irregular pores. When clay particles are in a dispersed (parallel arrangement) fabric, the effective porosity consists of poorly connected, ellipsoidal voids. When the particles are in a flocculated (random arrangement) structure, the pores tend to be more spherical, larger, and more efficiently interconnected. Hence, a clay soil at a given porosity will have a lower permeability in a dispersed structure than in flocculated condition [12]. During construction of clay liners, the fabric can be controlled to a certain extent by the moisture conditions and method of imparting compactive effort.

The saturated permeability of a clayey soil may tend to vary with the quality of a water solution [13, 14], because the thickness of the bound water layer and clay mineralogy itself may change. Cation exchange, wherein ions of low valence are replaced by ones of higher valence, will tend to shrink the bound water layer [10]. Certain toxic organics reduce the dielectric constant and thus shrink the double layer as well [13].

Other situations exist wherein waste disposal and seepage affect permeability of soils in general. Filtration of suspended materials and precipitation (by neutralization, etc.) will tend to clog pores and reduce permeability. This may also occur in subsurface leaching fields, where the scavenging biomass accumulation fills some pores, especially if conditions are anaerobic [15]. Admixtures of stabilized sewage sludge in soils may tend to increase the porosity and the permeability [16].

Directional and spatial variations in permeability are also of interest. Most soils are anisotropic and most geologic formations are heterogeneous. The heterogeneity may be in the form of a relatively distinct stratification, as is common with alluvial deposits. There may also be a gradual, or trending, variation in soil properties, including permeability. This latter condition is often found along a stratum or in the vertical direction through residual soils.

Seepage across several strata is a series flow, such that the least permeable material dominates the flow rate, whereas flow parallel to the stratification is dominated by the most permeable strata. Often, soil bedding planes and the water table roughly follow the surface topography. Consequently, it can be seen from Fig. 1 that seepage in the vadose zone will tend to be a series flow, especially if a liner is used, whereas regional aquifer discharge occurs largely along the more permeable strata.

Flow of other fluids

The general principles stated above apply exactly to the flow of incompressible liquids such as petroleum products, except that they are generally not as reactive with clay particles as is water [7]. The viscous drag opposing flow is a function of fluid viscosity and unit weight as well as the soil properties. Hence, resistance to oil movement will differ from that to water, but the saturated permeability of a soil with one fluid can often be estimated from that of the other. The concept of intrinsic permeability is used, such that this parameter represents a generalized soil resistance to flow. When the saturated permeability and the basic properties of one fluid are known, then the intrinsic permeability of the soils can be computed [11]:

$$k = \frac{K_f \mu_f}{\rho_f g} \tag{7}$$

where K_f = generalized fluid permeability (L/T); k = intrinsic permeability (L^2); μ_f = dynamic viscosity (F—T/L^2); ρ_f = mass density (M/L^3); and g = gravitational constant.

It is then possible to estimate the permeability of a second fluid. This is done frequently in studies of petroleum reservoirs. Equation (7) can also be used to compute the variation in water permeability due to changes in density and viscosity of water as a result of salinity, temperature and leachate content variations.

Equation (7) does not apply when there is a significant electrochemical reaction between a fluid and the porous medium. This is the case with clay soils, such that the permeability of a clay liner to a leachate must be determined experimentally.

Darcy's formula also applies to the bulk flow of compressible fluids such as gases. The gravity potential is often very low for such low-density materials, so that elevation head is neglected. However, changes in the unit weight along the flow path of a compressible fluid do affect the pressure head (see eqn. 7). The expression of Darcy's formula that applies to such circumstances is:

$$\begin{aligned} Q_f &= -K_f \frac{d}{dl}(u_f/\rho_f g)A \\ &= -K_f \frac{d}{dl}(u_f/\gamma_f)A \end{aligned} \tag{8}$$

The unit weight or mass density of air can be computed with the ideal gas law in terms of absolute air pressure and temperature.

Situations where air flow due to pressure gradients may be of interest include air displacement by water tables or moving wetting fronts, and displacements by solids in a compressing landfill. Within the overall gas movement, individual gases such as radon and methane are transported by advection. Otherwise, their movement is analyzed by Fick's law [17]. Equation (8) applies to volume discharge only. It can be shown that the mass or weight discharge is a function of the gradient of the square of absolute air pressure.

Unsaturated permeability to water

The permeability of all fluids in a soil is affected by the moisture content, volumetrically expressed as the degree of saturation. It is often convenient to express the permeability (to water) at a given degree of saturation as a proportion of the saturated value. However, as shown on Fig. 3 [18], permeability does not vary linearly with saturation. The pores which have drained at any given moisture content are the larger ones which have the least resistance to fluid movement. Hence, permeability decreases precipitously with a reduction in moisture content, approaching a zero permeability as the bound, or residual, moisture content is approached.

In a given problem situation, the degree of saturation in the vadose zone may vary considerably as the flow rate or other environmental conditions vary. However, it is very difficult to measure unsaturated permeability directly even when it is possible to maintain a constant moisture content during the test [8, 19]. Thus it is often necessary to use a predetermined function for the variation in permeability as the degree of saturation changes. While many such expressions have been derived [8], those in most widespread use for soil and liquid-bearing rock are of the general form:

$$K_e = K_w S_e{}^B \tag{9}$$

where K_e = unsaturated permeability (L/T); K_w = saturated permeability (LT/); S_e = effective water-carrying saturation; and B = experimentally derived exponent (often a function of pore-size distribution).

Hydraulic parameters and static moisture retention

The parameters necessary to compute the B and S_e terms of eqn. (9) are often determined from interpretation of a static moisture distribution (capillary-retention) curve. Such a plot shows the variation in moisture content within a soil as the capillary pressure, $u_a - u_w$, varies. When the air pressure is atmospheric, changes in water pressure and capillary pressure are equal. Hence, a hydrostatic moisture profile such as that shown in Fig. 2 is a capillary-retention curve, except near the surface where evapotranspirative drying may result in extremely negative water pressures. For a single soil formation, a retention curve is characteristic except for some hysteresis between wetting and drying (imbition and desaturation) cycles. Other hydraulic properties of interest besides those necessary to employ eqn. (9) may also be determined from the static moisture curve.

