# Eleventh Canadian Geotechnical Colloquium:[1] Contaminant migration through groundwater—the role of modelling in the design of barriers

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The role of analysis in the evaluation and design of barriers is discussed. Factors considered include (i) the mechanisms controlling contaminant migration through barriers; (ii) the determination of diffusion and distribution coefficients; (iii) leachate mounding and the effect of clogging of leachate collection systems upon contaminant migration through barriers; (iv) the importance of considering the finite mass of contaminant available for transport into the soil and a method of modelling the effect of finite mass of contaminant; and (v) examples of how analysis may improve the geotechnical engineer's feel for the effectiveness of potential contaminant attenuation mechanisms in both glacial till deposits and fractured rock.

*Key words*: contaminant migration, analysis, diffusion, advection, clays, groundwater, fractured rock, design, barriers.

Le rôle que l'analyse peut jouer dans l'évaluation et la conception de coupures étanches est discuté. Les facteurs considérés comprennent (i) les mécanismes contrôlant la migration des contaminants à travers les coupures; (ii) la détermination des coefficients de diffusion et de distribution; (iii) la formation de mottes de lixiviat et l'effet du colmatage des systèmes de collecteurs de lixiviat sur la migration du contaminant à travers les coupures; (iv) l'importance de considérer la masse finie de contaminant disponible pour le transport dans le sol et une méthode pour modeler l'effet de la masse finie de contaminant; et (v) des exemples de la façon que l'analyse peut améliorer le sentiment que l'ingénieur géotechnicien a quant à l'efficacité des mécanismes d'atténuation du contaminant potentiel tant dans les dépôts de tills glaciaires que dans la roche fracturée.

*Mots clés* : migration de contaminant, analyse, diffusion, advection, argiles, eau souterraine, roche fracturée, conception, coupures.

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## Introduction

The potential impact of waste disposal on groundwater quality is a major consideration in the design, evaluation, and certification of waste disposal facilities. Invariably, this impact will be dependent on the effectiveness of a "barrier" that separates the waste from the general groundwater system. As will be discussed in the following sections, the nature of the barrier may vary substantially from case to case. In general, these barriers will not completely "contain" all potential contaminants and some contaminant may be expected to pass through the barrier. This situation is well recognized and "reasonable-use" policies for groundwater have been proposed by authorities such as the Ontario Ministry of the Environment (MOE 1986). If one accepts this approach, then the fact that some contaminant may escape is not, in itself, a cause for concern provided that the impact on off-site groundwater quality is within allowable limits. Implicit in this approach is a requirement that one be able to quantify the potential impact of a proposed design, and that monitoring and contingency measures be proposed to detect and rectify unacceptable levels of contamination before off-site impact has occurred. Thus, theoretical modelling has an important role to play in the design of these barrier systems. This role may include the determination of some relevant design parameters from laboratory tests and the evaluation of the potential impact of a proposed design on groundwater quality both under working conditions and in the event of a failure of part (or all) of the engineered system.

The state-of-the-art has now advanced to a point where modelling can be used as an aid to engineering judgement in most landfill designs. Clearly, however, the results of an analysis are only as good as the assumptions and parameters on which it is based. There will always be some uncertainty associated with parameters and design assumptions, and for this reason, analysis will be most useful for performing sensitivity studies to examine the implications of different design scenarios and for determining the potential effects of uncertainty regarding key parameters (see, e.g., Frind 1987).

Thus, the objective of this paper is to discuss the role of analysis in the evaluation of barriers and to highlight some important considerations. This discussion will include comments on the mechanisms controlling contaminant migration through barriers; the determination of relevant parameters; leachate mounding and the effect of clogging of the leachate collection systems upon contaminant migration through the barrier; the importance of considering the finite mass of contaminant available for transport into the general groundwater system; different methods of analysis; and examples of how analysis may improve the geotechnical engineer's "feel" for the effectiveness of potential contaminant attenuation mechanisms in both glacial till deposits and fractured rock.

## Barriers

The impact of a waste disposal facility on groundwater quality will depend on the nature of the site, the type of waste, the local hydrogeology, the presence of a dominant flow path, and, perhaps most importantly, the nature of the barrier that is intended to limit and control contaminant migration. Barriers

[1]Editor's footnote: The Canadian Geotechnical Colloquium is presented annually by a young engineer to the annual meeting of the Canadian Geotechnical Society. The subject of the lecture, as chosen by the Associate Committee on Geotechnical Research of the National Research Council of Canada, is one of national interest and importance.

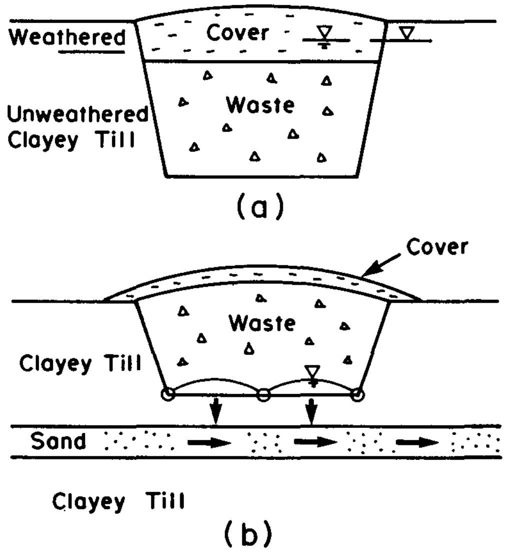


FIG. 1. Natural clayey barriers: (*a*) deep natural clayey deposit; (*b*) clayey barrier underlain by an aquifer.

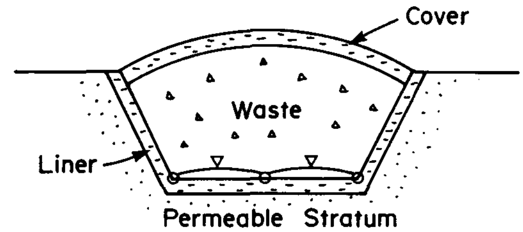


FIG. 2. Compacted clay liner separating waste from the general groundwater system.

will usually fall within one of the following categories: (i) natural clayey deposits (e.g, clayey till), (ii) compacted clayey liners, (iii) cutoff walls and permeable surrounds, (iv) natural rock deposits, and (v) synthetic liners (including geomembranes, spray-on asphalt, hydraulic asphalt concrete, etc.).

*Natural clayey deposits*

Natural clay deposits (see Fig. 1*a*) can provide an almost ideal barrier in many situations. Provided the site is selected or designed to ensure low hydraulic gradients, then the movement of contaminants will be slow and primarily controlled by diffusion. The clay itself can also act as an important medium for the attenuation of many contaminants, by processes such as sorption, precipitation, and biodegradation. Nevertheless, migration does occur and it is important to be able to estimate the rate of movement and the potential impact on surface waters (e.g., due to migration through the cover: Fig. 1*a*) and on any underlying groundwater resources (due to migration downward through the barrier and into an underlying aquifer: Fig. 1*b*.

The diffusive movement of contaminants through saturated clayey deposits is well understood and has been demonstrated by the fieldwork of Quigley and his co-workers (e.g., Goodall and Quigley 1977; Crooks and Quigley 1984; Quigley and Rowe 1986). Furthermore, the research by Desaulniers *et al.* (1981) has shown that natural diffusion profiles established over thousands of years are consistent with simple theoretical predictions.

Analysis of contaminant migration for systems such as that shown in Fig. 1 is the primary focus of this paper.

*Compacted clay liners*

Compacted clay liners (see Fig. 2) have been the subject of much recent debate with respect to both the hydraulic conductivity that can be achieved in the field (e.g., Day and Daniel 1985 and related discussion) and the potential impact of soil–leachate interaction upon hydraulic conductivity (e.g., Green *et al.* 1981; Brown and Anderson 1983; Brown *et al.* 1983, 1984; Fernandez and Quigley 1985; Bowders and Daniel 1987). However, experience to date would suggest that with good engineering practice and quality control, good-quality, low hydraulic conductivity compacted clay liners can be constructed (see, e.g., Reades and Thompson 1984; Reades *et al.* 1987; Richards and Thompson 1987). Furthermore, it would appear that although some very concentrated organic wastes may increase the hydraulic conductivity of clay, problems due to large increases in hydraulic conductivity can be avioded provided one considers clay–leachate compatibility in the selection of the liner material (see, e.g., Fernandez and Quigley 1988; Quigley and Fernandez 1987; Quigley *et al.* 1987*a*).

As in the case of a well-designed natural clayey barrier, the primary transport mechanism through a well-designed compacted clayey barrier will usually be diffusion.

*Cutoff walls and permeable surrounds*

Cutoff walls are most commonly used to limit contaminant migration from existing sites that have not been adequately designed; however, they may also be to control migration from new sites where it may be desirable to isolate groundwater in a relatively thin and shallow aquifer beneath the landfill. For example, in the case shown schematically in Fig. 3*a*, the thickness of the natural clay barrier may not be enough to prevent potential contamination of water flowing along the underlying minor aquifer. By constructing cutoff walls around the site and hence locally reducing the flow in the aquifer, it is possible to change an advection-controlled system beneath the landfill into a diffusion-controlled system, thereby substantially reducing the impact on off-site groundwater quality. It is, of course, still necessary to consider diffusive migration through the cutoff wall and into the aquifer. This can be achieved using techniques similar to those that will be discussed for natural clayey barriers.

An interesting variation of this concept, developed by Matich and Tao (1984) and referred to as the "pervious surround concept," involves minimizing advective transport through a waste pit by surrounding it with a multilayered pervious envelope with less permeable material adjacent to the waste and more permeable material outside of this, as shown schematically in Fig. 3*b*. In this way, water flow is directed around the outside of the pit rather than through the pit, and contaminant migration would be predominantly by diffusion from the waste through the less permeable material together with advective dispersive transport within the more permeable outer zone. Thus, from the standpoint of modelling, determination of contaminant loading of the groundwater for this case is also very similar to that for waste sites separated from an underlying aquifer by a clayey barrier as shown in Fig. 1*b*.

*Natural rock deposits*

A topic of particular interest in southern Ontario is the migration of contaminants from existing or proposed landfills excavated into, or sitting on top of, fractured rock. Typically,

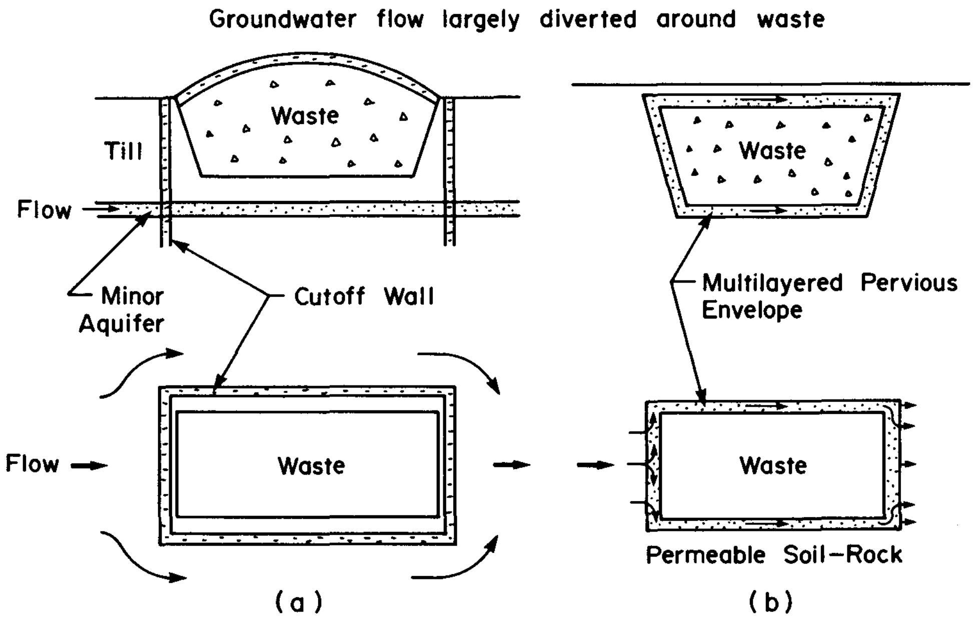


FIG. 3. Schematic drawing showing (*a*) cutoff wall is used to divert groundwater flow from beneath a natural clayey barrier and (*b*) pervious material placed outside the waste to divert groundwater flow around (rather than through) the waste.

the intact rock has a very low hydraulic conductivity and contaminant migration will primarily involve advective–dispersive transport along the fractures in the rock (see Fig. 4). In these cases, the primary mechanism limiting the movement of contaminant is the process of matrix diffusion whereby contaminant is removed from the fracture as it diffuses into the matrix of the rock. Monitoring of an existing landfill at Burlington, Ontario (Gartner Lee and Associates Ltd. 1986), suggests that during 15 years of migration, contaminant movement downgradient of the Burlington landfill has been very slow. The evidence would suggest that substantial attenuation has occurred. Attenuation in fractured rock will be discussed in more detail at the end of the paper.

*Synthetic liners*

Synthetic liners are usually relatively thin and are frequently used in double-liner systems such as that shown in Fig. 5 (note that the dimensions shown in Fig. 5 will vary from case to case). Synthetic liners were discussed in some detail by Folkes (1982) and although analysis does have a role to play in the design of these barriers, the role is somewhat different to that applicable to the other four barrier systems described above and will not be considered further in this paper. The interested reader is referred to Geoservices Inc. (1987*a*, *b*) for background information on double-lined landfills.

## Transport mechanisms and governing equations

Evaluation of the design for a waste disposal facility involves making a quantitative prediction of potential impact of the waste on groundwater quality, keeping in mind that under most circumstances involving movement through a barrier and into an aquifer, the best one can expect to do is to predict trends and a likely range of concentrations at any given point in space and time.

There are four aspects of any attempt to make quantitative predictions, namely,

—to identify the controlling mechanisms;
—to formulate or select a theoretical model;
—to determine the relevant parameters;
—to solve the governing equations.

