MODELLING OF 2D CONTAMINANT MIGRATION IN A LAYERED AND FRACTURED ZONE BENEATH LANDFILLS

by

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ABSTRACT

A new 2D finite layer formulation which allows consideration of both vertical and horizontal contaminant migration in systems which may consist of both fractured and unfractured layers is described. The practical application of the theory is illustrated with respect to a number of hypothetical cases. The results indicate that even relatively widely spaced small fractures can have a significant effect on potential impact. It is also shown that when dealing with relatively impermeable tills, significant impact on an underlying aquifer may not occur until after the landfill leachate is at a low strength; but the impact may be quite significant and may last for hundreds of years.

KEYWORDS: clay, fractures, contaminant impact; analysis; design; diffusion

INTRODUCTION

Rapid migration of contaminant through what was originally thought to be relatively tight (low hydraulic conductivity) clayey soils, both in the U.S.A. (e.g. at Wilsonville, IL U.S.A.) and Canada (e.g. at Smithville, Southern Ontario) has awakened concern regarding the potential fracturing of stiff-very stiff clays and clayey tills. The Wilsonville problem prompted research by Herzog et al. (Herzog and Morse, 1986; Herzog et al., 1989) which has provided evidence to suggest that the "unweathered" till (below the obviously weathered and fractured zone) at Wilsonville was also fractured to extensive depths. Similarly, recent investigations in Southern Ontario (Ruland, 1988; D'Astous et al., 1989; Rowe, pers. comm.; McKay, pers. comm.) have indicated that clayey till which appears unweathered and unfractured in conventional borehole investigations may be fractured to depths of as much as 10 m below ground surface (e.g. to depths of up to 6 m below the weathered crust). In some cases, this may mean that there is little or no unfractured clayey till between the base of the landfill and an underlying aquifer. Careful examination of deep test pits has demonstrated that at some locations fracturing at 1-2 m spacings may exist both above and below an upper "permeable" zone (which could potentially act as a conduit for contaminant transport if a landfill were constructed above this unit). Similarly, hydrogeologic investigations at other sites have identified that even though there may be 4-5 m of till separating the base of a proposed landfill from an underlying permeable zone, only 1-2 m of the material may be unfractured. When fractures are identified, the question then arises as to what effect they will have on contaminant transport from a proposed landfill facility.

The authors have recently prepared two articles (Rowe & Booker, 1989b, 1990b) which review past research on modelling of contaminant migration through fractured porous media and which describe the development and application of a simple semi-analytic, finite layer model. This model has been implemented in program POLLUTE v5 (Rowe & Booker, 1990c), which allows consideration of vertical migration through both fractured and unfractured layers into an underlying aquifer. This approach allows one to obtain a very quick estimate of potential impact on an underlying aquifer. The use of this type of model in design situations has been discussed by Rowe (1990). By incorporating an appropriate sink term, this model also allows one to approximately consider multiple aquifers; however, because of the intrinsic 1D nature of the model, it only provides an estimate of the concentrations in the aquifers and does not allow one to either examine the development of the plume beneath a landfill or to consider attenuation as contaminant migrates laterally from the landfill towards the site boundary.

The first objective of this present paper is to develop a more general 2D finite layer formulation which will allow consideration of both vertical and horizontal migration in systems which may consist of both fractured and unfractured layers. This formulation represents an extension of the authors' earlier 2D finite layer formulation (Rowe & Booker, 1985, 1987) to include consideration of fractured layers.

The proposed 2D finite layer approach does not have the full flexibility of finite element (e.g. Huyakorn et al., 1983) and LTG (e.g. Sudicky, 1990) formulations, however it does have the advantage of being applicable to many practical situations while being substantially easier to use than numerical methods which involve finite element descretization of the soil. The 2D finite layer approach allows relatively easy determination of concentrations of contaminant both beneath the landfill and at points away from the landfill. Clearly, however, any 2D formulation involves considerably more computation than a 1D formulation. In practical design situations the authors' 1D formulation is the most convenient means of performing preliminary analyses and sensitive studies. The full 2D formulation can then be effectively used to 'fine tune' the calculated impacts.