It can be seen from Fig. 2 that up to three subdivisions of the vadose zone can exist under hydrostatic conditions. The major features of each are listed in Table 2, along with specific parameters indicated by each subdivision.

The capillary fringe is the zone of full saturation of water in tension above the water table. In this region, $K_e = K_w$, so eqn. (9) need not be used. The capillary fringe is analogous to the height of capillary rise in a tube, such that the height (or thickness) h_d is directly proportional to the average pore diameter. The average pore diameter is primarily a function of soil gradation. Hence, a high value of h_d will be observed in silts and clays and a low value in sand and gravel.

The height of the capillary fringe can often be readily determined from test boring samples and observation of the water table level in wells. From preceding discussions, it can be seen that the thickness of the fringe, as a function of gradation, also indicates the relative value of saturated permeability. The capillary fringe also reduces the depth of the zone of aeration beneath a site, so the zone of potential aerobic treatment in the subgrade may be much thinner than that indicated by water table depth alone. Finally, for purposes of estimating dissolved contaminant dilution and dispersion, the saturated capillary fringe can be considered to be an extension of the water table. It can be seen from Fig. 1 that a contamination plume often tends to spread from the upper region of the phreatic zone and the capillary fringe. However, water will not flow readily from the fringe into a monitoring well by gravity, so that extra measures are required to obtain water quality samples, such as the use of lysimeters [20].

TABLE 2

Division of the static groundwater profile

Division	Saturation	Pressure	Key parameter
Phreatic zone	100%	positive	saturated permeability, K_w
Capillary fringe	100%	negative	height of fringe, h_d
Unsaturated capillary zone	varies	negative	pore-size distribution (exponent B)
Discontinuous moisture zone	varies	negative	residual saturation, S_r

In the unsaturated zone, $K_e \neq K_w$, and eqn. (9) must be used. However, the shape of the retention curve reflects the pore-size distribution, and it can be used to estimate the exponent B. In this zone, there are curved menisci between the air- and water-filled voids, with each fluid at a different pressure. Equilibrium across such interfaces is maintained by surface tension. In general, the radii of the menisci are similar to the radii of the pores in which they are contained. The equilibrium is represented in the capillary equation:

$$r = \frac{2T_s}{p_c} = \frac{2T_s}{-u_w} \tag{10}$$

where r = interface (and pore) radius (L); and T_s = surface tension (F/L).

At a given value of water pressure, pores of smaller radius than that indicated by eqn. (10) are saturated. The overall degree of saturation is thus a function of the pore size distribution, and a unique relationship between pressure and saturation is manifest (Fig. 2). In turn, relationships between relative permeability, K_e/K_w, and negative pressure for various types of soils have also been derived empirically, using the pore-size distribution as an indicator [7, 8]. Table 3 shows typical values of B.

At some height above the water table under static conditions, the saturation curve becomes asymptotic to the residual saturation. This parameter is identified as S_r on Fig. 2. The residual moisture is strongly bound to the solid particles by attractive forces, and will not drain due to gravity. However, evapotranspiration may reduce the moisture content below S_r, especially at the surface and in the root zone [21]. The residual saturation represents a particular soil type and structure, and is analogous to the field capacity, representing the upper part of a formation. When the moisture content is at or below S_r, eqn. (10) does not apply and the water pressures are extremely negative.

TABLE 3

Experimentally obtained values of exponent B, eqn. (9) (adapted from Ref. [7])

Soil fabric	B
Uniform gradation, granular	3.5
Well-graded, granular	4
Aggregated fine-grained	5

The residual saturation is useful in the present context for two reasons. First, if the actual moisture content is below S_r, the soil will adsorb and retain moisture made available to it form surface sources. Subsequently, this water may be removed by evapotranspiration. The use of these concepts in land treatment and leachate retention were discussed in Part 1 [1].

Secondly, the permeability to water in liquid form is negligible at the residual saturation [22]. Hence, the effective saturation for water conveyance in eqn. (9) is essentially zero at S_r. The value of S_e is computed as follows:

$$S_e = \frac{S - S_r}{1 - S_r} \tag{11}$$

To transmit fluid at a given rate (eqn. 1) a finite permeability must exist (eqn. 9). Consequently, in situations of water or leachate flow, $S > S_r$. Clay soils have a higher affinity for water, such that the value of S_r may be 50% or more [23]. Remolded, dispersed clays such as those in compacted liners may have still higher values. In contrast, free-draining materials such as clean sands and gravels will have residual saturation values on the order of 5% to 15%. Moreover, granular materials desaturate to S_r in a short distance above the water table, whereas finer-grained materials retain significant amounts of moisture well beyond the capillary fringe [24].

A compacted clay liner must be placed in an unsaturated condition [10, 12]. However, it will adsorb more liquid readily, and remain in an essentially saturated condition. Consequently, the saturated permeability is used for liners in most practical conditions even if the subgrade is unsaturated.

Unsaturated air permeability and gas diffusion coefficients

Air permeability, K_a, decreases as the moisture content increases, as shown on Fig. 3. The maximum value is often taken at the residual saturation [7], and variations in air permeability due to changes in saturation are extensively reported in the petroleum engineering literature. Air permeability reaches a zero value when the bubbles are isolated from each other, often at saturations exceeding 80% [9]. Bubbles can be entrapped with a rising water table [25] or be physically occluded by soil or waste fill compression [26]. Hence, it is not reasonable to assume that air circulates freely just because the soil is unsaturated.