In the case of contaminant transport through saturated clayey barriers or the matrix of an intact rock (e.g., shale), the primary transport mechanisms are advection and diffusion. Advection involves the movement of contaminant with flowing water, whereas diffusion involves the movement of contaminant from points of high chemical concentration to points of low chemical concentration (see, e.g., Freeze and Cherry 1979; Quigley *et al.* 1987*b*). In the mathematical modelling of contaminant migration, these transport mechanisms are combined with the requirement that there be conservation of mass within any small region, and for one-dimensional conditions this gives rise to a differential equation:

$$[1] \quad n\frac{\partial c}{\partial t} = \left(nD\frac{\partial^2 c}{\partial z^2} - nv\frac{\partial c}{\partial z}\right) - \rho K\frac{\partial c}{\partial t}$$

where $n$ = effective porosity of the soil, $c$ = concentration at depth $z$ and time $t$, $D$ = diffusion coefficient, $v$ = seepage (average linearized groundwater) velocity, $\rho$ = soil dry density, $K$ = distribution or partitioning coefficient, and $v_a = nv$ is the Darcy or discharge velocity. This equation simply states that the increase in contaminant concentration within a small volume of soil is equal to the increase in mass due to advective-diffusive transport minus the mass of contaminant removed from solution by what are loosely referred to as "sorption processes." Here, it is assumed that the sorption processes are linear and can be represented in terms of a partitioning or distribution coefficient $K$ (see Freeze and Cherry for more details regarding "sorption processes"). The product $\rho K$ is a dimensionless measure of the amount of sorption that is likely to occur. A contaminant species is said to be "conservative" if there is no sorption (i.e., $\rho K = 0$).

When modelling contaminant migration along an aquifer, equations such as [1] (or its two- or three-dimensional counterparts) may be used; however, in this case the parameter $D$ is given by

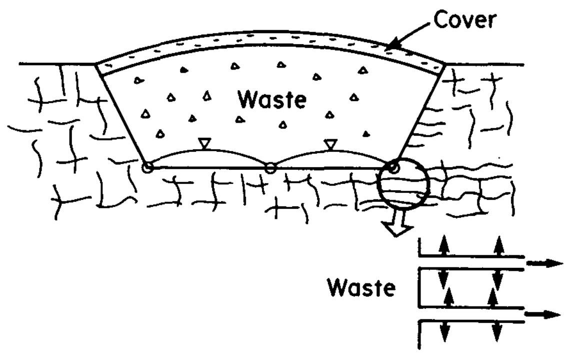


FIG. 4. Landfill located in fractured shale. Contaminant transport along the fractures is attenuated by diffusion into the matrix of the shale adjacent to the fractures.

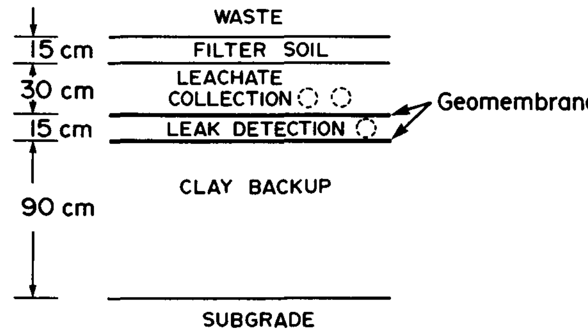


FIG. 5. Geosynthetic liner system with a compacted clay backup.

[2] $D = D_e + D_m$

where $D_e$ = the diffusion coefficient for the species of interest and $D_m$ = the coefficient of mechanical dispersion. Thus, here the coefficient $D$ (often called the "coefficient of hydrodynamic dispersion") incorporates two transport mechanisms (diffusion and mechanical dispersion) that are inherently very different, although for most practical purposes they can be mathematically modelled in the same way and hence are lumped together as a composite parameter. For transport through intact clayey soil, diffusion will usually control the parameter $D$ and dispersion is negligible (see, e.g., Gillham and Cherry 1982; Rowe 1987). In aquifers, the opposite tends to be true and dispersion tends to dominate. It is often convenient to model the dispersive process as a linear function of velocity (Bear 1979; Freeze and Cherry 1979), viz.,

[3] $D_m = \alpha v$

where $\alpha$ = dispersivity, although the dispersivity $\alpha$ tends to be scale dependent and is not a true material property (see, e.g., Gillham and Cherry 1982).

By invoking conditions of continuity of contaminant concentration and flux (flux is the mass of contaminant transported through a unit area in a unit time) between layers, it is then possible to take equations such as [1] (together with [2] and [3], where appropriate) and develop a system of equations for a layered system. Details of how this can be done for one-, two- and three-dimensional layered systems have been described by Rowe and Booker (1987). A similar procedure can be followed to develop equations for matrix diffusion from a series of fractures (see, e.g., Tang *et al.* 1981; Sudicky and Frind 1982; Rowe and Booker 1988*a*). When combined with appropriate boundary and initial conditions, this gives rise to a "theoretical model." It is then necessary to determine appropriate parameters and, finally, to solve the governing equations. This will be discussed in the following sections.

## Determination of parameters

Inspection of [1]–[3] indicates that there are a number of parameters that must be determined or estimated before any quantitative predictions can be attempted.

The advective transport through the soil and the initial concentration and mass of contaminant will be discussed in the following two sections. In this section attention will be focussed on the remaining parameters.

The diffusion coefficient, the partitioning or distribution coefficient (expressed in terms of the dimensionless product $\rho K$) and the effective porosity can be estimated from laboratory tests. For example, Rowe *et al.* (1985, 1988) have developed a technique that allows the determination of these parameters using "undisturbed" samples of the proposed barrier material. The approach involves placing the soil in contact with a source solution containing a known initial concentration and mass of the contaminant species of interest, as shown in Fig. 6. The boundary conditions at the bottom of the soil may vary depending on what parameters are to be determined and on the contaminants being examined.

Typically, the base will be either an impermeable barrier (Fig. 6*a*) or a collection chamber containing a reference fluid (in some cases a fluid with pore-water chemistry similar to that of the soil) (Fig. 6*b*). Contaminant is allowed to migrate from the source chamber through the soil into, if present, the collection chamber. If no additional contaminant is added to the source chamber, then the concentration of contaminant will decrease with time as mass of contaminant diffuses into the soil (see Fig. 7). (The rate of decrease can be controlled by the choice of the height of leachate $H_f$; see Fig. 6.) Conversely, as contaminant diffuses into the collection chamber (Fig. 6*b*), the increase in mass gives rise to an increase in contaminant concentration in this reservoir. The rate of decrease in concentration in the source and increase in the collection chamber should be monitored with time. At some time $t_f$, the test is terminated and the concentration profile through the soil sample may be determined. Assuming linear sorption (i.e., [1]), theoretical models can then be used to estimate the parameters $n$, $D$, and $\rho K$ from this test data, as described by Rowe *et al.* (1988) and Barone *et al.* (1988).

## Advective transport and leachate mounding

In geotechnical engineering, advective transport through the soil is typically represented in terms of the Darcy or discharge velocity $v_a$. This, of course, depends on the hydraulic conductivity ("permeability") of the soil and the hydraulic gradient. Folkes (1982) has discussed hydraulic conductivity in some detail and therefore the determination of this parameter will not be discussed in detail herein. However, it is noted that when dealing with the impact of a landfill on groundwater quality (and hence public health), the geotechnical engineer should be mindful of uncertainty regarding this parameter as well as the implications that the uncertainty might have. When determining bearing capacity of foundations or stability of slopes it is common practice to apply a partial factor to one's expected undrained shear strength (i.e., one does not design with a factor of safety of 1 when the consequences of failure are significant). For similar reasons, it would seem appropriate to design

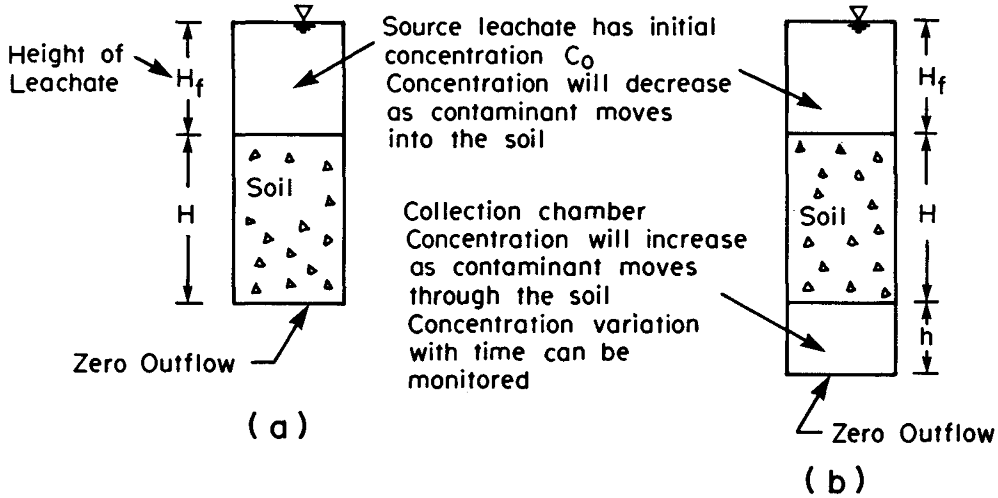


FIG. 6. Schematic of tests used to determine the diffusion and distribution coefficients: (*a*) zero flux at base of the soil; (*b*) migration into a collection chamber.

landfills such that an appropriate "partial factor" has been applied to the parameters such as the hydraulic conductivity when assessing the implications of a design on groundwater that is being (or will be) used for domestic or agricultural use.

As in many aspects of geotechnical engineering, the level of uncertainty is related to the level of control that the engineer may have over the uniformity of the material. For compacted clay barriers constructed with sound quality control and quality assurance procedures, the hydraulic conductivity of the liner material obtained from laboratory tests exhibits a log normal distribution[2] and in these cases it may be reasonable to design on the basis of the geometric mean[2] of the hydraulic conductivity data, recognizing that the harmonic mean (which controls flow through the liner) will be less than the geometric mean (see, e.g., Richards and Thompson 1987). In these cases, the partial factor applied to the geometric mean might be unity.

When dealing with natural deposits (e.g., glacial till) where there is a wide variation in field test data within the proposed barrier layer (e.g., till), there cannot be the same confidence in the applicability of the geometric mean based on relatively few tests as there would be for the compacted liner such as that used at Keele Valley (see, e.g., Richards and Thompson 1987). In these cases it would be prudent to apply a partial factor to the geometric mean in determining a design parameter for hydraulic conductivity. This partial factor could be related to the statistical distribution of the available data and would reflect the uncertainty associated with the choice of a hydraulic conductivity for the critical barrier materials beneath the site.

The Darcy velocity (and hence the seepage velocity) will also depend on the hydraulic gradient. The construction of a landfill will frequently change the hydraulic conditions at a site. For example, modern landfills are commonly designed to have a leachate underdrain system. These collection systems may serve several functions. Firstly, by lowering the height of leachate mounding, leachate seeps (and consequent contamination of surface waters) can be minimized. Secondly, by reducing the head in the leachate, the hydraulic gradient through the underlying barrier and the Darcy velocity out of the landfill can be reduced to acceptable levels. Thirdly, by removing contaminant from the landfill, the mass of contaminant available for transport into the hydrogeological system will be reduced.

Various methods of estimating the height of the leachate mound (and hence the head within the landfill) have been proposed. Once the height of leachate mounding has been calculated, it is a relatively straightforward geotechnical calculation to estimate the average hydraulic gradient and Darcy velocity through the barrier. As demonstrated in the following section, it is then possible to estimate the impact of the leachate collection system on the mass of contaminant available for transport into the barrier. The following paragraphs will discuss the methods available for estimating the height of leachate mounding.

If the barrier beneath the waste is flat and relatively impermeable compared with the waste, then the height $h$ of the mound between two drains separated by a distance $l$ (see Fig. 8*a*) may be estimated from an equation, given by Harr (1962), that (on simplification) can be written as

$$[4a] \quad h = \sqrt{\Omega}\,\sqrt{(l - x)x}$$

$$[4b] \quad \Omega = q_0/k$$

where $h$ is measured relative to the head at the drawdown points, $l$ is the spacing between drawdown points, $x$ is the distance from one of these points, $q_0$ is the steady-state infiltration, and $k$ the hydraulic conductivity of the layer in which mounding is occurring (e.g., the waste). At the midpoint between drawdown points, this equation reduces to

$$[5] \quad h_{\max} = 0.5l\sqrt{\Omega}$$

where $h_{\max}$ is the maximum height of mounding above the barrier. Based on the variation in $h$ with position $x$, it can be shown that the average value of $h$ is given by

$$[6] \quad \bar{h} = 0.785h_{\max} = 0.393l\sqrt{\Omega}$$

Using the same basic assumptions as adopted in the development of [5], Moore (1983) developed a solution for the case of a sloping geometry as indicated in Fig. 8*b*, and derived an

[2]If one plots frequency of occurrence of a given hydraulic conductivity versus the logarithm of hydraulic conductivity, it approaches a normal distribution. If the mean of this data is $x$ (in log units), the geometric mean is $k = 10^x$.