The second objective of the paper is to illustrate the practical application of the 2D finite layer formulation for a number of hypothetical cases which illustrate some of the factors that may warrant consideration in landfill design.

It should be emphasized that this paper is concerned with the migration of contaminants from municipal and non-hazardous waste disposal sites. The migration of contaminant from hazardous waste facilities and, in particular, the migration of concentrated dense non-aqueous phase contaminants involves additional transport mechanisms to those considered herein, and is beyond the scope of this paper.

THEORY

Consider a situation similar to that shown in Figure 1 where the soil deposit beneath a landfill can be idealized as a number of layers with node planes $z = z_0, z_1, \ldots, z_n$ and that each layer $k(z_{k-1} \leq z < z_k)$ may be either fractured or unfractured.

For unfractured layers, the fluxes f_x, f_z transported in the x and z directions within layers k are given by

[1a] $$f_x = nv_x c - nD_{xx} \frac{\partial c}{\partial x}$$

[1b] $$f_z = nv_z c - nD_{zz} \frac{\partial c}{\partial z}$$

and from consideration of conservation of mass, the governing differential equation for 2D contaminant transport is

[2] $$nD_{zz} \frac{\partial^2 c}{\partial z^2} - nv_z \frac{\partial c}{\partial z} + nD_{xx} \frac{\partial^2 c}{\partial x^2} - nv_x \frac{\partial c}{\partial x} - n \frac{\partial c}{\partial t} - \rho K \frac{\partial c}{\partial t} = 0$$

where c is the concentration at a point (x,z) in the layer at time t;

n is the effective porosity of the layer;

D_{xx}, D_{zz} are the coefficients of hydrodynamic dispersion in the x and z directions;

v_x, v_z are the groundwater velocities (and the quantities nv_x, nv_z are the Darcy velocities) in the x, z directions;

ρ is the dry density of the soil;

K is a partitioning or distribution coefficient for linear sorption.

If it is assumed that in the fractured layers, contaminant transport will be along the fractures but that contaminant can be lost from the fractures to the adjacent matrix material due to matrix diffusion (e.g. see Freeze & Cherry, 1979), then by considering mass transport through a unit area and conservation of mass within a unit volume, equations analogous to Eqs. [1] and [2] can be developed for the fractured layer. Thus for the fractured layers, the fluxes f_x, f_z are given by

$$[3a] \qquad f_x = v_{ax} c_f - D_{ax} \frac{\partial c_f}{\partial x}$$

$$[3b] \qquad f_z = v_{az} c_f - D_{az} \frac{\partial c_f}{\partial z}$$

and from conservation of mass the governing differential equation is

$$[4] \quad D_{az} \frac{\partial^2 c_f}{\partial z^2} - v_{az} \frac{\partial c_f}{\partial z} + D_{ax} \frac{\partial^2 c_f}{\partial x^2} - v_{ax} \frac{\partial c_f}{\partial x} - n_f \frac{\partial c_f}{\partial t} - \Lambda K_f \frac{\partial c}{\partial t} - \dot{q} = 0$$

where c_f is the concentration in a fracture at a point (x,z) at time t; v_{ax}, v_{az} are the Darcy velocities in the x,z directions; n_f is the fracture porosity; D_{ax}, D_{az} are the bulk coefficients of hydrodynamic dispersion; Λ is the surface area of fractures per unit volume; and K_f is the fracture distribution coefficient, defined by Freeze and Cherry (1979) which accounts for sorption onto the fissure walls; $\dot{q}$ is the rate per unit volume at which contaminant is being transported into the matrix material by matrix diffusion (to be given in Appendix A).