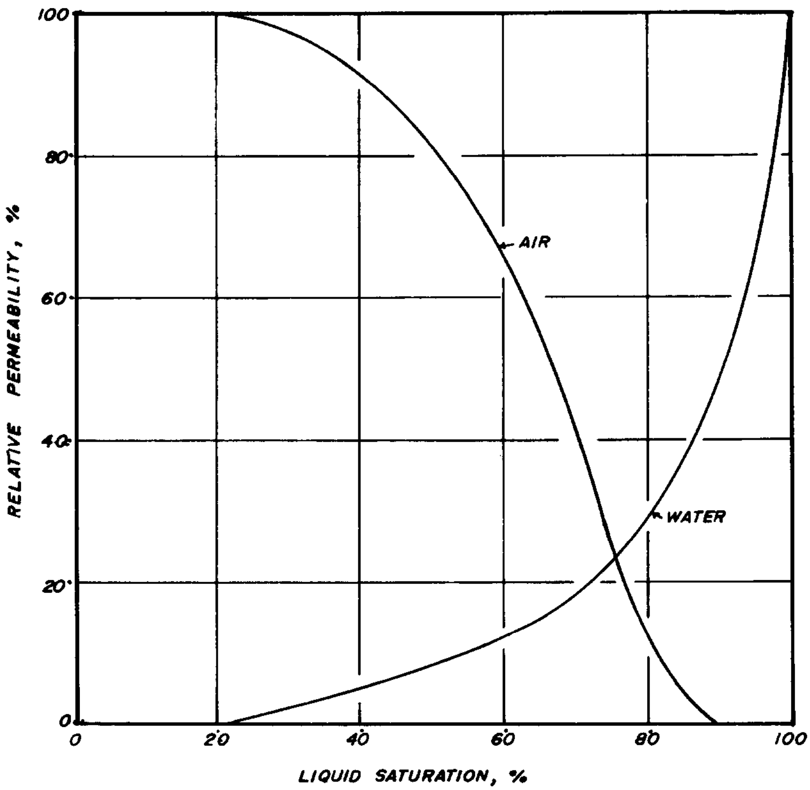

Fig. 3. Influence of saturation on the relative permeability (conductivity) to fluids in porous media.

Many important situations for subsurface gas transport depend on diffusion, rather than pressure flow (eqn. 8). Examples include biological treatment of wastewaters and hydrocarbon spills [27, 28]. However, gradation, porosity and the degree of saturation influence the effective diffusion coefficient of a gas in soil, such that the value of this coefficient in free air must be corrected [29]. In the absence of more definitive data, it may be useful to vary the value of the diffusion coefficient as a function of moisture content (Fig. 3). A linear variation of the diffusion coefficient is reported in Ref. [30].

Examples of seepage in the vadose zone

In this section, several examples of seepage and the methods used to analyze them are presented. Hydraulic analysis yields data on the flow rate, fluid pressure and moisture distribution. These results are used in concert with other studies, such as water quality sampling, in a particular project. The following examples illustrate Darcy's law and unsaturated soil properties in the context of practical situations:

- steady vertical seepage
- steady flow in vadose zone parallel to a water table
- development of groundwater mounds under liquid-filled impoundments
- wetting front advances through homogeneous media and under lined impoundments

Example 1: Steady vertical seepage

It is often useful to model deep percolation of liquid from surface or near-surface sites as a steady flow. Such analysis may be sufficient for design and operation, or else be a first approximation of more complex transient conditions. Steady-flow models are often used for slow rate discharge and treatment of wastewater, such as spray irrigation systems [31] and septic tank leaching fields [32]. In these and other cases, the average rate of deep percolation is calculated as the resultant of a water balance:

$$\text{percolation} = \text{precipitation} + \text{dosage} - \text{runoff} - \text{evapotranspiration} \qquad (12)$$

The present discussion is limited to deep percolation, which is a groundwater recharge, as opposed to surface infiltration. Some water that penetrates the surface re-emerges as runoff downslope or as evapotranspiration. Estimation of infiltration is quite complex, but excellent references are available [33, 34].

To operate the wastewater treatment site and prevent groundwater contamination, the controllable factors in eqn. (12) must be adjusted so that the percolation rate is not excessive. There are three major modes of failure for various land treatment systems that are related to the percolation rate:

1. Inability of the vadose zone or underlying phreatic zone to transmit the flow. Flooding of the application area or high groundwater mounding will result.
2. Insufficient air circulation or retention time in the treatment zone such that treatable contaminants are not stabilized.
3. Percolation of soluble salts at a rate above the dilution capacity of the groundwater, or exhaustion of the ion-exchange and absorptive capacity of the soil.

Empirical guidelines to avoid these situations are reported in the literature [31, 32]. However, it is often useful to understand the basic physics of deep percolation under steady flow. When a steady flow is assumed to be vertical, both discharge and velocity are constant. Consequently,

$$V = -K_e i = \text{constant} \tag{13}$$

Figure 4 shows several saturation and pressure head profiles for a homogeneous soil under different steady percolation rates. The upper part of the profile is of most interest for wastewater stabilization or contaminant attenuation. The vertical gradient is primarily due to gravity (i.e., change in elevation head), so the pressure head, and thus the saturation, is constant. Hence, with a gradient of 1.0,

$$V = -K_e\,(1.0) \tag{14}$$

Higher percolation rates require higher effective permeabilities and moisture contents. The maximum flow rate that can be conveyed without surface ponding or submergence of a leaching field is the saturated permeability, K_w.