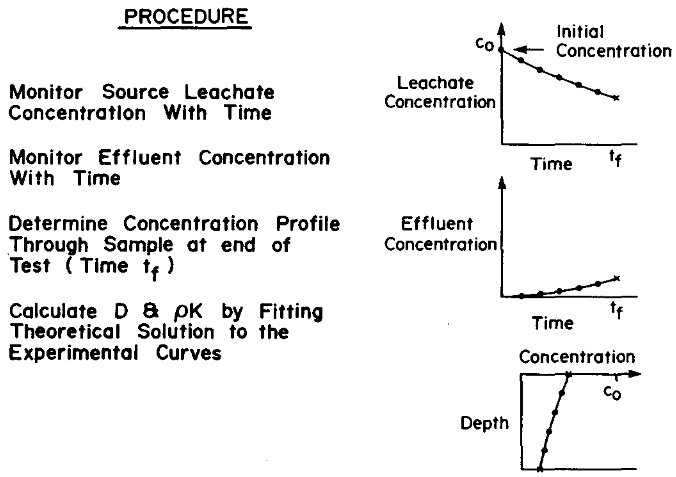


FIG. 7. Experimental procedure used to determine the diffusion coefficient $D$ and distribution coefficient $K$ from the test shown schematically in Fig. 6*b*.

equation for the maximum height of mounding above the barrier, $h_{max}$, viz.,

[7] $$h_{max} = 0.5l(\sqrt{\Omega + s^2} - s)$$

where $s = \tan \alpha$ is the slope of the barrier.

McBean *et al.* (1982) considered a different sloping collection system as shown in Fig. 8*c*. This case is somewhat more complicated than the previous two cases and one cannot write a simple explicit equation for the height $h$ of leachate above the barrier. However, assuming zero pressure head within the drains, the height $h$ and the distance at which it occurs can be related by the equation

[8*a*] $$x = \lambda(1 - A \exp B)$$

where

[8*b*] $$A = \frac{\Omega^{1/2}}{\left[\frac{h^2}{(\lambda - x)^2} - \frac{sh}{(\lambda - x)} + \Omega\right]^{1/2}}$$

[8*c*] $$B = \frac{s}{\sqrt{4\Omega + s^2}} \times \left[\tan^{-1}\left(\frac{-s}{\sqrt{4\Omega + s^2}}\right) - \tan^{-1}\left(\frac{\frac{2h}{(\lambda - x)} - s}{\sqrt{4\Omega + s^2}}\right)\right]$$

As noted by McBean *et al.*, the leachate mounding between drains can be readily calculated by determining the location $x$ for an assumed height of mounding $h$ using a successive substitutions algorithm. However, to do so, one must first know the value of $\lambda$. McBean *et al.* recommend that this be done by a process of trial and error: first, estimate $\lambda$; second, calculate the leachate mound to the right of the lower drain (see Fig. 8*c*) over a distance $\lambda$; third, calculate the leachate mound to the left of the upper drain over a distance $l - \lambda$ (noting that both positive and negative slopes $s$ are permitted); fourth, if the height $h$ at the location $\lambda$ (calculated in the second and third steps) do not agree, then revise the estimate of $\lambda$ and repeat the procedure until the calculated mound is continuous between the two adjacent drains.

Equations [4]–[8] have been developed assuming that the

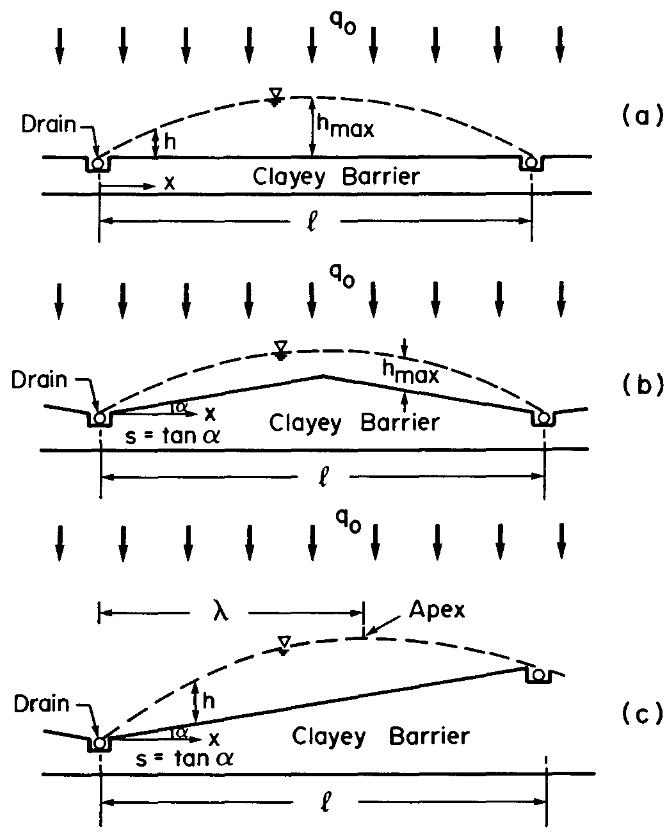


FIG. 8. Schematic showing different collection sytems.

infiltration $q_0$ is equal to the flow passing into the collection points. The use of these equations for estimating the mound height when a portion of the water moves down through the barrier involves some approximation, since, under steady-state conditions, the acutal flow to the collection points is equal to the difference between the infiltration $q_0$ and the flow into the barrier $q_i$ (i.e., is equal to $q_0 - q_i$). Thus, these equations provide a very approximate (conservative) estimate of the head above the barrier.

More elaborate equations for estimating the height of leachate mounding have been proposed (see, e.g., Wong 1977; Demetrocopoulos *et al.* 1984). These equations have been developed to consider leakage through the barrier and in this sense are more realistic than [4]–[8]. However, to make the problem tractable, a number of other assumptions have been made. For example, Wong assumes, *inter alia,* that the leachate collection system is above the water table; the effects of groundwater flow beneath the barrier are negligible; and the leachate instantly saturates a rectilinear volume above the liner and retains this shape while draining towards the collection drain and through the barrier. The adoption of these more elaborate equations is only likely to improve the estimate of leachate mounding if these assumptions are reasonably applicable for a given situation. In many cases they are not. If the hydraulic conductivity of the barrier is significantly lower than that of the overlying waste and drainage layers, then it may be appropriate to use the simple equations given above ([4]–[8]), recognizing the simplifications involved. If the hydraulic conductivity of the "barrier" (i.e., the soil beneath the collection drains) is of an order similar to that of the waste, then the simplified equations are not valid and it may be necessary to use numerical methods (e.g., the finite element technique) to model the entire flow system (i.e., waste, barrier, and underlying hydrostratigraphy). The results from some preliminary research in this area have recently been described by St. Arnaud *et al.* (1986).

Irrespective of the method of analysis or equation used in calculating the leachate head, probably the greatest uncertainty is associated with the hydraulic conductivity of the waste. To date, relatively little research has been conducted into the determination of appropriate values for use in design. A hydraulic conductivity commonly used in design is $10^{-4}$ cm/s. Recognizing that waste is likely to be heterogeneous, it is not surprising that the limited number of field measurements (e.g., Hughes *et al.* 1971; Page *et al.* 1982) indicate a wide range ($9 \times 10^{-6} - 8.5 \times 10^{-3}$ cm/s) of hydraulic conductivities. Furthermore, if the waste is anisotropic, then these values may not be representative of the parameters controlling the leachate mounding. Consequently, the design of leachate collection systems and the calculation of potential impact on groundwater quality should involve some consideration of the implications of uncertainty regarding the hydraulic conductivity of the waste.

*Failure of collection systems*

The foregoing discussion has revolved around the calculation of leachate mounds assuming that the leachate collection system is functioning. However, reviews of the potential for clogging of these systems (e.g., Bass *et al.* 1984) have indicated that clogging may, in fact, occur owing to physical, chemical, biochemical, and biological mechanisms. Based on their study, Bass *et al.* concluded, *inter alia,* that "it is reasonable to expect clogging to occur in a probabilistic manner during the active and post-closure operational lifetime."

Recent research reported by Puig *et al.* (1986) and Cancelli and Cazzuffi (1987) indicates that clogging of both drains and geotextile filters can occur very quickly. Puig *et al.* reported a field case where a drain clogged and was inoperative within 1 month of being laid. Cancelli and Cazzuffi have shown that in laboratory tests using landfill leachate, clogging of geotextile filters can reduce the hydraulic conductivity of the filter from initial values of $10^{-2}-10^{-3}$ cm/s to values of $10^{-6}-10^{-7}$ cm/s after a relatively short time.

It should be noted that clogging of both pipes and filters is likely to be highly dependent on factors such as whether aerobic or anaerobic conditions prevail and the flow rate through the pipe or filter. For example, the bacterial growth under the anaerobic conditions must commonly expected below landfills may be different to that encountered by Puig *et al.* (1986) or the time scale for clogging of geotextile filters is likely to be longer in the field than in the laboratory because of the lower field flow rates. The findings of Cancelli and Cazzuffi indicated that geotextile filters may clog; however, this should not be taken as implying that granular filters are therefore superior, since it might be anticipated that they can also clog as a result of similar mechanisms. Clearly, care should be taken in extrapolating the findings of these two papers to landfill underdrain systems. Nevertheless, the findings of these two papers do indicate the need for concern regarding the potential for clogging and also the need for additional research.

It is the author's opinion that clogging of the drains and failure of part (or all) of the leachate collection system should be considered in the evaluation of contingency measures at a waste disposal site. Also, given the opportunity to do so, it would seem prudent to locate a landfill in an environment such that if the leachate underdrain system fails, the impact of leachate on potable groundwater will be minimal.

## Mass of contaminant

The impact of a waste disposal site upon groundwater quality is usually judged by monitoring the concentration of potential contaminants at a number of specific monitoring points. As will be demonstrated, the variation in concentration with time will be a function of the mass of contaminant in the system, the infiltration through the cover, the proportion of leachate collected, and the proportion of leachate passing into the hydrogeological system.

Experience has shown that the concentration of potential contaminants increases during operation of the disposal facility, reaches a peak after closure, and then declines. The increase in concentration may be related to (i) the physical processes of leaching of contaminant from solid waste as water infiltrates through the waste and (or) (ii) chemical and biological processes that generate the chemical species of interest from the synthesis, or breakdown, of existing chemical species in the waste (e.g., biological action). Likewise, the decrease in concentration with time can be related to (i) the physical process of removal of contaminant, in the form of leachate, from the landfill and (or) (ii) chemical and biochemical processes that result in precipitation and (or) the synthesis or breakdown of the chemical species of intersest into other chemical forms.

In the design of barrier systems, it is generally not practical to model the details of the leaching processes and any associated chemical or biological processes. However, reasonable engineering approximations can be made that will allow the designer to obtain some insight regarding the potential impact of the finite mass of contaminant. Thus, for the purposes of performing design calculations, it is usually conservative to
(i) assume that the concentration of a contaminant of interest reaches the peak concentration, $c_0$, instantaneously;
(ii) assume that all the mass of this contaminant species, $M_{TC}$, is in solution at the time the peak concentration occurs;
(iii) neglect any decrease in concentration due to chemical or biological processes within the landfill itself.

The peak concentration, $c_0$, of a given contaminant species can usually be estimated from past experience with similar landfills. The total mass of contaminant is more difficult to determine. Nevertheless, upper-bound estimates can be made by considering the observed variation in concentration with time at landfills where leachate concentration has been monitored or by considering the composition of the waste.

Until fairly recently, there has been a paucity of data concerning the available mass of contaminants within landfills; however, this situation is changing now that many landfills have leachate collection systems. Given that concentration is simply mass per unit volume, the mass of a given contaminant collected in a year is equal to the average concentration multiplied by the volume of leachate collected. By monitoring how this mass varies with time, it is then possible to estimate the total mass of that species of contaminant within the landfill. In the absence of this information, studies of the composition of waste (e.g., Cheremisinoff and Morresi 1976; Kirk and Law 1985) can be used to estimate the mass of a given contaminant or group of contaminants. For example, Table 1 summarizes an estimate of refuse composite reported by Hughes *et al.* (1971). For contaminant species predominantly formed from breakdown or synthesis of other species (e.g., by biological

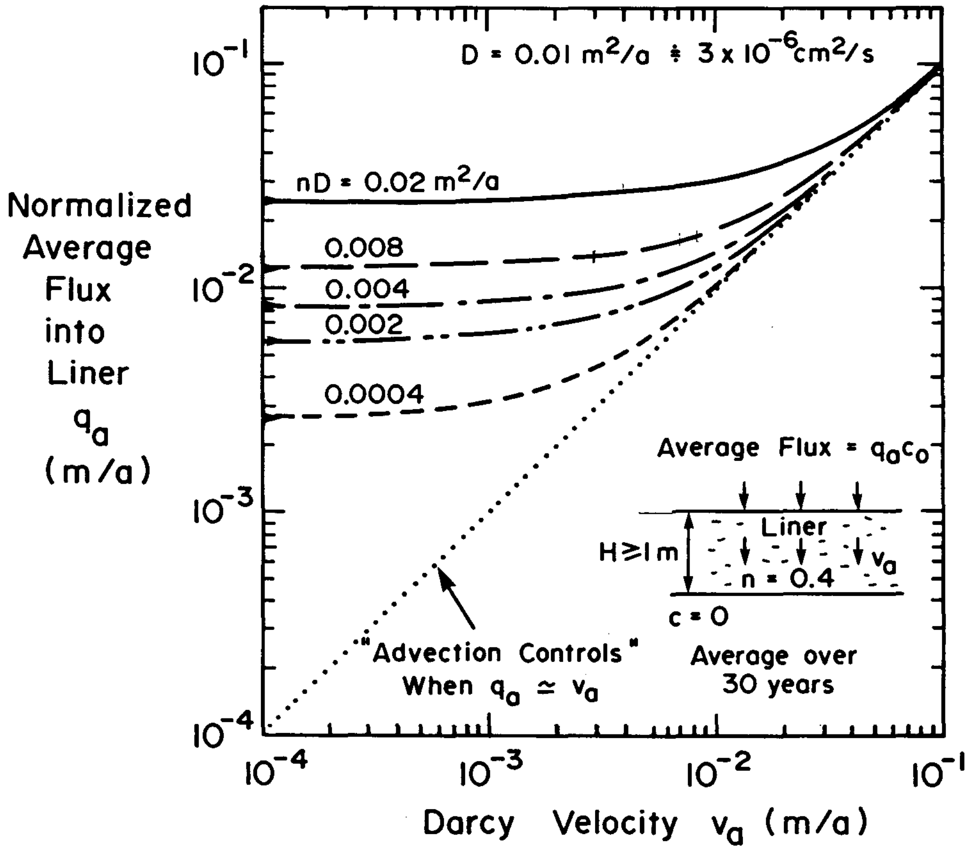


FIG. 9. Relationship between normalized average flux into a liner and the Darcy velocity for a range of diffusion coefficients ($\rho K = 0$).

action), an upper-bound estimate of the mass of contaminant may be obtained from the estimates of the mass of chemicals that form the derived contaminant.