Assuming that layer k has regular fractures at spacing $2H_x$, $2H_y$, $2H_z$ and fracture opening sizes $2h_x$, $2h_y$, $2h_z$ in the x, y and z directions (see Fig. 1) then

$$[5a] \qquad n_f = \frac{h_x}{H_x} + \frac{h_y}{H_y} + \frac{h_z}{H_z}$$

$$[5b] \qquad \Lambda = \frac{1}{H_x} + \frac{1}{H_y} + \frac{1}{H_z}$$

$$[6a] \qquad D_{ax} = \frac{h_y}{H_y} D_{x1} + \frac{h_z}{H_z} D_{x2}$$

[6b] $$D_{az} = \frac{h_x}{H_x} D_{z1} + \frac{h_y}{H_y} D_{z2}$$

[6c] $$v_{ax} = \frac{h_y}{H_y} v_{x1} + \frac{h_z}{H_z} v_{x2}$$

[6d] $$v_{az} = \frac{h_x}{H_x} v_{z1} + \frac{h_y}{H_y} v_{z2}$$

where D_{x1}, D_{x2} are the coefficients of hydrodynamic dispersion in the x direction in the fracture sets parallel to the xy and xz planes respectively;

D_{z1}, D_{z2} are the coefficients of hydrodynamic dispersion in the z direction in the fracture sets parallel to the xz and yz planes respectively;

v_{x1}, v_{x2} are the groundwater velocities in the x direction along the fractures parallel to the xy and xz planes;

v_{z1}, v_{z2} are the groundwater velocities in the z direction along the fractures parallel to the xz and yz planes.

It is assumed here that the initial conditions in any layer k are given by:

[7a] $$c = 0 \quad \text{for all } (x,z) \text{ at } t = 0$$

[7b] $$c_f = 0 \quad \text{for all } (x,z) \text{ at } t = 0.$$

Suppose that the interface between the landfill and the underlying layer is at plane $z = z_o = 0$. In many practical situations it can be assumed that contaminant is placed over a relatively short period of time and the landfill is subsequently closed. This means that the concentration of contaminant in the landfill, c_{oL}, will reach a maximum concentration, c_o, and that this concentration will subsequently reduce due to transport of contaminant into the underlying soil and due to collection of leachate.

The mass of a given contaminant in the landfill can be represented in terms of a "reference height of leachate" H_r, as described by Rowe (1990), which is given by

[8] $$H_r = \frac{m_{TC}}{c_o A_o}$$

where H_r is the representative height of leachate; m_{TC} is the total mass of the contaminant species of interest; c_o is the maximum concentration in the landfill; A_o is the area through which leachate can migrate into the underlying layer.

Thus, H_r represents the mass of contaminant available for transport into the soil and/or for collection by the collection system. It does not correspond to the actual height of leachate in the landfill; rather, it corresponds to the height of leachate that would be required for all the contaminant to be in solution at a concentration c_o.

Assuming that the landfill is at field capacity, then for an infiltration into the landfill, q_o, the volume of leachate per unit area, q_c, collected (or escaping as surface seeps) is given by

[9] $$q_c = q_o - v_a$$

where v_a is the Darcy velocity into the soil beneath the landfill.

Consideration of conservation of mass within the landfill then allows us to establish a boundary condition at the node plane $z = z_o$, viz.

[10a] $$c_T = c_{oL} \quad |x| < \tfrac{1}{2} L$$

[10b] $$c_T = 0 \quad |x| > \tfrac{1}{2} L.$$

[11] $$c_{oL} = c_o - \frac{1}{H_r} \int_0^t \left(\frac{1}{L} \int_{-L/2}^{L/2} f_z \, dx\right) d\tau - \frac{q_c}{H_r} \int_0^t c_{oL} \, d\tau$$

where c_T, f_z are the concentration and flux evaluated at $z = z_o$, and L is the length of the landfill as shown in Figure 1.

It should be noted that the 'reference height of leachate' represents the total mass of contaminant in the landfill. Only a portion of the mass will be available for transport into the underlying soil; the remainder will either be collected by the collection system or will escape to the surface water. An alternative to the approach outlined above is to estimate the proportion of mass which is only available for transport at the soil and to represent this by the "equivalent height of leachate" H_f as discussed by Rowe (1988). If this approach is adopted then one does not explicitly consider the leachate collection and Eq. [11] is replaced by

[12] $$c_{oL} = c_o - \frac{1}{H_f} \int_0^t \left(\frac{1}{L} \int_{-L/2}^{L/2} f_z \, dx\right) d\tau$$

which is the boundary condition adopted in the authors' earlier formulation for unfractured soil (Rowe & Booker, 1985, 1987). Clearly, Eq. [12] is a special case of Eq. [11] where $H_r \to H_f$, $q_c \to 0$.