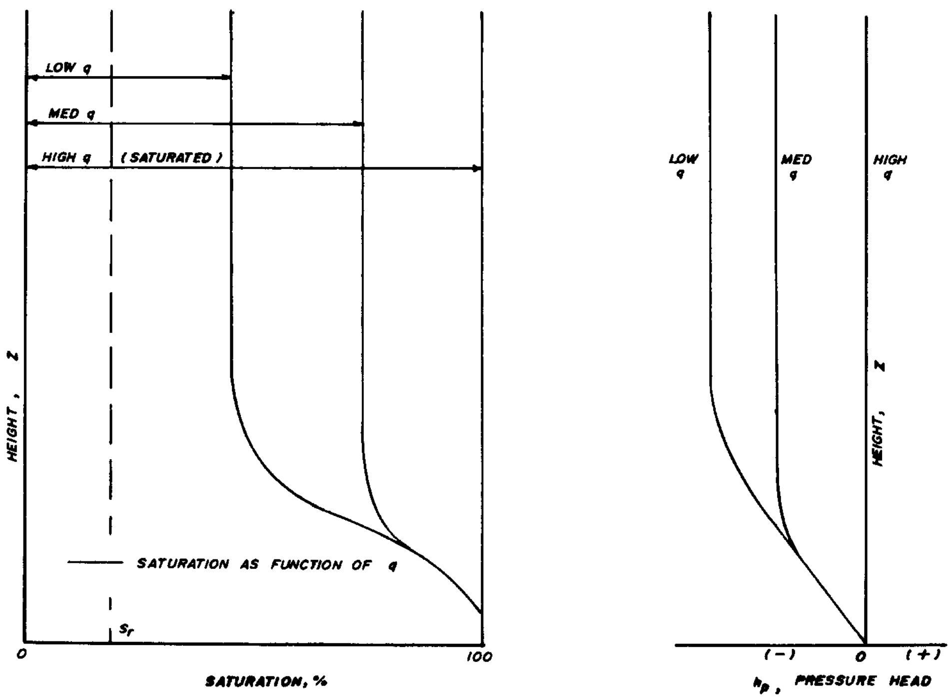

Fig. 4. Relationship between flux (unit area discharge), saturation and pressure head for steady, vertical flow.

To provide air circulation and detention time for chemical and biological reactions, it is necessary to maintain aerobic conditions with a low flow rate. As noted previously, oxygen diffusion rates vary with saturation [27, 30]. In the root zone, plant activity diminishes rapidly at saturations above 70—75%

[31]. Hence, the deep percolation rate required to biologically decompose wastes is far below that of the saturated permeability. The effects of extended anaerobic conditions are discussed in Refs. [15] and [32].

Consideration of layered soil systems is beyond the present scope of discussion. However, it is obvious that the least permeable layer governs a steady flow rate of both air and water, as discussed in Ref. [7].

At lower portions of the vadose zone, as the capillary fringe is approached, the static degree of saturation (Fig. 2) provides higher permeability than that required to satisfy eqn. (13). The assumption of one-dimensional flow is less tenable, and the seepage spreads out at the water table as shown on Fig. 1.

It is also common to use a steady-state analysis with an annual water balance to determine the average leachate percolation rate from a landfill, and thus the average conditions in the subgrade that are relevant to various attenuation processes. However, these average values cannot be confidently assumed to represent conditions at any particular time. The water balance of eqn. (12) must be adjusted for landfills. There is usually no positive dosage of liquid. The only fluid supply parameters, precipitation and the initial moisture content of the landfill, are randomly distributed [35].

If the amount of fluid infiltrating through the cover exceeds that which

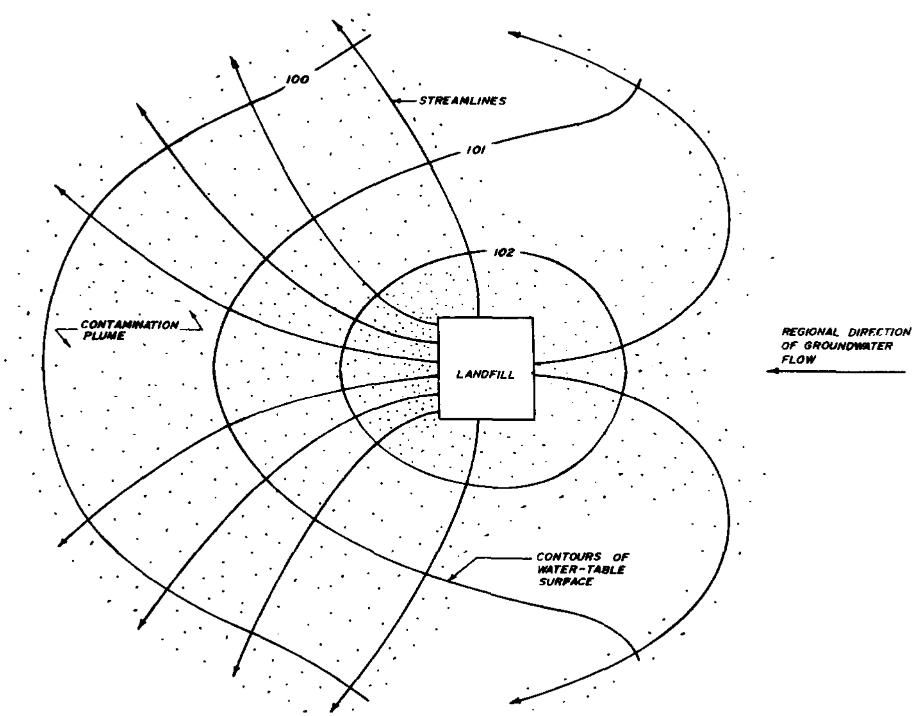

Fig. 5. Plan view of mound on groundwater system.

can be readily transmitted through a liner (if any), then a "bathtub" condition results, such that leachate collects on the liner. Analysis of transient seepage for this condition is dicussed in the last example herein. Leachate that overflows through the landfill sides or is captured in a leachate collection system must be accounted for in a water balance.

Deep percolation is a recharge to the groundwater, and the slope of the water table must adjust to continue moving recharge liquid. A local groundwater mound may form under a landfill or land treatment site, as shown on Fig. 1. Such a mound locally distorts the water table both longitudinally and transversely, as shown on Fig. 5, so that three-dimensional flow and dispersion analysis [4, 17, 24] may be required.

Methods to estimate the height and extent of a groundwater mound are discussed in Refs. [36] and [37]. The height of the mound is a function of the percolation rate, recharge-area geometry, permeability and other factors. A groundwater mound reflects percolation in an area that exceeds natural recharge from the surface. Where the seepage from a landfill is less than that of the surrounding land, a groundwater mound will not form. This condition is brought about with the use of tight, well-graded covers, liners and leachate collection systems. A fortunate result of such minimization of leachate is that the depth of aeration in the soil is not reduced by mounding. However, it is difficult for oxygen to diffuse under a wide landfill, and rest periods (no dosage) do not occur as at a land treatment facility.