For the purposes of modelling the decrease in concentration in the leachate due to movement of contaminant into the collection system and through the barrier, it is convenient to represent the mass of a particular contaminant species in terms of an "equivalent height of leachate," $H_f$, as described in the following subsections.

*Mass of contaminant available for transport,* $M_0$

Let the symbol $M_{TC}$ denote the total mass of a particular contaminant species present in the landfill. Now suppose that a proportion of this mass given by the ratio $q_a/q_0$ is transported into the hydrogeologic system (the remainder being collected by the leachate collection system), then mass available for contamination of groundwater, denoted by $M_0$, is given by

[9] $$M_0 = M_{TC}(q_a/q_0)$$

where $q_a/q_0 \leq 1$.

In this expression, $q_0$ represents the volume of leachate generated within the landfill (per unit area) and can usually be taken to be equal to the infiltration into the landfill. The quantity $q_a$ may be defined as the average mass flux into the barrier normalized (divided) by the average concentration within the landfill and referred to here as the normalized average mass flux. This quantity can be readily estimated from a simple hand calculation.

It will be shown that the normalized average flux, $q_a$, has units of velocity and, indeed, for situations where advection is the dominant transport mechanism, $q_a$ is approximately equal to the Darcy velocity, $v_a$ ($v_a = nv = ki$, where $k$ is the hydraulic conductivity of the barrier and $i$ is the outward hydraulic gradient in the barrier). However, for situations where diffusion is a significant transport mechanism, the normalized average flux, $q_a$, will be greater than the Darcy velocity and can be estimated from Fig. 9 as described subsequently.

*Mathematical definition of the normalized average mass flux,* $q_a$

The mass flux $f(\tau)$ of contaminant into the barrier beneath the landfill at time $\tau$ is given by

[10] $$f(\tau) = nvc - nD\left(\frac{\partial c}{\partial z}\right)$$

where $c$ and $\partial c/\partial z$ represent the concentration and concentration gradient at the boundary between the landfill and the barrier. Up to some time $t$, the total mass of contaminant passing into the soil is obtained by integrating [10]. Thus, the average flux, $\bar{\bar{f}}$, is obtained by dividing the total mass by the period $t$ to give

[11] $$\bar{\bar{f}} = \int_0^t \frac{f(\tau)}{t}\, d\tau$$

The normalized average mass flux into the barrier over this time period is then obtained by dividing the average mass flux by the average concentration $\bar{\bar{c}}$, viz.,

[12] $$\bar{\bar{c}} = \int_0^t \frac{c(\tau)}{t}\, d\tau$$

giving

TABLE 1. Refuse composition (after Hughes *et al.* 1971)

| Component | Milligrams pollutant/gram dry refuse or weight percent |
|---|---|
| Crude fiber | 38.3% |
| Moisture content | 18.2% |
| Ash | 20.2% |
| Free carbon | 0.57% |
| Nitrogen: | |
| Free | 0.02 mg/g |
| Organic | 1.23 mg/g |
| Water solubles: | |
| Sodium | 2.33 mg/g |
| Chloride | 0.97 mg/g |
| Sulfate | 2.19 mg/g |
| Chemical oxygen demand | 42.29 mg/g |
| Phosphate | 0.15 mg/g |
| Hardness | 10.12 mg $CaCO_3$/g |
| Major metals: | |
| aluminum, iron, silicon | >5.0% (by spectrographic analysis) |
| Minor metals: | |
| calcium, magnesium, potassium | 1.0–5.0% (by spectrographic analysis) |

$$[13a]\quad q_a = \frac{\bar{\bar{f}}}{\bar{\bar{c}}} = \frac{\int_0^t f(\tau)\,\mathrm{d}\tau}{\int_0^t c(\tau)\,\mathrm{d}\tau} = \frac{nv\int_0^t c(\tau)\,\mathrm{d}\tau - nD\int_0^t \left(\frac{\partial c}{\partial \tau}\right)\mathrm{d}\tau}{\int_0^t c(\tau)\,\mathrm{d}\tau}$$

For the case where advection governs, the ratio $D/v$ approaches zero and [13$a$] reduces to

$$q_a = \frac{nv\int_0^t c(\tau)\,\mathrm{d}\tau}{\int_0^t c(\tau)\,\mathrm{d}\tau} = \frac{nv\bar{\bar{c}}}{\bar{\bar{c}}} = nv$$

and, thus,

$$[13b]\quad q_a = nv = v_a$$

where $v_a$ is the Darcy velocity through the barrier. Clearly, $q_a$ has units of velocity.

For the case where advection governs, $q_a$ is independent of time once steady-state flow has been established. For other cases, $q_a$ will depend on time. The designer may examine the implications of variations in the choice of time period or the value of $q_a$ from [13] by using a theoretical solution to determine the average flux $\bar{\bar{f}}$ and the average concentration $\bar{\bar{c}}$. Thus, the normalized average mass flux can be directly determined for any given time period using a computer program (e.g., POLLUTE: Rowe *et al.* 1984). However, for many practical situations it is reasonable to consider a time period of 30 years and for this case an engineering estimate of the normalized average mass flux, $q_a$, can be simply obtained as described below.

*A simple engineering estimate of the normalized average mass flux,* $q_a$, *and the mass available for transport into the soil,* $M_0$

Consider a barrier of thickness $H$ as shown in the insert to Fig. 9. For the purposes of estimating the available mass $M_0$, it is conservative to determine the normalized average mass flux assuming (i) the concentration in the landfill remains constant and (ii) the concentration in the aquifer is zero. The results obtained for this case are shown in Fig. 9 for a barrier of thickness greater than or equal to 1 m. Here, the normalized average flux ($q_a$) is plotted against the Darcy velocity through the liner for a conservative species and a range of values of the product $nD$ where $n$ is the porosity of the barrier and $D$ is the effective diffusion coefficient of the contaminant being considered.

The results presented in Fig. 9 may be used for situations where the concentration in the leachate decreases with time and (or) the concentration beneath the barrier is greater than zero.

Advection controls contaminant transport when the normalized average flux ($q_a$) is approximately equal to the Darcy velocity ($v_a$). For typical situations involving clayey barriers this will be the case for Darcy velocities greater than 0.03 m/a. For velocities less than this, diffusion may noticeably affect the normalized average flux and by inspection of Fig. 9 it can be seen that for a given diffusion coefficient there is a minimum value of $q_a$. Thus, even if there were no flow into the soil (i.e., zero hydraulic gradient), contaminant would still pass into the barrier and the potential impact will depend on the mass of contaminant $M_0$ available for transport into the barrier.

The results given in Fig. 9 are based on an average mass flux over a 30 year time period. This is considered to be a reasonable averaging period for many practical situations. The normalized average mass flux will, in fact, decrease with increasing time period $t$. Putting aside the possibility that there may be inward flow, the minimum value of the flux will correspond to steady-state diffusion and can be calculated from [13$a$], viz.,

$$[13c]\quad q_a(\text{minimum}) = nD/H$$

The flux $q_a$ will tend to the value given by [13$c$] for large averaging periods (i.e., well in excess of 30 years). As can be seen from Fig. 9, the average over a 30 year period may considerably exceed the minimum value given by [13$c$]. Clearly, if there is flow into the landfill that opposes the outward diffusive flux, the normalized average flux would be even smaller than this (and can be calculated from [13$a$]).

*Example calculation of* $q_a/q_0$

To illustrate the use of Fig. 9, consider a clayey barrier with a porosity of 0.4 and a contaminant with an effective diffusion coefficient of 0.01 $m^2/a$ (i.e., $nD$ = 0.004 $m^2/a$):

(i) If the Darcy velocity into the barrier were 0.03 $m^2/a$, then from Fig. 9 the normalized average flux $q_a$ would be approximately 0.032 $m^2/a$. If the infiltration into the site $q_0$ was 0.3 m/a (a typical value used in the design of landfills in southern Ontario), then the proportion of mass entering the system $q_a/q_0$ would be 0.1 (i.e., $M_0 = 0.1M_{TC}$ or 10% of the total mass is available for transport into the barrier).

(ii) If the Darcy velocity was only 0.003 m/a, then from Fig. 9, $q_a \simeq 0.01$ m/a (for $nD$ = 0.004 $m^2/a$). Thus the tenfold decrease in Darcy velocity (compared with case (i)) has only reduced the normalized flux by a factor of 3 and in this case diffusion has a significant influence on contaminant transport into the barrier. Again assuming $q_0$ = 0.3 m/a, this case would correspond to $M_0 = 0.03M_{TC}$ or 3% of the total mass being available for transport into the barrier, the remaining 97% being collected by the leachate collection system.

*Equivalent height of leachate* $H_f$

If we assume that the available mass of contaminant, $M_0$, is in solution at the time that the peak concentration, $c_0$, occurs in the leachate, then this mass can be represented as an "equivalent volume of leachate," $V_0$, that is,

[14] $V_0 = M_0/c_0$

In general, this volume will not correspond to the actual volume of leachate because (i) it only represents the portion of the total mass available for transport into the hydrogeologic system and (ii) it is based on the conservative assumption that all this available mass can be quickly leached from the solid waste. It is convenient for mathematical and physical reasons to express the volume $V_0$ in terms of an "equivalent height of leachate," $H_f$, where $H_f$ is defined as the volume of leachate divided by the area, $A_0$, through which contaminant passes into the primary "barrier," that is,

[15*a*] $H_f = V_0/A_0$

or

[15*b*] $$H_f = \frac{M_{TC}}{c_0 A_0}\frac{q_a}{q_0}$$

If the hydrostratigraphy and volume of leachate passing into the barrier (per unit area) is the same beneath the entire site under consideration, then the area $A_0$ would generally correspond to the plan area of the landfill. For sites where the hydrostratigraphy is not uniform, the calculations indicated above can be performed for each cell of the landfill rather than the entire landfill.

It should be noted that the normalized average mass flux $q_a$ determined from Fig. 9 or [13*a*] implicitly includes an assumed $H_f$. For example, a value of $q_{a0}$ determined from Fig. 9 is based on an initial estimate of $H_f = \infty$ (i.e., constant source concentration). This is then used in [15] to obtain an improved estimate value of $H_f = H_{f1}$. With this revised value of $H_f = H_{f1}$, one could use [13*a*] (and a computer program such as POLLUTE) to obtain a revised value of $q_{a1}$ and hence from [15] a revised value of $H_f = H_{f2}$. This procedure will converge rapidly and, indeed, is rarely necessary.

For most practical purposes involving conservative species, it is adequate to estimate $q_a$ from Fig. 9 or [13*a*], estimate $H_f$ from [15], and then use this value of $H_f$ in subsequent calculations without any additional refinement in the value of $H_f$. For highly reactive contaminants (i.e., with high values of $\rho K$) it may be necessary to use [13*a*], since Fig. 9 may underestimate the flux $q_a$ for these species.

*Example calculation of* $H_f$

Suppose that an examination of the composition of typical municipal waste indicates that chloride represents less than 0.2% of the total mass of the waste in the landfill. For a proposed landfill of area $A_0$ of 50 ha with a total mass of waste of 2 Mt, this corresponds to a total mass of chloride in the waste $M_{TC} = 0.002 \times 2 \times 10^6$ t = 4000 t. Supposing that the peak concentration of chloride $c_0$ = 2000 mg/L = 2000 g/m$^3$, then from [15*b*],

$$H_f = \frac{M_{TC}}{c_0 A_0}\frac{q_a}{q_0} = \frac{4000 \times 10^6}{2000 \times 50 \times 10^4} q_a/q_0 = 4 q_a/q_0 \text{ (m)}$$

The proportion of mass available for transport into the soil ($q_a/q_0$) may be determined from Fig. 9 as shown in the previous example.

Inspection of [15*b*] shows that for landfills resting on a barrier, the equivalent height of leachate is proportional to the total mass of contaminant ($M_{TC}$) divided by the plan area ($A_0$). This represents the mass per unit area and is directly related to the height of the waste mound. Thus, for a given height of waste, $H_f$ is independent of the size of the landfill. Thus, for example, if the mass of waste per unit area is 4 t, $H_f$ is the same for a landfill with an area of 50 ha and a total mass of 2Mt as it is for a landfill with an area of 20 ha and a total mass of 0.8 Mt.

The effect that the height of leachate can have on the calculated impact of a landfill will be demonstrated in a later section.

## Techniques for calculating contaminant migration through barriers

A contaminant transport model consists of the governing equations together with the boundary and initial conditions. Once the model has been formulated and the appropriate parameters have been determined (as discussed in the preceding sections), it remains to find a solution to the equations. The most frequently used solution techniques can be subdivided into five broad categories, namely, analytic, finite layer, boundary element, finite difference, and finite element techniques.

*Analytic solutions*

Analytic solutions can be obtained for simplified cases typically involving a single homogeneous layer (barrier) subject to simplified boundary conditions. Numerous solutions have been reported in the literature (e.g., Lapidus and Amundson 1952; Ogata and Banks 1961; Ogata 1970; Lindstrom *et al.* 1967; Selim and Mansell 1976; Rowe and Booker 1985*b*; Booker and Rowe 1987). Although these analytic solutions have sometimes been expressed in graphical form suitable for hand computations (e.g., Ogata 1970; Booker and Rowe 1987), the evaluation of the analytic solution generally requires the use of a computer. The primary uses of analytic solutions are (i) to perform quick sensitivity studies and preliminary design calculations and (ii) to check the results of more sophisticated analyses.