As discussed by Rowe and Booker (1987), the bottom aquifer in a system such as that shown in Figure 1 can be modelled either as a physical layer (just like any overlying layer) or as a boundary condition. If the lower aquifer is modelled as a physical layer, then the boundary condition at the base (i.e. below the bottom aquifer) is given by

[13] $$f_z = 0 \quad \text{at } z = z_{n+1} \text{ , for all } x,t$$

If the bottom aquifer is modelled as a boundary condition, then (as shown by Rowe & Booker, 1985) at node plane $z = z_n$, the boundary condition becomes

[14] $$c_b = \int_0^t \left[\frac{f_b}{hn_b} - \frac{v_b}{n_b}\frac{\partial c_b}{\partial x} + D_H \frac{\partial^2 c_b}{\partial x^2}\right] d\tau$$

where c_b, f_b are the concentration, c, and flux, f_z, evaluated at $z = z_n$; v_b is the horizontal Darcy velocity in the base; D_H is the horizontal coefficient of hydrodynamic dispersion; n_b is the porosity of the aquifer.

We now seek a solution to the governing equations [2] and [4] subject to the initial condition [7] and boundary conditions [10], [11] and [13] or [10], [11] and [14]. These equations can be simplified by the introduction of the Laplace Transform

[15] $$(\bar{c},\bar{f}_x,\bar{f}_z) = \int_0^\infty (c,f_x,f_z)e^{-st}\, dt$$

and the Fourier Transform

[16] $$(C,F_x,F_z) = \frac{1}{2\pi}\int_{-\infty}^\infty (c,f_x,f_z)e^{-i\xi x}\, dx$$

Thus Eq. [2] reduces to

[17] $$nD_{zz}\frac{\partial^2\bar{C}}{\partial z^2} - nv_z\frac{\partial\bar{C}}{\partial z} - \xi^2 nD_{xx}\bar{C} - i\xi nv_x\bar{C} - s(n+\rho K)\bar{C} = 0$$

If we further suppose that the Laplace Transform of the rate of mass transported from fractures into the adjacent matrix $\bar{q}$ is given by

[18] $$\bar{q} = s\bar{\eta}c_f$$

where $\bar{\eta}$ is determined in Appendix A, then Eq. [4] reduces to

[19] $$D_{az}\frac{\partial^2\bar{C}_f}{\partial z^2} - v_{az}\frac{\partial\bar{C}_f}{\partial z} - \xi^2 D_{ax}\bar{C}_f - i\xi v_{ax}\bar{C} - s(n + \Lambda K_f + \bar{\eta})\bar{C}_f = 0$$

Equations [17] and [19] both have solutions of the form

$$\bar{C} = Ae^{\alpha z} + Be^{\beta z}$$

where for an unfractured layer, $m = \alpha,\beta$ are the roots of the equation

[20] $$nD_{zz}m^2 - nv_z m - [\xi^2 nD_{xx} + i\xi nv_x + s(n+\rho K)] = 0$$

and for a fractured layer, $m = \alpha,\beta$ are the roots of

[21] $$D_{az}m^2 - v_{az}m - [\xi^2 D_{ax} + i\xi v_{ax} + s(n + \Lambda K_f + \bar{\eta})] = 0$$

It follows that for every layer k, the concentration at any point in the layer can be given exactly in terms of the concentration at the node planes z_j ($j = k-1$), z_k at the top and bottom of the layer. As shown by Rowe and Booker (1987), one can then develop a relationship between the fluxes $\bar{F}_{zj}$, $\bar{F}_{zk}$ at the top and bottom of the layer in terms of the concentrations $\bar{C}_j$, $\bar{C}_k$ at the top and bottom of the layer: viz.

$$[22] \qquad \begin{bmatrix} \bar{F}_{zj} \\ -\bar{F}_{zk} \end{bmatrix} = \begin{bmatrix} Q_k & R_k \\ S_k & T_k \end{bmatrix} \begin{bmatrix} \bar{C}_j \\ \bar{C}_k \end{bmatrix}$$

where Q_k, R_k, S_k, T_k depend on α, β and the layer thickness $H_k = z_k - z_j$ and are given in Appendix B. The form of Eq. [22] is identical for both fractured and unfractured layers; the effect of fracturing is incorporated in α, β which must be determined from Eq. [20] if the layer is unfractured and from Eq. [21] if the layer is fractured.