Example 2: Flow in the vadose zone parallel to a water table

When a vertical seepage of contaminated liquid encounters the highly saturated zone above a water table, the streamlines curve and align with the groundwater mound and regional water table slope (Figs. 1 and 5). The contaminants will disperse into the phreatic zone both under and downstream of the source of seepage [4, 38], as shown on Fig. 1. Immiscible fluids which are lighter than water, such as petroleum products, form a "pancake" on top of the capillary fringe and also float downstream. Products of hydrocarbon spill decomposition are often more soluble than the feedstock [28], and thus the floating oil will be a further source of aquifer pollution beyond a spill location.

Consequently, the movement of contaminants and conveying water in the vadose zone downstream of a site may be of considerable interest in assessing both potential for contaminant attenuation and also further movement into an aquifer. Estimates of the discharge and flow thickness are of value in planning monitoring programs that indicate the profile of contaminant distribution below and beyond a site. The downstream vadose zone situation is shown schematically on Fig. 6.

It was noted previously that the vadose zone is subject to a gradient equal to the water-table slope even at locations where there is no gradient (and flow) normal to the water table. Furthermore, there is a finite permeability at all depths where the degree of saturation is in excess of the residual value,

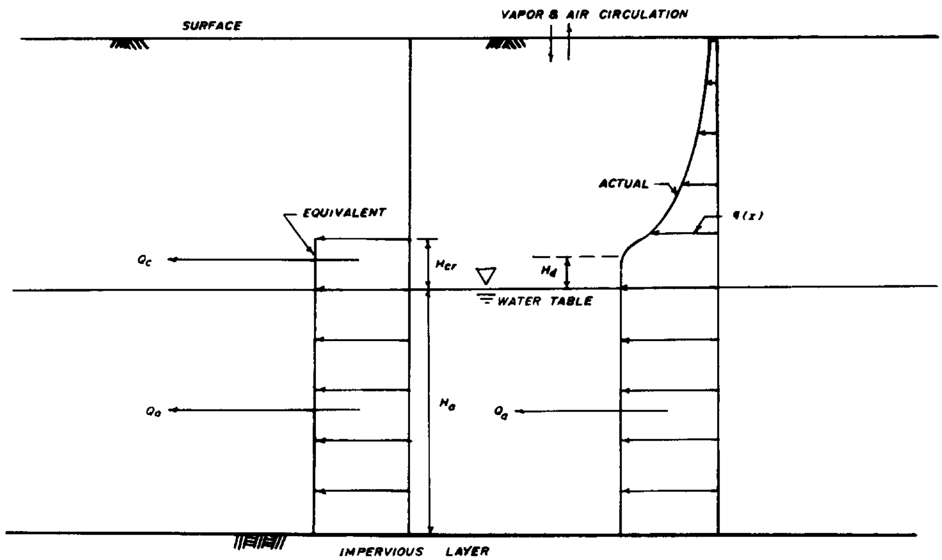

Fig. 6. Flow in the vadose zone parallel to a water table; reduction to an equivalent saturated thickness.

S_r, as shown on Fig. 3. The discharge in the saturated capillary fringe can be readily computed with eqn. (1). For a unit width of the flow zone:

$$Q_f = K_w\ i_s\ h_d\ (1.0) \tag{13}$$

where Q_f = discharge in the capillary fringe (L^3/T); and i_s = slope of the water table (L/L). The thickness of the capillary fringe can be estimated [10]:

$$h_d = \frac{4T_s}{\gamma d_{50}} \tag{14}$$

where d_{50} = median soil grain size. The value of surface tension, T_s, is sensitive to the water quality [7].

Equation (13) accounts for flow in the capillary fringe only. In well-graded or fine-grained soils, the zone of significant downslope flow in the vadose zone may be much thicker than h_d. However, the thickness of this zone is indistinct, and the permeability varies with moisture retention above the capillary fringe. Methods to estimate an equivalent saturated thickness in the vadose zone have been derived [38], such that the discharge in the vadose zone parallel to a water table can be calculated:

$$Q_c = K_w\ i_s\ H_{cr}\ (1.0) \tag{15}$$

where Q_c = net discharge in the vadose zone; and H_{cr} = equivalent saturated thickness in the vadose zone ($H_{cr} \geqslant H_d$).

The approximation of H_{cr} is shown schematically on Fig. 6. The total discharge in an unconfined aquifer is, of course, the sum of Q_c and Q_a, the aquifer discharge. The value of this analysis is not so much in estimating the discharge of the groundwater solvent, as Q_c is often quite small, but rather in the mass balance accounting of contaminants seeping from a site.

It must also be noted that evapotranspiration from a polluted capillary fringe area may occur when the water table is shallow. Uptake of contaminants by plants may be a serious problem in the locality of a landfill over a shallow water table. Materials deposited in the soil due to off-site evaporation may be re-mobilized by rising water tables (seasonal) or precipitation recharge (intermittent). Methods to estimate upward movement of liquid from a shallow water table are described in Ref. [39].

Example 3: Development of a groundwater mound under liquid-filled impoundments

When seepage rates change, unsteady (transient) phases exist as the distributions of soil moisture and pressure adjust to the new hydraulic loading. An increase in seepage will cause a wetting, or increased saturation front to advance from the source. This is shown in Figs. 8 and 9. A decrease in the recharge will result in desaturation. The vadose zone may drain to lower saturation corresponding to a different flow rate (Fig. 4), or drain completely to the no-flow (vertical) hydrostatic moisture distribution as shown on Fig. 2.

When a liquid-filled impoundment is maintained at the surface for an extended period, the unsteady transition tends to consist of a series of stages leading to a severe rise in the groundwater mound, as shown in Fig. 7. The water table mound may eventually contact the base of the impoundment if the water table is shallow or the transmission capacity of the underlying aquifer is limited; this is shown as stage III on Fig. 7.

However, it takes a finite time after pond installation before water table contact with an impoundment occurs. A considerable volume of seepage is necessary to saturate the soil when it either has a low initial moisture content or when the water table is deep. If the rate or total volume of seepage is limited, or the design life of the pond is short, then stage III might never fully develop. In this case, steady-state seepage analyses may be grossly overconservative.