Probably the best known "analytic" solution is that for the concentration $c$ at a depth $z$ beneath the surface of a barrier assumed to be infinitely deep and subject to a constant surface concentration $c_0$ (see, e.g., Ogata 1970). This solution can be written in the form

[16*a*] $c = J[z,t]$

where

[16*b*] $$J[z,t] = \frac{c_0}{2}\left\{\exp\left(\frac{vz}{2D}\right)\exp(-ab) \times \operatorname{erfc}\left(\frac{a}{2\sqrt{t}} - b\sqrt{t}\right) + \exp(ab)\operatorname{erfc}\left(\frac{a}{\sqrt{t}} + b\sqrt{t}\right)\right\}$$

where $a = z\{(n + \rho K)/nD\}^{1/2}$ and $b = v\{n/[4D(n + \rho K)]\}^{1/2}$; $v = v_z$ = average linearized ground water velocity (seepage velocity) in the $z$ direction, $D = D_{zz}$ = effective diffusion coefficient in the $z$ direction, and $n$, $\rho$, and $K$ are as previously defined.

Clearly, the thickness of a barrier is never infinite. The effect of the presence of an aquifer at some distance $H$ below the top of the barrier can usually be bracketed by (i) the concentration given by [16] and (ii) the concentration obtained for a layer of finite thickness $H$ where the base of the layer is "washed" such that the concentration of contaminant is zero. The solution for this case (see, e.g., Rowe and Booker 1985*b*) is given by [17]:

$$[17] \quad c = -\left\{\sum_{p=0}^{\infty} \exp\left(-\frac{v}{D}[(p+1)H - z]\right) J(2(p+1)H - z,t) - \sum_{p=0}^{\infty} \exp\left(-\frac{v}{D}pH\right) J(2pH + z,t)\right\}$$

where $J[Z,t]$, given by [16*b*], is evaluated for $Z = 2(p + 1)H - z$ and $Z = 2pH + z$ for the first and second summations respectively; all other terms are as previously defined. Although this solution involves an infinite sum, in fact the series converges quickly (requiring only a few terms) and so the computation time is small even on a microcomputer.

The velocity $v$ in [16] and [17] is considered to be positive if it is directed away from the contaminant source.

In the previous section, it was shown that the finite mass of contaminant within a landfill could be represented in terms of an equivalent height of leachate $H_f$. An analytic solution for one-dimensional migration in an infinitely deep deposit where the source concentration varies with time (as mass is transported into the barrier) has been given by Booker and Rowe (1987) and can be written as

$$[18a] \quad c = c_0 \exp(ab - b^2t)[bf(b,t) - df(d,t)]/(b - d)$$

where

$$[18b] \quad f(b,t) = \exp(ab + b^2t) \operatorname{erfc}\left(b\sqrt{t} + \frac{a}{2\sqrt{t}}\right)$$

$$[18c] \quad f(d,t) = \exp(ad + d^2t) \operatorname{erfc}\left(d\sqrt{t} + \frac{a}{2\sqrt{t}}\right)$$

$$d = \frac{nD}{H_f}\sqrt{\frac{n + \rho K}{nD}} - b$$

and all other terms are as previously defined. The function $f(q,t)$ (for $q = b$ or $d$) may be evaluated directly by computer or using a hand calculator observing that

$$f(q,t) = \exp(-a^2/4t)\phi(x)$$

where $x = q\sqrt{t} + a/2\sqrt{t}$ and the function $\phi(x)$ is given in Fig. 10. Equation [18] reduces to [16] for $H_f \rightarrow \infty$ (i.e., $d = -b$).

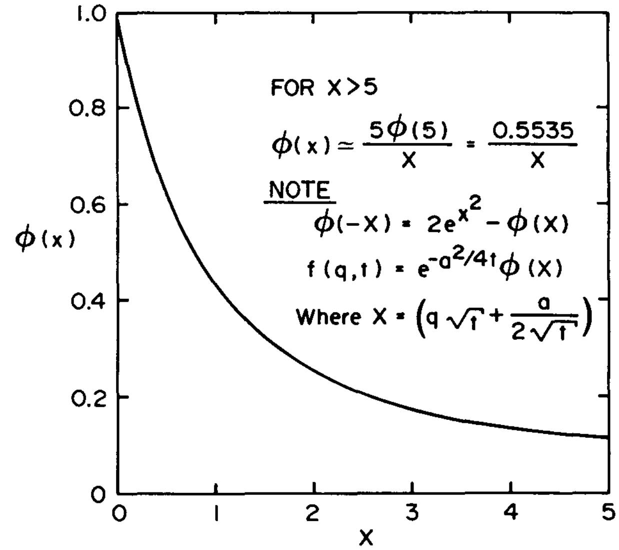


FIG. 10. Relationship for determining the variation in concentration with depth in a deep deposit for the case where the source concentration varies with time (see [18*a*]). Modified from Booker and Rowe (1987).

*Finite layer techniques*

The finite layer technique is applicable to situations where the hydrostratigraphy can be idealized as being horizontally layered (with the soil properties being the same at any horizontal location within the layer). For these conditions, the governing equations can be considerably simplified by introducing a Laplace and Fourier transform (the latter being required only for two- or three-dimensional problems). The transformed equations can then be readily solved. This procedure parallels that adopted in the development of many analytic solutions. The difference between finite layer solutions and analytic solutions arises from the fact that in the finite layer approach, the solution is inverted numerically rather than analytically. As a consequence, it is possible to examine more complicated and realistic situations. The formulation of the finite layer approach has been published by Rowe and Booker (1985*a*, *b*, 1986) for one-, two-, and three-dimensional conditions. The technique has been implemented in computer programs POLLUTE (Rowe *et al.* 1984) and MIGRATE (Rowe and Booker 1988*b*).

This approach is well suited to the use of microcomputers. Due to the semianalytic nature of the approach, contaminant concentrations and the total mass flux into the barrier can be very accurately determined at any specified times and locations of interest without determining the entire solution field. This can lead to significant computational savings (compared to finite element or finite difference methods) if the concentration is only required at a few key locations (e.g., monitoring points). However, it should be noted that the computational savings associated with this approach may be lost if it is necessary to calculate concentration at a large number of locations and times; this is usually not the case in the design of barriers.

Finite layer techniques have many of the advantages of analytic techniques. They are easy to use and the user need not be an expert in numerical analysis. They also require minimal input and only give results at the times and locations of interest. This technique is particularly well suited for performing sensitivity studies to identify the potential impact of uncertainty regarding the value of key design parameters. The technique is also well suited for performing checks on the results of numerical analyses using finite element or finite difference techniques.

Many of the advantages of the finite layer approach arise from the fact that the properties of the layers are assumed to be the same at any point in the horizontal plane. A consequence of this is that one cannot vary the vertical and horizontal velocities with lateral position. Nevertheless, for many practical situations reasonable engineering estimates of potential impact can be obtained by (i) using downward flow velocities that represent average values in each layer beneath the landfill and (ii) using the horizontal flows expected at the downgradient edge of the landfill. Sensitivity studies can then be performed to assess the implications of varying these velocities within the ranges expected for the field problem.

Finite layer techniques are not appropriate for modelling situations where there is a complex geometry or flow pattern (which cannot be reasonably modelled in terms of horizontal layers) or where modelling of nonlinearity is essential.

*Boundary element techniques*

The boundary element technique is suitable for solving the advection–dispersion equation (see, e.g., Brebbia and Skerget 1984). Its primary advantage over finite layer methods is the ability to model more complicated geometries. To date, boundary elements have not found wide application in contaminant migration studies.

*Finite difference and finite element techniques*

There has been extensive research into the use of finite difference and finite element techniques for the analysis of contaminant migration through soils. These numerical techniques are likely to be used for

(i) calculating the flow pattern within a hydrogeologic system, thereby defining the velocity field;

(ii) calculating the rate of migration of contaminants by solving the advection–dispersion equation (using velocities determined from (i)).

The techniques for modelling steady-state flow are well established (see, e.g., Frind 1987) and numerous commercial software packages are available. Many of the packages will run on microcomputers; however, there is a potential danger that inappropriate finite element mesh arrangements may be used simply to fit the problem onto a microcomputer. It is essential that the suitability of any finite element mesh be checked either against a known solution of by comparison with results obtained by substantially refining the mesh.

Finite element techniques provide the opportunity of modelling problems with complex geometries, complicated flow patterns, heterogeneity, and nonlinearity. There is wealth of literature dealing with different algorithms that have been proposed for solving the advection–dispersion equation. The sheer volume of literature is itself a warning that the use of these techniques is not as simple as might first appear. This is particularly so when dealing with problems where there are high advective velocities, low dispersivities, and (or) high contrast in dispersivity (see, e.g., Yeh 1984; Allen 1984). While good results can be obtained (e.g., Frind and Hokkanen 1987), particular care is required to ensure the selection of a suitable computational algorithm, finite element mesh, and time step.

*General comments*

All five classes of techniques discussed above have a role to play in the design of barriers. It is the author's opinion that, of these techniques, the two most useful are finite layer and finite element methods. Finite layer techniques require some idealization of the problem; however, they also provide a means of quickly and accurately assessing the implications regarding different design scenarios and different assumed parameters. The cost of performing these analyses is relatively small. The finite element method provides the most general technique for solving the governing equations and including complex geometry, velocity fields, and nonlinearity. However, use of the approach requires an experienced user as well as relatively large data preparation and computational cost.

## Assessing the impact of landfills on underlying aquifers

Proposed landfills are often sited above aquifers. This is particularly so in glacial deposits, which frequently consist of beds of clayey or silty till separated by relatively thin granular units that are used for water supply. In these situations it is necessary to evaluate the potential impact of the landfill on water quality in the aquifer(s) at the site boundary.

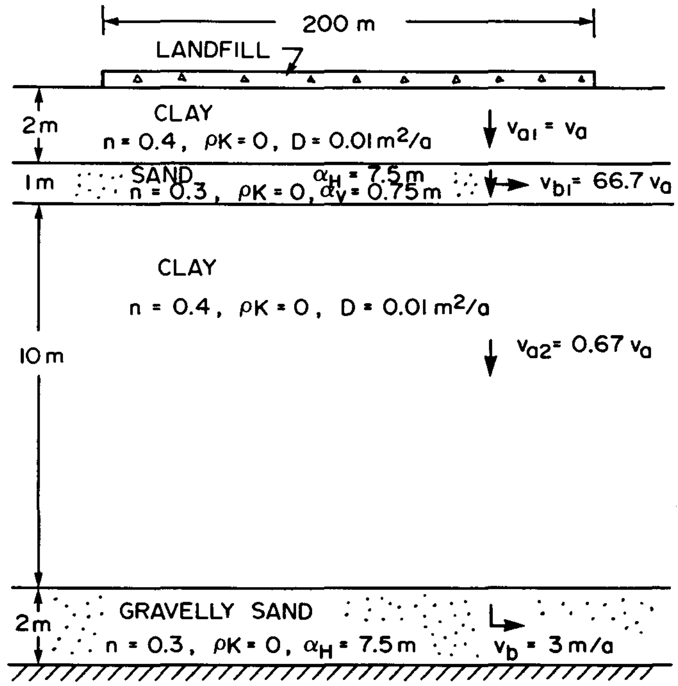


FIG. 11. Soil profile considered in example.

Because of factors such as the uncertainty associated with defining the groundwater system from limited data as well as the limitations regarding the adequacy of the data itself, it is not possible to make a prediction of the exact time at which contaminant would first migrate offsite no matter how sophisticated the theoretical model used. However, as indicated by Frind (1987), theoretical models can be particularly useful for examining the implications of different possible scenarios and different key parameters. The results from such a study can then be used in the formulation of an engineering opinion as to the potential for contamination of groundwater at the site boundaries by a proposed landfill.

*Effect of considering the finite mass of contaminant*

To illustrate the significance of parameters such as the equivalent height of leachate $H_f$ and the downward Darcy velocity $v_a$ through the barrier, consideration will be given here to the potential impact of a hypothetical landfill on groundwater quality at the boundary of the site (i.e., 100 m downgradient from the landfill). The assumed hydrostratigraphy is shown in Fig. 11.

The migration of contaminant was modelled using a two-dimensional finite layer solution to the two-dimensional advection–dispersion equation for a multilayered system described by Rowe and Booker (1986, 1987). The input to the model consists of the horizontal and vertical components of the Darcy (discharge) velocity, the distribution coefficient, and the coefficient of hydrodynamic dispersion for each layer, together with the density, porosity, and thickness of each layer. In addition, it is necessary to specify the initial concentration of contaminant in the landfill and the equivalent height of leachate (which represents the mass of contaminant available for transport into the soil). It is noted that this model directly considers

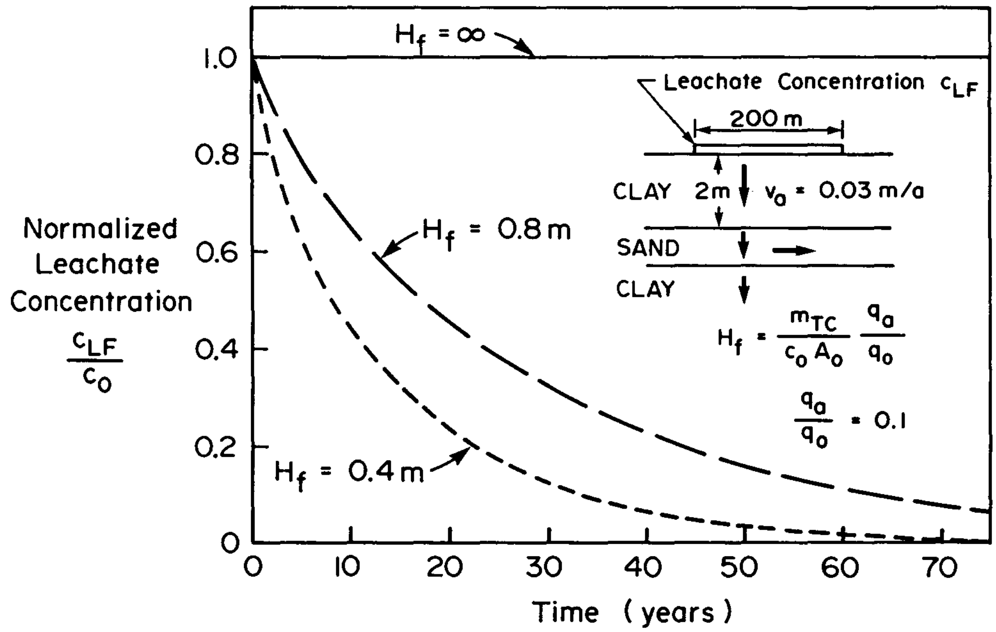


FIG. 12. Effect of equivalent height of leachate ($H_f$) on the variation in leachate concentration with time for the example problem.

the variation in concentration with time within the leachate as contaminant is removed from the landfill. The model also considers the mechanism of attenuation due to diffusion of contaminant from the granular units into the till (above and below) as it moves from beneath the landfill towards the boundary of the site (this will tend to reduce concentrations at the site boundary).