By invoking continuity of concentration and flux at the layer boundaries, the layer matrices for each layer k in the deposit may be assembled to give

$$[23] \qquad \begin{bmatrix} Q_1 & R_1 & & & & & \\ S_1 & T_1+Q_1 & R_2 & & & & \\ & S_2 & T_2+Q_3 & R_3 & & & \\ & & \vdots & \vdots & & & \\ & & & S_{n-1} & T_{n-1}+Q_n & R_n \\ & & & & S_n & T_n \end{bmatrix} \begin{bmatrix} \bar{C}_T \\ \bar{C}_1 \\ \bar{C}_2 \\ \vdots \\ \bar{C}_{n-1} \\ C_b \end{bmatrix} = \begin{bmatrix} \bar{F}_T \\ 0 \\ 0 \\ \vdots \\ 0 \\ -\bar{F}_b \end{bmatrix}$$

where $\bar{F}_T$ and $\bar{F}_b$ are the transformed fluxes at the top $(z = z_o)$ and bottom $(z = z_n)$ node plane respectively and $\bar{C}_T$ and $\bar{C}_b$ are the corresponding transforms of the concentrations at these points. It is noted that this matrix is generally not symmetric however since it is tridiagonal the solution of these equations is computationally very easy and requires very little computer memory.

Equation [23] must be solved subject to the appropriate boundary conditions. If the lower permeable layer is modeled as a physical layer, then the base boundary condition given by Eq. [13] is readily implemented by taking $\bar{F}_b = 0$ in Eq. [23].

If the bottom aquifer is modelled as a boundary condition, then transforming Eq. [14] and rearranging terms gives

[24a] $$\bar{F}_b = \omega\, \bar{C}_b$$

where

[24b] $$\omega = h[n_b s + i\xi v_b + n_b D_H \xi^2]$$

Substituting Eq. [24a] into the last line of Eq. [23] gives

[25] $$S_n \bar{C}_{n-1} + (T_n + \omega)\bar{C}_b = 0$$

Thus replacing the last line of Eq. [23] by Eq. [25] allows the horizontal advective-dispersive transport in the bottom aquifer to be modelled as a boundary condition.

The implementation of the upper boundary condition (Eqs. 10, 11) directly parallels that of Eq. [12] as described in detail by Rowe and Booker (1987) and, following some algebra, leads to the relationship

[26a] $$\bar{c}_{oL} = \frac{LH_r c_o}{sLH_r + \Lambda + q_c L}$$

where

$$\Lambda = \int_{-\infty}^{\infty} \frac{2\sin(\xi/2)}{s} \bar{\chi}\, dx \qquad [26b]$$

and $\bar{\chi} = \bar{F}_T$ is obtained by solving Eq. [23] (modified with the appropriate base boundary condition) for $\bar{C}_T = 1$. The reference concentrations $\bar{c}_{rj}$ are obtained for the reference condition with $\bar{C}_T = 1$ by invoking

$$\bar{c}_{rj} = \int_{-\infty}^{\infty} \bar{C}_j\, e^{i\xi x}\, d\xi \quad (1 \leq j \leq n)$$

and by performing the integration using numerical quadrature, where $\bar{C}_j$ are obtained by solving Eq. [23] (as modified for the base boundary condition) with $\bar{C}_T = 1$. At the same time, Λ may be determined from Eq. [26b]. The leachate concentration $\bar{c}_{oL}$ can then be determined from Eq. [26a] and hence the concentrations of the nodal planes are given by

$$\bar{c}_j = \bar{c}_{oL} \cdot \bar{c}_{rj} \qquad [27]$$

It is then a simple matter to numerically invert the Laplace Transform using a technique such as that proposed by Talbot (1979).