Therefore, it is useful to examine the two transitional stages shown on Fig. 7 that preceed establishment of steady saturated seepage. In stage I, a wetting front advances from the site toward the water table. This front may be saturated or unsaturated, depending upon whether the pond is lined or unlined, and also on whether air is freely displaced by the water. In stage II, the positive pressure mound forms and rises. If the initial wetting front was saturated, this occurs rapidly as a pressure wave moves up from the water table, transforming the water pressure from positive to negative. If the initial wetting front was unsaturated, a further delay of stage III occurs as continued seepage is diverted to form the rising mound.

The analysis of these stages is described in Refs. [40] and [41]. Two important variations of stage I are described in the following examples. These final examples are presented to show unsteady (transient) cases of flow in the vadose zone.

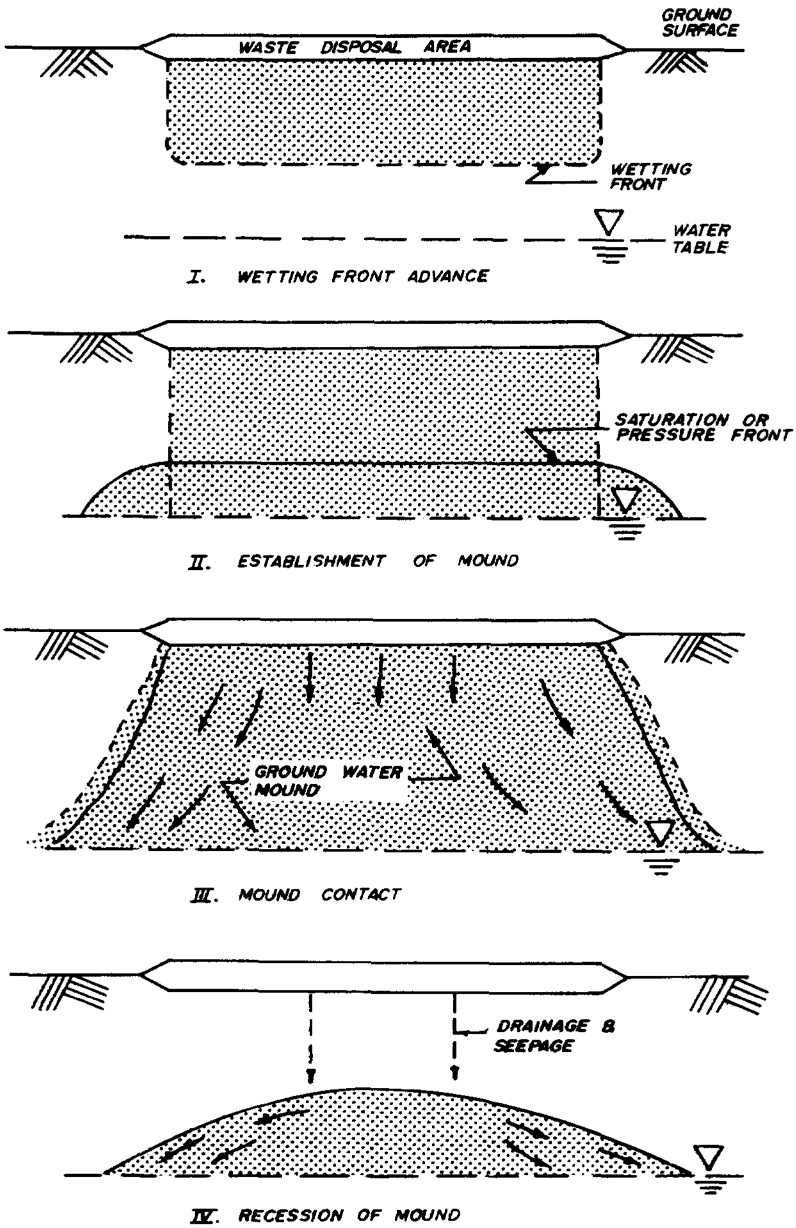

Fig. 7. Development of groundwater mound under liquid impoundments (adapted from Ref. [33]).

Example 4a: Wetting front advance through homogeneous material

One general case of considerable importance is that of seepage from a standing body of water such as an unlined lagoon, tailings pond or wastewater percolation basin. This is illustrated on Fig. 8. The common factors in these cases are that the water pressure is positive at the base of the impoundment, and the impoundment has an extended, but not indefinite design life. The wetting front moving through the soil from such ponds will be saturated if the air initially present in the pores is readily displaced.

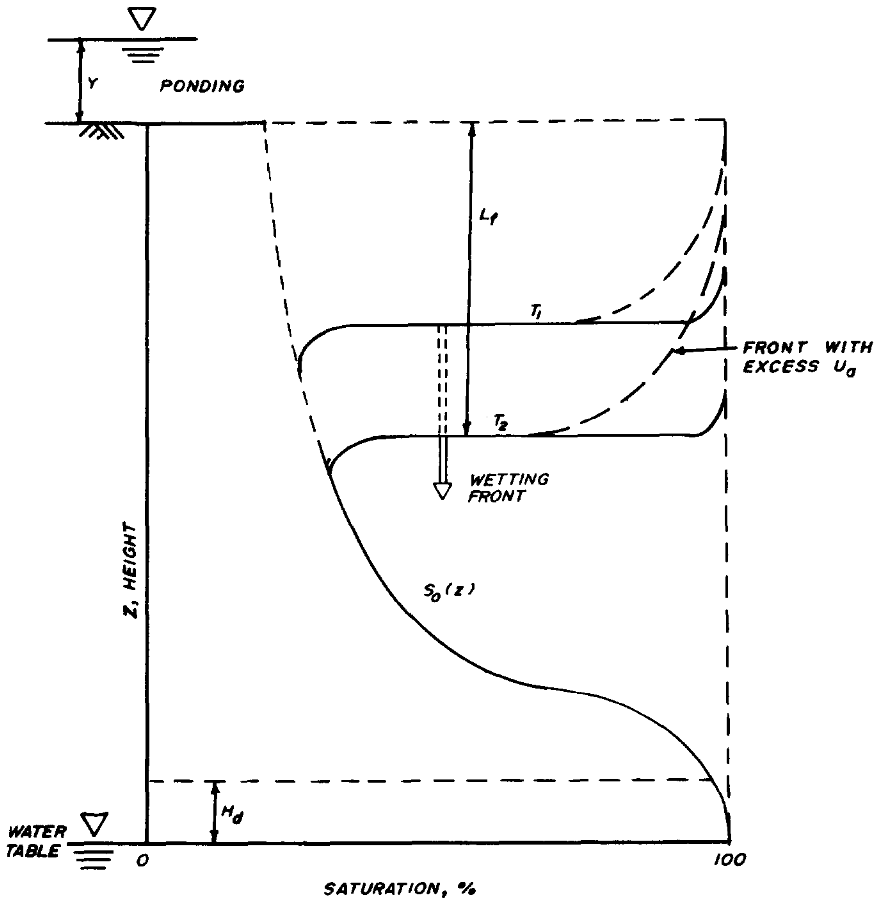

Fig. 8. Wetting front advance through homogeneous material.