In an earlier section, it was shown that the mass of a given contaminant available for transport into the barrier can be expressed in terms of the equivalent height of leachate $H_f$. For the case shown in Fig. 12, the concentration of a conservative contaminant in the leachate is plotted against time for three different assumed values of $H_f$ and for a downward Darcy velocity, $v_a$, of 0.03 m/a. If the mass of contaminant is infinite ($H_f \rightarrow \infty$), then the concentration of contaminant within the landfill remains constant for all time. (Conversely, the assumption of a constant source concentration is equivalent to assuming an infinite mass of contaminants.) For the most realistic assumption of a finite mass of contaminant, the calculated concentration in the leachate decreases with time as contaminant is removed from the landfill. The rate of decline is related to the mass of contaminant (and hence $H_f$). For example, for $H_f$ = 0.8 m, the calculated concentration in the landfill has reduced to one-third the original value after 30 years, whereas for $H_f$ = 0.4 m, it takes about 15 years for the same reduction to occur. It is also apparent from this that the mass of contaminant can be back-calculated if the variation in concentration has been monitored in the leachate.

The effect of the equivalent height of leachate upon the calculated impact of the landfill or water quality at point "$x$" in the upper aquifer (100 m downgradient) of the landfill is shown in Fig. 13. If one assumes that the concentration of contaminant in the leachate remains constant (i.e., $H_f = \infty$), then the calculated concentration at point "$x$" increases until it reaches a steady-state value equal to 46% of the source concentration. When one considers a finite mass of contaminant (i.e., finite $H_f$) the concentration at point "$x$" increases to a peak value and then subsequently decreases. The magnitude of the peak and the time at which this occurs depends on the equivalent height of leachate $H_f$. As one might expect, the smaller the mass of contaminant available for transport, the smaller is the impact on a downgradient monitoring point. For the case of $v_a$ = 0.03 m/a and $H_f$ = 0.4 m, the peak concentration at point $x$ is approximately 11% of the original source concentration in the leachate compared with 46% of the source value obtained for $H_f = \infty$. Thus, in this case consideration of a realistic mass of contaminant reduces the calculated potential impact on groundwater quality by a factor of 4, illustrating that the assumption of a constant source concentration may be unrealistically conservative.

Inspection of Fig. 9 indicates that the Darcy velocity of 0.03 m/a is relatively large and contaminant transport through the barrier is being dominated by advection. This is a situation that might occur if there were a failure of the leachate collection system. In many landfill designs, the Darcy velocity into the barrier will be substantially smaller than 0.03 m/a, particularly while the leachate collection system is functioning. To examine the effect of this Darcy velocity, analyses were also performed for $v_a$ = 0.003 m/a. At this velocity, diffusion dominates contaminant transport and as indicated by Fig. 9 (and previously discussed), the tenfold reduction in velocity from $v_a$ = 0.03 m/a to 0.003 m/a only reduces the normalized average flux from about 0.03 m/a to 0.01 m/a. Assuming an infiltration of 0.3 m/a this corresponds to a threefold reduction in the proportion of mass available for transport into the barrier (i.e., from $q_a/q_0$ = 0.1 to $q_a/q_0$ = 0.033).

The calculated variation in concentration with time at the downgradient monitoring point $x$ is shown in Fig. 14 for an infinite mass of contaminant ($H_f = \infty q_a/q_0$) and a finite mass of contaminant ($H_f = 4q_a/q_0$ m) for assumed downward Darcy velocities of 0.03 and 0.003 m/a. If one assumes that the concentration in the source remains constant for all time (i.e., $H_f = \infty$), then a tenfold decrease in Darcy velocity $v_a$ only reduces the peak concentration by about 35%, from $0.46c_0$ to $0.3c_0$. However, when one considers the finite mass of contaminant (specifically $H_f = 4q_a/q_0$), then this tenfold decrease

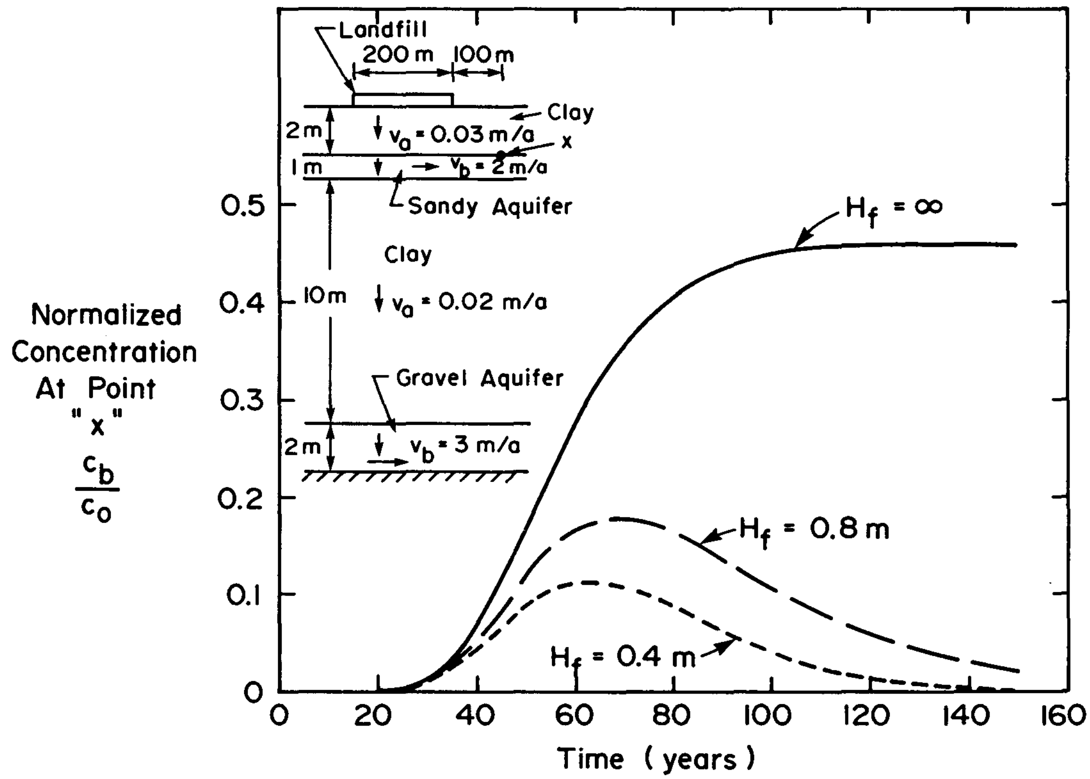


FIG. 13. Effect of the equivalent height of leachate on the variation in concentration with time at a point within an aquifer at the site boundary for the example problem.

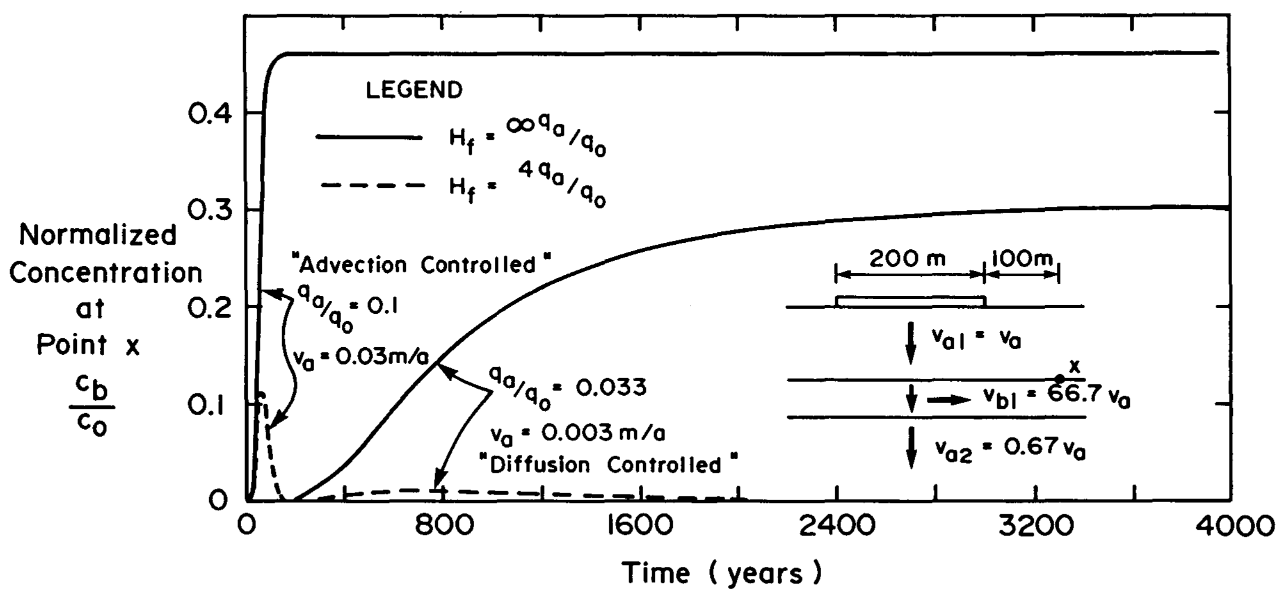


FIG. 14. Effect of equivalent height of leachate ($H_f$) on the variation in concentration and with time and contaminant impact at a point in the aquifer beneath the site boundary.

in Darcy velocity gives rise to a more than tenfold decrease in peak concentration, from $0.11c_0$ to $0.01c_0$. The corresponding increase in the time required to reach this peak was from a little over 60 years to about 700 years. For a Darcy velocity of 0.003 m/a as considered here, assumption of a constant source of concentration would result in an overestimate of the peak concentration by a factor of 30 if the mass of contaminant corresponds to $H_f = 4q_a/q_0$ m (e.g., 4000 t over a site of area 50 ha at an initial source concentration of 2000 mg/L).

It may be concluded that the mass of contaminants (expressed in terms of an equivalent height of leachate $H_f$) is a very important parameter to be considered when calculating attenuation of contaminants as they move into the groundwater system.

The effects of uncertainty regarding the advective velocity and dispersivity within granular units have been examined by

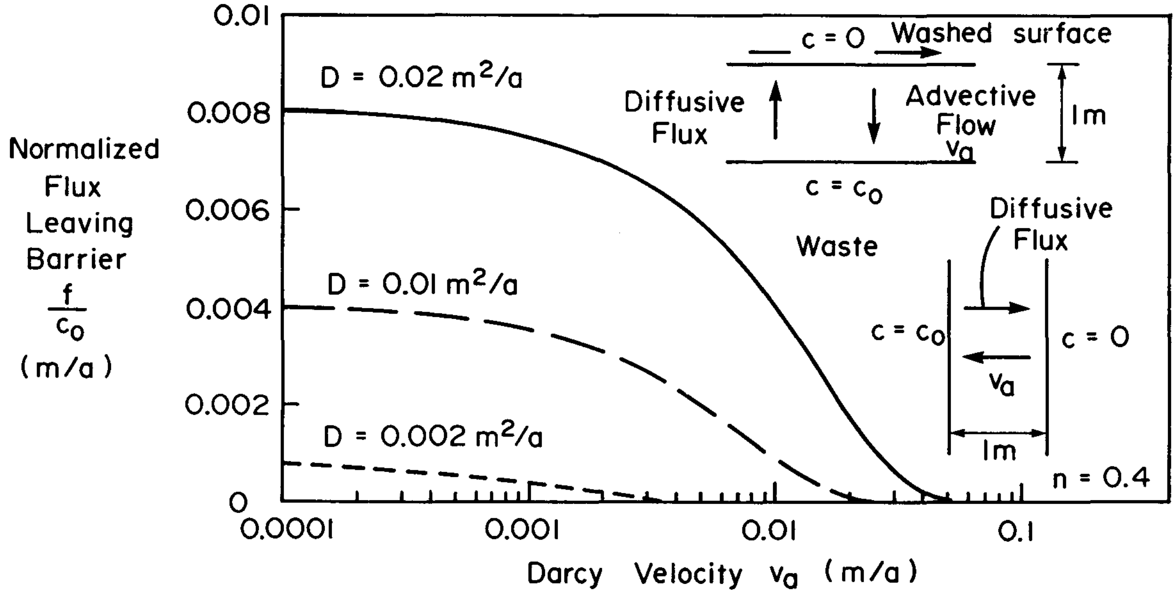


FIG. 15. Diffusive flux of contaminant into an aquifer against an inward flow at a Darcy velocity $v_a$ (steady-state conditions).

Rowe and Booker (1985*a*, 1986) and although important, will not be discussed here.