The theory presented in this section has been implemented in the computer program MIGRATE v8 which runs on IBM AT or 386 class microcomputers.

RESULTS

To illustrate the potential application of the theory presented in the previous section, consideration will be given to the situation where the base of a hypothetical landfill is to be located below the 'weathered' and highly fractured till in unweathered till which is fractured with an average fracture spacing of 1 m. The base of the landfill is assumed to be 4 m above a 1 m thick upper permeable unit (aquifer 1) and 7 m above a 2 m thick lower permeable unit (aquifer 2) as shown in Figure 2.

For the purposes of this particular example, it is assumed that the landfill is 400 m wide and is to be located in a recharge zone directly over a groundwater divide. For the sake of definiteness, it is assumed that the average (long term) height of leachate mounding in the landfill is 2.7 m above a reference datum 200 m downgradient of the proposed landfill and that there are boundary heads as shown in Figure 2. Other heights of mounding and boundary heads could have been considered equally as well. Clearly, greater levels of leachate mounding would increase the potential impact compared with the cases considered here, and lesser levels of leachate mounding would decrease the potential impact.

The symmetry associated with the assumed hydrogeologic conditions implies that the centre of the landfill will be a no-flow boundary and that only half (i.e. 200 m of the landfill) needs to be considered in flow and contaminant transport modelling. This also implies that any flow in the aquifer at the edge of the landfill arises from the landfill and that there is no dilution due to flow entering the aquifer upgradient of the landfill. [This is a result of the assumption that the landfill is on a groundwater divide; if this were not the case then one would consider the full 400 m wide landfill and there would be potential for dilution in the aquifer due to water flowing in the aquifer from upgradient of the landfill.]

Figure 2 shows the general hydrostratigraphy considered. There are potentially six different hydrostratigraphic units with vertical and horizontal hydraulic conductivities of k_v, k_h. Table 1 summarizes the values of k_v and k_h assumed for the seven cases examined. Where the till is assumed to be fractured it has $k_v = 10^{-7}$ cm/s, $k_h = 10^{-6}$ cm/s. Where it is not fractured, it is assumed to have $k_v = 10^{-8}$ cm/s, $k_h = 10^{-7}$ cm/s. The clay and clayey liner are assumed to be isotropic with $k_v = k_h = 2\text{x}10^{-8}$ cm/s. Table 2 summarizes the flow parameters as determined for a finite element flow analysis performed for each assumed hydrostratigraphy. The velocities v_{a1} and v_{a2} represent average downward velocities beneath the landfill. The velocities v_{b1}, v_{b2} are the horizontal velocities in the upper and lower aquifer at the edge of the landfill. Figure 3 schematically illustrates the seven cases considered. The details will be discussed in subsequent paragraphs. Table 3 summarizes the parameters used in the contaminant transport modelling using program MIGRATE v8.

The 'reference height of leachate' H_r was determined assuming an average thickness, H_w, and dry density, ρ_w, of waste of 10 m and 500 kg/m^3 respectively. Assuming that chloride is the contaminant species to be considered and that it represents 0.2% (p = 0.002) of the dry weight of waste and that the peak concentration, c_o, is 1000 mg/L (1 kg/m^3) then the 'reference height of leachate' H_r is given by

$$H_r = \frac{m_{TC}}{A_o c_o} = \frac{p\ H_w \rho_w}{c_o} = \frac{0.002\text{x}10\text{x}500}{1} = 10 \text{ m}$$

The average volume of leachate collected (or escaping as surface seeps) per unit per unit time is assumed to be 0.15 m^3/a/m^2. The fractured layers were considered to have orthogonal fractures at an average spacing

$2h_x = 2h_y - 8.5\ \mu m$.