Assuming a saturated wetting front, it can be seen that the vertical seepage is uniform at all levels behind the front at a given instant. Consequently, the discharge through the bottom of the pond equals the flow rate at the front, and the unsteady flow can be analyzed with the macroscopic total head gradient.

At the top of the wetted soil column, h_p is positive and equal to the pond depth. It has been shown [38] that the average value of h_p at the wetting front is $-H_{cr}$. In the vertical flow, the difference in elevation head between the source boundary and the wetting front is equal to the depth of frontal penetration at any given point in time. Note that the convenience of head notation also allows a distance parameter (z, h_d, H_{cr}) to be used to represent a pressure or potential level where appropriate. Therefore, applying eqn. (2) yields

$$V = -K_w \frac{(L_f + Y) - (-H_{cr})}{L_f} \tag{16}$$

where Y = pond depth (L); and L_f = wetting front penetration (L). It can be seen from eqn. (16) that the gradient, velocity and discharge decrease as the wetting front advances into the soil. Seepage also decreases as the pond depth is lowered, as would occur if the lagoon is not replenished. Equation (16) will not apply after the pond drains, when the upper part of the wetted column commences desaturation. This condition was shown schematically in Part I. The total seepage volume over a given period of time is evaluated by integrating eqn. (16) for the entire pond surface.

It is often useful to determine the depth of saturated wetting that can occur from intermittent surface flooding such as in high-rate wastewater discharge. The depth of seepage penetration is quite dependent upon the antecedent moisture, or initial saturation:

$$L_f = \frac{Y n}{(1.0 - S_i)} \tag{17}$$

where S_i = initial saturation before flooding.

It can be seen that eqn. (17) is a simple volumetric relationship between ponded water depth and the height of water in a porous, initially unsaturated medium. Penetration depth increases if the soil becomes compacted or clogged, or if insufficient time has elapsed since the preceeding dosage to allow drainage and evapotranspiration, as discussed in Part I.

In a transient flow, water replaces air in the pores. When a wetting front extends over a very wide area, the path length for air escape to the surface can be quite long, because the wetting front itself blocks upward air flow. Consequently, a higher air pressure gradient is necessary to displace all air from the recharge area (eqn. 8).

The major practical result of this situation is that the advancing wetting front is unsaturated, as shown on Fig. 8. The permeability to water behind the wetting front is K_e, a value below that of K_w. Therefore, the seepage rate is lower. Stage III (Fig. 7) is also delayed since the remaining air-filled porosity must be saturated after the wetting front contacts the water table. Therefore, the wider and more rectangular the recharge area is, the more conservative will be the discharge as computed with eqn. (16).

Detailed description of the saturated frontal advance is given in Ref. [40]. The condition of delayed air displacement is discussed in Ref. [42].

Example 4b: Wetting front advance under lined impoundments

Seepage from ponds or lagoons which are underlain by aquitards (strata of low permeability) or liners is an occasion of series flow. Thus a disproportionate amount of the available head is dissipated in the least permeable layers and seepage is reduced. It is also possible that the use of a liner which is much less permeable than the subgrade soil will result in a transient unsaturated flow in the subgrade. This situation is shown on Fig. 9.

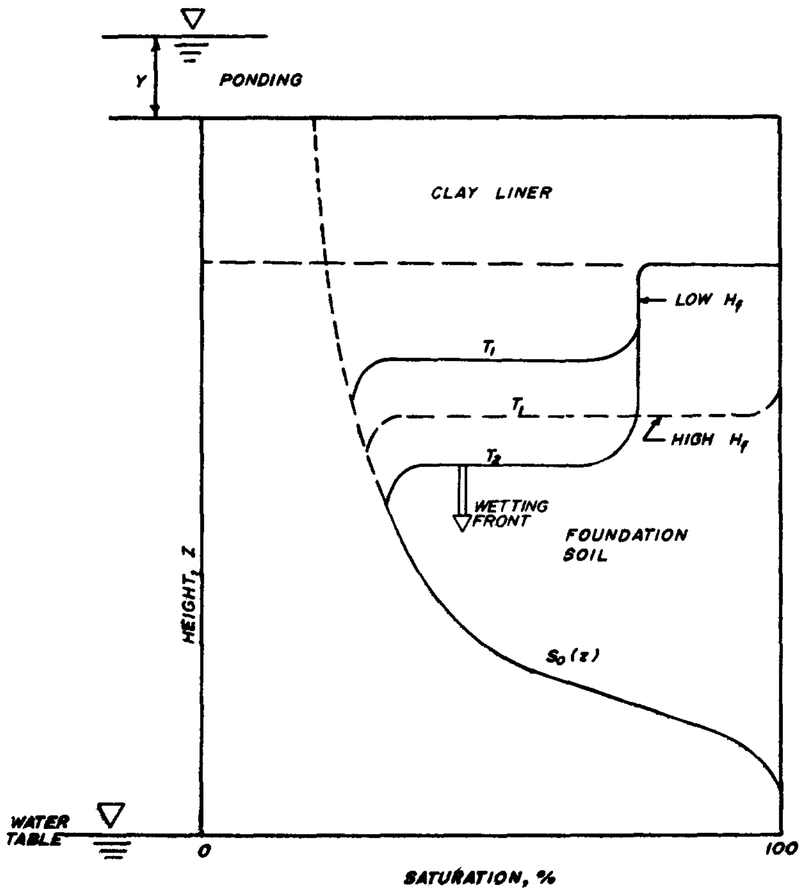

Fig. 9. Wetting front advance through foundation soil under a lined impoundment.