### Hydraulic trap

Contaminant migration from a landfill can be minimized if the landfill is located in a discharge zone. In this situation, flow (and advective transport) through the barrier and into the landfill will impede the outward movement of contaminants owing to the concentration gradient. If the inward flow is sufficient to prevent any significant contaminant migration out of the barrier, then this system may be referred to as a "hydraulic trap" (Trow Hydrology Consultants Ltd. 1986). However, the fact that there is inward flow does not necessarily mean that there will be no contaminant migration out of the barrier. The assessment of potential impact on groundwater quality involves two stages, viz.,

(i) determination of the advective velocities into the landfill;

(ii) determination of the diffusive movement out of the barrier.

The construction of a landfill will usually change the hydraulic characteristics of a given site. For example, a functioning leachate collection system may give rise to an average head in the leachate that is below the original groundwater level, thereby inducing inward flow at the sides of the landfill and, if the head is lowered sufficiently, at the base of the landfill. On the other hand, if significant leachate mounding were to occur due to failure of the collection system, then the head in the landfill will increase and may exceed the head outside the barrier, thereby reversing the flow direction and resulting in outward flow. In both cases, the mounding of the leachate must be calculated (as previously discussed) and the hydraulic gradient determined for the new flow system—this will require a seepage analysis.

Having determined the advective velocities, contaminant transport modelling may then be used to assess the potential movement of contaminants moving through the barrier. If the flow is outward, then there can be no hydraulic trap and the problem is similar to that discussed in the previous section. If the flow is inward, then analysis may be used to estimate the rate of movement and maximum extent of the contaminant movement.

If it is assumed that the concentration of contaminant within the landfill remains constant at $c_0$ and the base of the barrier is flushed by flowing water, such that the concentration is zero, then [17] may be used (taking the velocity to have a negative sign for inward flow) to determine the concentration at any depth $z$ and time $t$. For large times, the solution reduces to the steady-state case given by (Al-Niami and Rushton 1977)

$$[19] \qquad \frac{c}{c_0} = \frac{\exp\left[-v(H-z)/D\right]-1}{\exp\left[-vH/D\right]-1}$$

and hence the normalized flux into the aquifer ($z = H$) is given by

$$[20] \qquad \frac{f}{c_0} = \frac{-nv}{\exp\left[-vH/D\right]-1}$$

(where flow into the landfill corresponds to a negative velocity). Equations [17] and [19] may be useful for estimating the maximum extent of the contaminant plume. Note, however, that these equations *assume* zero concentration at the base of the barrier and hence the calculation of a small concentration near the base does not necessarily imply that the impact on an underlying aquifer would be small. For example, Fig. 15 shows the calculated steady-state contaminant flux passing into an aquifer beneath a 1 m thick clayey barrier for a range of inward Darcy velocities and assumed diffusion coefficients.

The flux $f$ has been divided by the initial source concentration and the resulting normalized flux $f/c_0$ has units of velocity. (This may, in fact, be thought of as the equivalent outward velocity of contaminant migration that occurs due to the outward diffusive transport, which is in opposition to inward flow.) The effective diffusion coefficient for many contaminants in clayey tills lies in the range $D = 0.01-0.02$ m²/a. For these conditions, Fig. 15 shows that the inward Darcy velocity would have to exceed 0.025 and 0.05 m/a for $D = 0.01$ and 0.02 m²/a respectively before the outward flux was reduced to negligible levels for a 1 m thick liner.

As a rule of thumb, the impact on an underlying aquifer is likely to be negligible if the concentration calculated from [19] is negligible for depths $z > 0.9H$. If the concentration is not negligible near the base (i.e., for $z > 0.9H$), then it is necessary to assess the potential impact of the proposed landfill upon groundwater quality using a more realistic model that explicitly considers the aquifer and the finite mass of contaminant (this can be easily done using finite layer techniques). If the calculated impact is unacceptable, then the barrier thickness must be increased and (or) the inward Darcy velocity must be increased.

## Migration through fractured rock

When dealing with fractured rock, the primary transport mechanism is advective–dispersive transport along the fractures. In these systems, the average linearized groundwater velocity $v_f$ is related to the Darcy velocity $v_a$ by the relationship

$$v_f = v_a/n_b$$

where $n_b$ is the effective porosity through which flow is occurring. Thus, the fracture velocity $v_f$ may be quite high because the fracture porosity of a rock mass $n_b$ is usually quite small, with a typical range between 0.1 and 0.001% based on hydraulic conductivity – fracture opening size – fracture frequency relationships (see, e.g., Hoek and Bray 1981). If conservative contaminants were to migrate along these fractures at the average linear groundwater velocity, $v_f$, then the rate of contaminant migration through the rock would be very fast. For example, the "Burlington landfill" is located in fractured Queenston shale. In the environs of the landfill, the Queenston shale has a typical fracture spacing between 0.05 and 0.35 m (Hewetson 1985). The Burlington landfill has been generating leachate for approximately 15 years. If contaminant moved at the speed of the average linear groundwater velocity, and if, for example, the velocity were 50 m/a, then 15 years after leachate enters the rock, contaminants should be readily detected at distances of up to 750 m downgradient of the landfill. However, the available field data (e.g., Gartner Lee and Associates Ltd. 1986) suggest that this is not the case.

While it may be true that migration of conservative contaminants could occur at a rate approaching that of the average linear groundwater velocity in porous media where the matrix of the media (e.g., the sand grains in a sandy aquifer) has a negligible effective porosity, it is not true that conservative contaminants (e.g., chloride) will migrate at the rate of the average linear groundwater velocity in fractured rock (or soil) systems where the matrix of the rock (or soil) between fractures has a significant effective porosity (see, e.g., Grisak and Pickens 1980). This is the case at the Burlington landfill where the Queenston shale has a matrix porosity of approximately 10%. In cases such as this, diffusion of contaminants from fractures or fissures into the adjacent matrix ("matrix diffusion") can control the migration of contaminants and represents an important attenuation mechanism.

The phenomenon of attenuation due to matrix diffusion is well recognized (e.g., Barker and Foster 1981; Foster 1975; Freeze and Cherry 1979; Grisak and Pickens 1980; Grisak *et al.* 1980; Sudicky and Frind 1982; Tang *et al.* 1981) and Gillham and Cherry (1982) have reviewed a number of cases where matrix diffusion has been shown to decrease concentrations of species moving along fractures or fissures. For example, Foster (1975) showed that the diffusion of tritium from flowing groundwater in the fractures of porous chalk (i.e., into the pore water of the porous rock matrix) could account for a rapid decrease in tritium concentration in the fractures. Similarly, Day (1977) used this "diffusion to the matrix" concept to account for a rapid decline in tritium concentration with depth in a fractured clay in the Winnipeg area. Finally, Grisak *et al.* (1980) have also demonstrated that the "diffusion to the matrix" model can give relatively good agreement between theoretical simulations and laboratory experiments in which a tracer solution was passed through a large column of fractured, clayey glacial till. From these studies, Gillham and Cherry concluded that

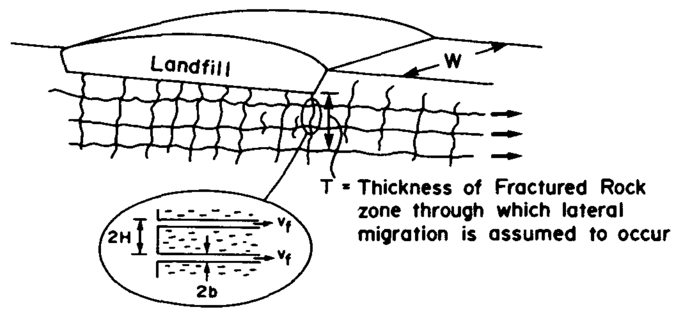


FIG. 16. Idealized parallel fracture system involving fracture with an opening size $2b$ at spacing $2H$. Groundwater velocity through the fractures is $v_f$.

> molecular diffusion can exert a major influence on the rates and patterns of migration of contaminants in fractured argillaceous deposits. Diffusive loss of contaminants from paths of active flow in the fractures to the matrix can be a dominant mechanism of attenuation.

Just as matrix diffusion can result in significant attenuation of conservative contaminant species, it can also result in significant attenuation of reactive contaminants. However, in the case of reactive contaminants, sorption processes will result in even greater attenuation. Some contaminants may be removed from free solution by cation exchange either at the surface of the fracture or within the matrix of the rock. Organics may also be removed by preferential partitioning of contaminants on organic matter. In principle, partitioning could occur both at the face of the fracture and within the rock matrix; however, relatively little is known about the effects of partitioning at the face and it would be prudent (and conservative) to consider only partitioning that occurs within the matrix of the rock.

Barker and Foster (1981) and Sudicky and Frind (1982) independently developed a solution for contaminant transport along a system of planar fractures (such as those shown in the insert of Fig. 16) assuming that contaminant transport can be regarded as one-dimensional along the fractures and one-dimensional into the matrix perpendicular to the fractures. Rowe and Booker (1988*a*) have extended this approach to consider the effects of the finite mass of contaminant available for transport along the fractures. As discussed in an earlier section, the mass of contaminant can be expressed as an "equivalent height of leachate," which is defined as the volume of leachate divided by the area, $A_0$, through which contaminant will pass.

To illustrate the determination of $H_f$, consider a landfill located on a fractured rock sytem. Suppose that the fractured rock extends to a depth of $T$ = 25 m below the base of the landfill (see Fig. 16). If one conservatively neglects the time that it would take for contaminant to move down into the fractured shale and is only concerned with lateral migration from beneath the landfill, then $H_f$ may be calculated as follows. Given that $M_{TC}$ = 4000 t, $c_0$ = 2000 mg/L, $q_a/q_0$ = 0.0066 (i.e., the recharge down into the shale is about 2 mm/a and

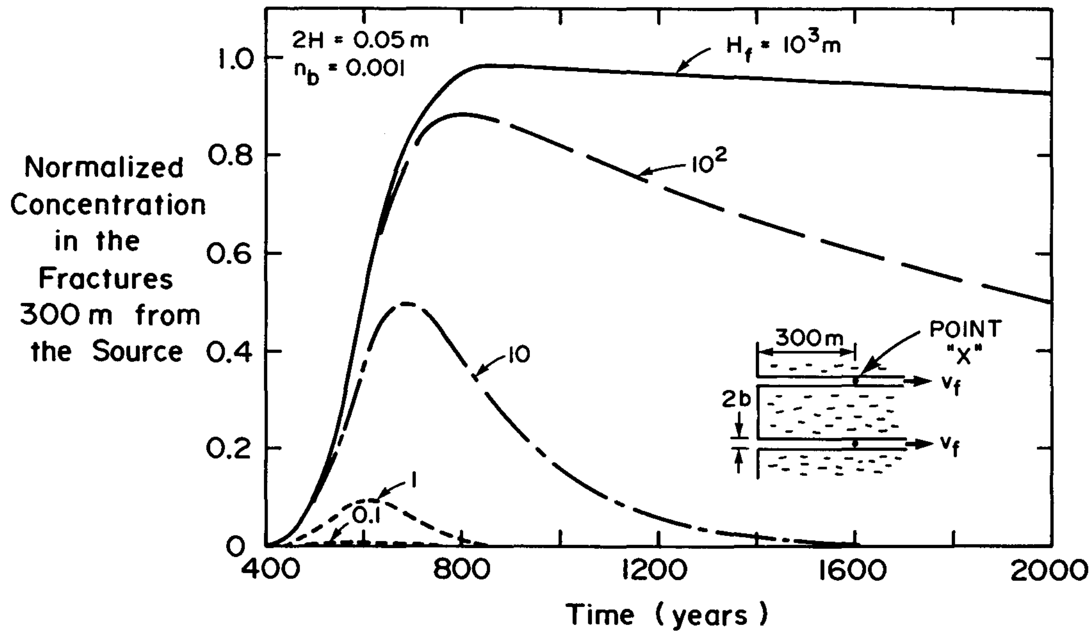


FIG. 17. Effect of the mass of contaminant (expressed in terms of an equivalent height of leachate $H_f$) on contaminant concentration 300 m from a hypothetical landfill in fractured rock.

TABLE 2. Parameters used in analysis of fractured rock

| Parameter | Symbol | Value |
|---|---|---|
| Darcy velocity (m/a) | $v_a$ | 0.05 |
| Fracture spacing (m) | $2H$ | $0.05 \leq 2H \leq 1.0$ |
| Fracture opening size ($\mu$m) | $2b$ | $5 \leq 2b \leq 1000$ |
| Fracture porosity | $n_b$ | $0.0001 \leq n_b \leq 0.01$ |
| Groundwater velocity (m/a) | $v_f$ | $5 \leq v_f \leq 500$ |
| Diffusion coefficient in fracture (m²/a) | $D_0$ | 0.04 |
| Diffusion coefficient in the rock matrix (m²/a) | $D_m$ | 0.0036 |
| Distribution or partitioning coefficient | $K$ | 0 |
| Dispersivity (m) | $\alpha$ | 3 |
| Matrix porosity | $n_m$ | 0.1 |

99.4% of leachate is collected), and the width of the landfill is $W$ = 500 m, then from [2],

$$H_f = \frac{M_{TC}}{c_0 \times W \times T} q_a/q_0$$

$$= \frac{4000 \times 10^6}{2000 \times 500 \times 25} 0.0066 = 1 \text{ m}$$

Clearly, if the recharge into the shale was 20 mm/a (i.e., $q_a/q_0 \times 0.066$ and 94% of the leachate is collected), then, all other factors being equal,

$$H_f = 10 \text{ m}$$

Using the analysis proposed by Rowe and Booker (1988*a*), it can be shown that the mass of contaminant available for transport into the shale (expressed in terms of $H_f$) can have a profound effect on the level of attenuation that can be achieved in a fractured system (it should be noted that the term $H_{LF}$ used by Rowe and Booker (1988) is related to $H_f$ as defined above by the fracture porosity $n_b$, viz., $H_{LF} = H_f/n_b$). For example, Fig. 17 shows the calculated variations in contaminant concentration with time at a point in the fractures 300 m downgradient of a landfill located in fractured shale. (The parameters used in this analysis are listed in Table 2.) Results are given for a range of values of $H_f$ between 0.1 and 1000 m, although the probable range of values is most likely to be from 0.1 to 10 m. In all cases, the concentration increases with time until a peak value is reached and then decreases for subsequent time. In general, the higher the value of $H_f$ the greater is the peak impact, and the time required to reach impact, whereas the rate of decline is much slower. For very high values of $H_f$, the mass loading on the rock is so great that very little attenuation due to matrix diffusion can occur. However, for more realistic values of $H_f$ of 10, 1, and 0.1, the peak impact is, respectively, less than 38, 8, and 1% of the initial concentration $c_0$ for this combination of parameters.