As illustrated in Figure 3 (and Table 1), cases [1] and [2] both consider a 4 m fractured zone with $k_v = 10^{-7}$ cm/s between the landfill and the upper aquifer. Case [1] also assumes that the till between the upper and lower aquifers is fractured whereas case [2] assumes that it is not. Because of the much higher transmissivity of the lower aquifer (compared with the upper aquifer), this assumption has a significant effect on the Darcy velocity, v_{a1}, leaving the landfill (see Table 2) which reduces from 0.0085 m/a [case 1] to 0.004 m/a [case 2]. In contrast, the Darcy velocity in the upper aquifer, v_{b1}, is increased from 0.23 m/a [case 1] to 0.32 m/a [case 2]. The flow in the lower aquifer is only reduced by about a factor of three (i.e. from 0.74 m/a [case 1] to 0.24 m/a [case 2]) even though the hydraulic conductivity of the lower aquitard was reduced by one order of magnitude.

Figure 4 shows the decrease in leachate strength with time determined for case 1. Also shown is a simple hand calculation of the leachate concentration based on the volume of leachate collected, q_c, viz.

$$\frac{c_{oL}}{c_o} = \exp\left(\frac{-q_c t}{H_r}\right)$$

where c_{oL} is the concentration of the leachate at the time of interest, t; c_o is the maximum leachate concentration and H_r is the 'reference height of leachate'. Since the volume of leachate collected is large compared to the mass flux into the underlying soil the simple hand calculation gives a good estimate of the concentration in the landfill compared with the more rigorous calculation performed using MIGRATE v8. Similar results to those for case 1 were obtained for each of the seven cases considered.

Reference to Figure 4 shows that for the assumed conditions, the average concentration of chloride in the leachate decreases to approximately half strength after about 45 years. After about 145 years, the leachate strength has reduced to 100 mg/L (assuming c_o of 1000 mg/L). Assuming that the background concentration chloride in the groundwater is less than 50 mg/L, then according to policies such as Ontario's "Reasonable Use Policy" (MOE, 1986), an increase in chloride concentration of up to 100 mg/L in the groundwater would be environmentally acceptable and hence one might be tempted to infer from this that there would be no need to monitor the landfill or groundwater after 145 years. That this is <u>not</u> the case is evident from Figure 5 which shows the calculated variation in concentration with time at two "monitoring points". Assuming that for thin 'aquifers' the monitoring wells would be screened across the entire thickness of the permeable unit, the concentrations c_{b1}, c_{b2} respectively represent the average calculated concentration in the upper and lower aquifer (i.e. the average of the calculated concentration at the top and bottom of each aquifer) at the edge of the landfill.

Inspection of Figure 5 shows that if the mounding of leachate to elevation 102.7 m occurred quickly (i.e. if this were the design level of mounding) then there would be no impact on either aquifer for at least 100 years however eventually there would be a significant impact. For case 1 (assuming both the upper and lower aquitards are fractured with k_v = 10^{-7} cm/s), the increase in chloride concentration in the upper aquifer is about 275 mg/L after 140 years even though the concentration in the leachate is only 100 mg/L. The chloride concentration continues to increase until a peak impact of about 460 mg/L is reached after approximately 200 years (at which time the concentration of chloride in the leachate is less than 50 mg/L). This clearly shows that the clayey till can act as a buffer and can provide attenuation of leachate strength since the peak concentration in the upper aquifer is only 46% of the peak value originally in the leachate. However, it is equally clear that the delay in the impact should also be considered when monitoring; once contaminant enters the till it may take a long time before it impacts on an underlying aquifer but that does not mean that there will not be an impact. It is not appropriate to terminate monitoring simply because the leachate strength has reduced to a level where it does not impose an environmental risk.

The analyses for case 1 assume relatively minor fracturing of both till layers (fractures with an opening size of 8.5 μm at 1 m spacing; $k_v \sim 10^{-7}$ cm/s); however, even this minor fracturing can have a significant long term effect as shown in Figure 5. As discussed above, the peak impact in the upper aquifer of about 460 mg/L occurs after approximately 250 years. At this time there is negligible impact on the lower aquifer, but after another 200 years the lower aquifer is also significantly impacted (i.e. with a peak increase in chloride of 375 mg/L at about 400 years).