In a steady, saturated, series flow in one dimension, a volume balance across two layers labeled 1 and 2 shows that:

$$V = -K_1 i_1 = -K_2 i_2 \tag{18}$$

consequently

$$\frac{K_2}{K_1} = \frac{i_1}{i_2} \tag{19}$$

The permeability ratio, K_2/K_1, indicates the relative rate of head loss across each layer.

In a transient flow where the liner has been saturated, and the wetting front is advancing in the foundation, a similar concept can be applied. A method has been developed [41] to use the ratio of the saturated permeabilities and other readily estimated parameters to determine if seepage in the foundation is unsaturated. This analysis is then extended to solve for the value of the discharge.

When the pressure head at the interface between the liner and the foundation is less than $-h_d$, then the subgrade must be unsaturated (recall Fig. 2). For this condition to occur, the positive head at the top of the liner (h_p = Y) must be dissipated in flow through the liner.

The pressure head at the base of the liner is approximately defined by

$$h_f = (D_l + Y) - \left(\frac{K_s}{K_l}\right) D_l \tag{20}$$

where h_f = pressure head at the base of the liner or aquitard (L): D_l = liner thickness (L); K_s = saturated K of the foundation (L/T); and K_l = saturated K of the liner (L/T).

The first set of terms in brackets represents the hydrostatic (positive) component of the pressure at the base of the liner that would result if there were no flow. The second set of terms represents the head loss due to flow through the liner. Unless the pond is extraordinarily deep, it can be seen that h_f will be negative if the ratio of the permeabilities, K_s/K_l, is high. This ratio can easily have a value of 100/1 or more if the subgrade is granular (see Table 1). As noted previously, the lower the moisture content, the lower the effective permeability and seepage rate.

To effect an unsaturated seepage condition in the subgrade, it is more effective to make the liner less permeable than to increase its thickness [41]. Simply changing a specification to compact a clay soil in a very moist rather than a dry condition can cause up to tenfold reduction in permeability as a result of the change in soil structure [12]. This is obviously much more economical in the general case than making the liner ten times as thick. However, a more permeable but thicker clay liner does present more opportunities for dissolved contaminant attenuation, and seepage may still be restricted sufficiently to allow significant dilution in the aquifer [5].

Anaerobic conditions in septic tank leaching fields result in clogging of pores with biomass and decomposition products [32]. Thus a high permeability ratio results, considering the anaerobic zone under the leaching field and the subgrade below it as separate strata. A restricted leaching capacity results, and proper stabilization of wastes in the unsaturated subgrade may or may not occur. Aerobic conditions can be restored by a rest period during which flow is diverted to another leaching field [43]. Similar dosage and rest cycles are used in land treatment of wastewaters for the same purpose [31].

Summary and conclusions

Darcy's formula applies to both the saturated and the unsaturated zones of a geologic formation. This relationship specifically describes the flow of fluids due to hydraulic (liquid) or pressure (gas) gradients. The discharge and velocity values obtained are used in the computation of contaminant convection, dispersion and dilution. In the unsaturated zone, the discharge and the degree of saturation are related. Therefore, Darcy's formula can also be used to indicate the conditions of saturation that are important in various subsurface attenuation processes. The rate and quality of seepage passing through the vadose zone is that which actually enters a saturated aquifer from the surface or near-surface source. Consequently, Darcy's formula and its application to the vadose zone are critical in assessing the impact of waste disposal and treatment sites on groundwater. The same principles also apply to the effects of surface spills and infiltration from hazardous material storage areas.

Several applications of Darcy's formula to simplified models of practical situations were shown herein. They show that the basic nature of the flow regime in the vadose zone can be assessed with limited data. Such analysis could include definition of the distribution of pressure, magnitude of the discharge and seepage velocity, and an indication of whether the seepage is saturated or unsaturated.

Many of the hydraulic properties can be derived from static capillary-retention curves. However, these and other parameters change with the installation of the source facility or seepage from it.

In conclusion, this two-part paper outlined many of the salient features of vadose zone concepts as applied to both static and active seepage conditions. By no means, however, is this meant to imply that the work is essentially complete. Indeed, applications of the basic concepts of the vadose zone have been developed in other fields (agriculture, petroleum, etc.) are only in a rudimentary stage with respect to groundwater pollution. It is necessary to advance the knowledge and technology in general, and in particular to catch up with the present state-of-the-art in saturated aquifer analysis. To accomplish this, the following items (among others) need a concerted effort to determine:

1. The influence of the degree of saturation on contaminant attenuation processes.
2. Multi-dimensional unsaturated seepage and concurrent air flow.
3. Dispersion and diffusion in the vadose zone.
4. Relationships between air permeability, water permeability and gas or solute diffusion coefficients.
5. Influence of chemical interactions between soil and leachate on the unsaturated permeability and moisture retention.
6. Effects of immiscible fluids on leachate seepage.
7. Permanency of contaminant immobilization in the vadose zone by adsorption, precipitation and cation exchange.

Hopefully. this two-part overview has helped to set a stage with which to make progress in the directions cited above, as well as being useful for current applications.

List of symbols

A area
B unsaturated permeability function exponent
g gravitional constant
H_{cr} critical height
h total (hydraulic) head
h_d height of capillary fringe; displacement pressure head
h_p fluid pressure head
h_z elevation head
i total head gradient
K permeability to water (hydraulic conductivity)
K_e effective (unsaturated) water permeability
K_f general fluid conductivity
K_w saturated permeability
k intrinsic permeability
L_f depth of wetting front
l distance along flow line
n porosity
Q discharge
S saturation (water)
S_e effective saturation
S_r residual displacement saturation
T_s surface tension
u_a air pressure (gage)
u_w water pressure
V Darcy velocity
V_s seepage velocity
Y depth of ponding

γ unit weight
ρ mass density
μ dynamic viscosity

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