The groundwater velocity $v_f$ used in the calculations for Fig. 17 was assumed to be 50 m/a. Neglecting matrix diffusion, one would expect significant quantities of contaminant to reach 300 m downgradient within about 6 years. When matrix diffusion is considered (see Fig. 17), it is seen that the time required for the peak concentration of contaminant to reach the 300 m monitoring point is increased by about two orders of magnitude. Thus, even for the case where $H_f$ is so large that there is little attenuation of the peak concentration, these calculations indicate that diffusion can substantially increase the

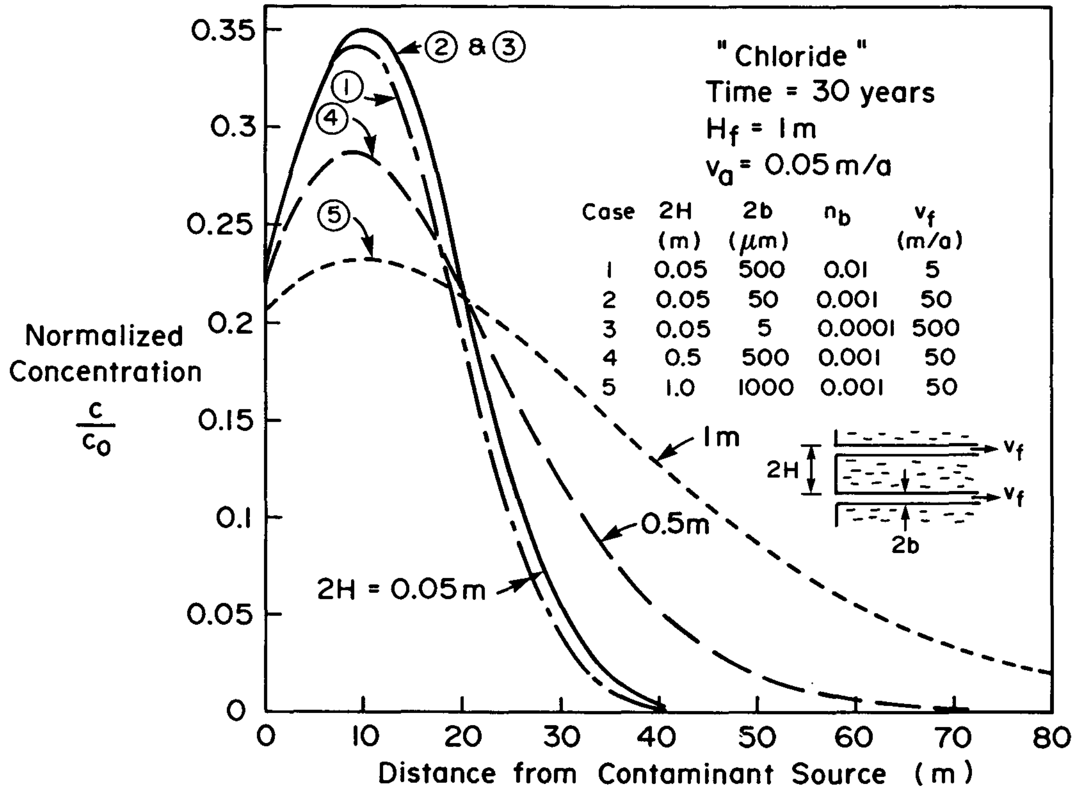


FIG. 18. Effect of fracture opening size (2*b*) and spacing (2*H*) on contaminant plume after 30 years' migration from a hypothetical landfill in fractured rock.

time it will take for this impact to occur. This factor needs to be considered in developing monitoring programs.

Fracture opening size and porosity are two of the most difficult parameters to determine for a fractured rock system. Fracture spacing is somewhat less elusive, although this, too, may vary substantially. This then raises the question as to what effect uncertainty regarding these parameters may have upon the calculated impact on groundwater quality.

Figure 18 shows the calculated extent of a contaminant plume after 30 years' migration from a hypothetical landfill. Results are given for five cases. In cases 1–3, the fracture spacing is held at 5 cm and the fracture opening size and porosity are varied by two orders of magnitude. As a consequence, the groundwater velocity $v_f$ and the equivalent height of leachate $H_f$ also vary by two orders of magnitude. Inspection of Fig. 18 shows that although there is a small effect on the contaminant plume, to all practical purposes the impact of the landfill is independent of the fracture porosity and opening size.[3] The reality of the situation is that the impact of the landfill (for a given fracture spacing) is controlled by the mass loading on the system and not by the actual speed at which groundwater moves along the fractures. In particular, notice that, based on consideration of "plug flow" (i.e., advection alone), one would expect the contaminant plume to have advanced between 150 and 15 000 m in 30 years for groundwater velocities of 5 and 500 m/a respectively. However, once matrix diffusion is considered it is seen that the plume would be expected to have advanced less than 40 m in that time period and the peak impact would be only 10 m from the landfill.

The range of porosities considered in cases 1–3 is very wide and may be considered to represent a very wide range in possible combinations of hydraulic conductivity, gradient, and fracture opening size. For example, the Darcy velocity of 0.05 m/a could correspond to a hydraulic gradient of 0.035 and a hydraulic conductivity of the rock of $4.5 \times 10^{-6}$ cm/s. For a given fracture spacing (of 0.05 m here), this hydraulic conductivity can be related to the fracture opening size. For example, Louis (1967) lists eight equations to describe flow under various conditions. The most commonly used relationship is for linear flow in a system of parallel smooth fractures (i.e., Hoek and Bray 1981). Adopting his relationship gives a fracture opening of about 15 μm for a value of hydraulic conductivity of $4.5 \times 10^{-6}$ cm/s. If the fractures are rough or are partially infilled, then opening size corresponding to this hydraulic conductivity will be larger than 15 μm. However, as indicated by cases 2 and 3, from the standpoint of contaminant migration, uncertainty as to fracture roughness and hence as to whether the opening size is 15, 30, or even 50 μm would not have a significant effect on the calculated contaminant plume.

Cases 2, 4, and 5 illustrate the effect of fracture spacing (all other factors being equal). This does have an influence on the contaminant plume, and as shown in Fig. 18, closely spaced fractures give rise to a smaller lateral extent of the plume in a given time, although the maximum impact is greater. The differences between the results for different spacings is related to the difference in time it takes for contaminant to diffuse

[3]This conclusion is valid for typical ranges of fracture opening size and spacing in a fractured media. The assumptions used in developing these results may not be applicable to situations where the fracture porosity $n_b$ is less than $10^{-5}$ or greater than $10^{-2}$. The results are only applicable to the situation where almost all of the advective contaminant transport is along the fractures (rather than through the matrix).

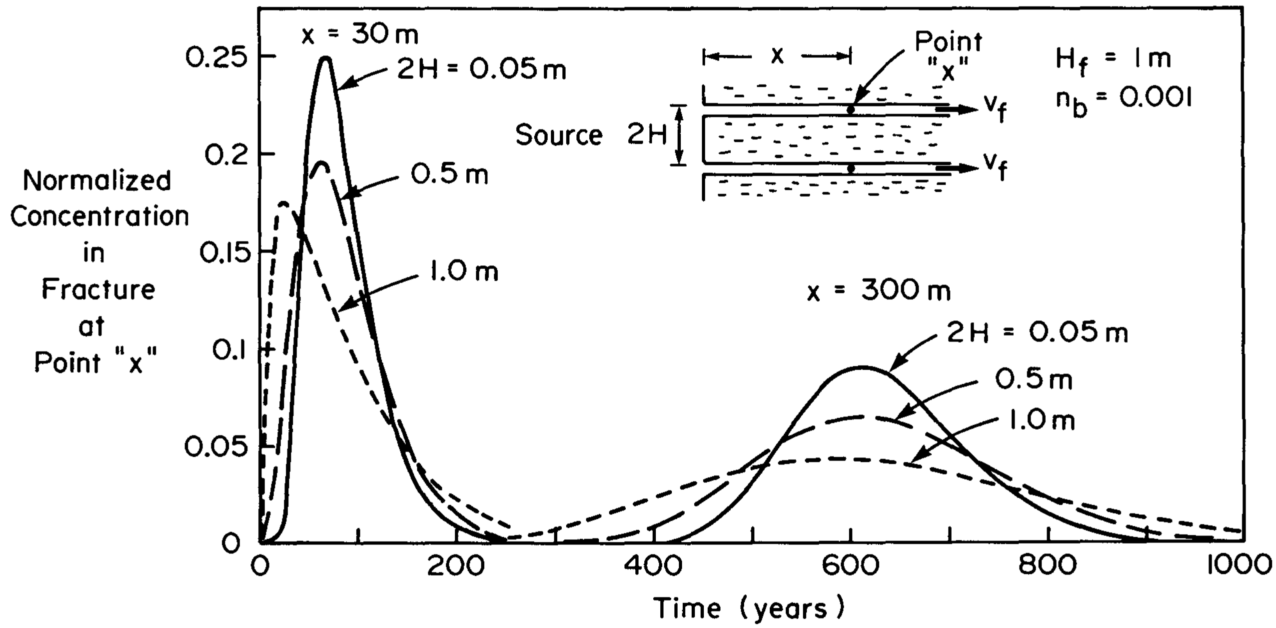


FIG. 19. Effect of fracture spacing on calculated impact 30 and 300 m from a hypothetical landfill located in fractured rock.

through the entire matrix of the rock. For the spacing of 0.05 m and diffusion coefficient considered, contaminant can readily diffuse throughout the rock matrix and approach equilibrium at a point in less than a year. However, this time is proportional to the square of the fracture spacing and may be of the order of tens to hundreds of years for wider spacings.

Figure 19 shows the variation in concentration with time at a point 30 and 300 m downgradient of the landfill for three assumed fracture spacings (i.e., 5, 50, 100 cm). It can be clearly seen that for a given Darcy velocity, smaller fracture spacing gives rise to a greater impact on groundwater quality, although the time required for the peak impact to occur may be significantly longer than at larger fracture spacings. The reason for this is that wide fracture spacings allow for slower uptake and release of contaminants into the matrix and a greater speed of the plume. Since the same mass of contaminant is being considered in each case, the more extensive plume implies that the mass is spread over a larger volume of rock (and hence the concentration, which is mass per unit volume of water, is reduced).

Uncertainty regarding the precise value of the fracture spacing can be dealt with by considering fracture spacings that bracket the probable range in the field. From the standpoint of maximum impact on groundwater quality, it is conservative to assume that the fracture spacing is at the smaller end of the expected range (all other factors being equal).

Figure 19 also shows the attenuation that will occur with distance from the contaminant source if one considers the finite mass of contaminant in the analysis. For example, at a fracture spacing of 5 cm the peak impact is reduced from about 25% of $c_0$ 30 m downgradient to about 9% of $c_0$ 300 m downgradient. The reduction for wider fracture spacing is even greater.

## Conclusion

This paper has discussed the role that analysis can play in the evaluation and design of barriers. Factors discussed include (i) the mechanisms controlling contaminant migration through barriers; (ii) the determination of diffusion and distribution coefficients; (iii) leachate mounding and the effect of clogging of leachate collection systems upon contaminant migration through barriers; (iv) the importance of considering the finite mass of contaminant available for transport into the soil and methods of modelling this; and (v) examples of how analysis may improve the geotechnical engineer's feel for the effectiveness of potential contaminant attenuation mechanisms in both glacial till deposits and fractured rock.

Based on review of the current state-of-the-art, it is suggested that

(i) theoretical models be used to estimate the diffusion and distribution coefficients for many contaminant species of interest using data from a column test on an "undisturbed" sample of the proposed barrier material;

(ii) consideration be given to the uncertainty associated with hydraulic conductivity values when selecting a design hydraulic conductivity to be used in assessing potential impact on groundwater that is being (or will be) used for domestic, industrial, or agricultural use;

(iii) the design of leachate collection systems and the calculation of potential impact on groundwater quality should involve some consideration of the implications of uncertainty regarding the magnitude of the hydraulic conductivity of the waste;

(iv) additional research is required into the potential for clogging of filters around leachate collection systems;

(v) the potential for clogging of drains and failure of the leachate collection system should be considered in the evaluation of contingency measures and the design of monitoring programs for waste disposal sites.

The role of analysis has been examined with respect to a number of examples and it has been shown that

(i) the mass of contaminant available for transport into the groundwater is a very important parameter to be considered when calculating attenuation of contaminants as they move into the groundwater system;

(ii) flow into a landfill through the barrier (a "hydraulic trap") may reduce contaminant transport to an underlying aquifer, and the impact on the aquifer will depend on the magnitude of the inward Darcy velocity, the diffusion coefficient, and the thickness of the barrier;

(iii) for fractured porous media, contaminant transport may be

substantially retarded owing to the process of matrix diffusion from the fractures into the adjacent porous matrix;
(iv) for the range of cases considered, the impact of contaminant transport depended on the mass of contaminant, the fracture spacing, and the Darcy velocity, but was almost independent of the fracture porosity and fracture opening size.

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