The magnitude of the impact could be reduced by reducing the height of mounding and if the leachate collection system were to maintain a leachate level at an elevation head of less than 100 m, so that there is inward flow into the landfill, the impact could be minimized even though the till is fractured. However for case 1, the results shown in Figures 4 and 5 imply that to ensure that the impact on the aquifer does not exceed 100 mg/L, the leachate collection system would have to function for 90 years (i.e. until the chloride concentration in the leachate reduced to c_{oL} = 100/0.46 = 217 mg/L). This issue of longevity of leachate collection systems is discussed in detail by Rowe (1990) and will not be pursued here.

The only difference between case 1 and case 2 is the absence of fracturing of the lower aquitard for case 2. As discussed, this reduces the flow through both aquitards and leads to an increase to the time of impact and a reduction in the magnitude of impact compared with case 1. Nevertheless, even though the average Darcy flow through the upper aquitard is only 0.004 $m^3/a/m^2$ the peak impact of about 200 mg/L at approximately 500 years is twice that permitted in the Province of Ontario based on the 'Reasonable Use Policy' (assuming a background concentration of 50 mg/L). It is also noted that there is significant impact on the lower aquifer even though the lower aquitard is not fractured and has a hydraulic conductivity of 10^{-8} cm/s.

Given that the impact for cases 1 and 2 would be unacceptable based on the requirement to protect the 'reasonable use' of waters in these aquifers at the site boundary, the question then arises as to what could be done to minimize impact. As previously noted, one could reduce the level of leachate mounding. If this option were adopted the reduced (negligible) mounding would have to be maintained for 90 years and 45 years respectively. Another option would be to remove the upper 1 m of till below the proposed landfill base and rework it as a compacted clay liner. For the purposes of this example, it is assumed that the liner has a hydraulic conductivity of $2\text{x}10^{-8}$ cm/s. Cases 3 and 4 consider a 1 m thick liner and either a fractured or unfractured lower aquitard respectively. As might be expected, installation of the liner decreases the Darcy flow from the landfill and, consequently, the impact on the aquifer (see Figure 6). However, it is equally evident that this liner is not, of itself, enough since the peak impacts shown in Figure 6 are still quite significant even though the time required for their impact to occur is large.

Cases 5 and 6 (see Figs. 3 and 7) consider the situation where the upper aquitard consists of 3 m of fractured till ($k_v = 1 \times 10^{-7}$) and 1 m of unfractured lacustrian clay ($k_v = 2 \times 10^{-8}$ cm/s). It is further assumed that the top 1 m of the fractured till beneath the landfill is removed and re-compacted as a liner with $k_v \sim 2 \times 10^{-8}$ cm/s. In case 5 the lower aquitard is assumed to be fractured ($k_v \sim 1 \times 10^{-7}$ cm/s); in case 6 it is not fractured ($k_v = 1 \times 10^{-8}$ cm/s). The presence of the unfractured clay layers in the upper aquitard serves to reduce flow and calculated impact as shown in Fig. 7. For these cases, the impact on the lower aquifer is probably acceptable based on "Reasonable Use Criteria" (MOE, 1986); however, the impact on the upper aquifer is still unacceptable based on these calculations.

At this point it is worth emphasizing the assumption made in modelling fractured flow; that contaminant is transported along the fractures rather than through the matrix of the fractured media. This may be a reasonable assumption when the fractures control the bulk hydraulic conductivity of the till and where there is an easy path for contaminant to pass into the fractures (e.g. cases 1 and 2). However, this assumption may not be valid when the fractured till is sandwiched between two unfractured layers of clay (e.g. in the upper aquitard for cases 5 and 6). For these cases, the modelling of migration primarily through the fractures assumes that once leachate passes through the liner it spreads out and passes through the underlying fractures; this is equivalent to assuming that there is a thin sand layer between the liner and the fractured till which serves to distribute the 'leachate' to the fractures. If this does not exist, then the contaminant will not have an easy path to the fractures and much of it is likely to pass through the matrix rather than the fractures. It is not a simple matter to model this situation, however it is relatively simple to model the two limiting cases viz. migration along the fractures only (as shown in Fig. 7) and migration in the matrix only (i.e. neglecting the fractures in the contaminant transport modelling). Figure 8 shows the effect on the two different assumptions on calculated impact for case 6. The results obtained for the 'fractured' assumption are the same as those shown in Fig. 7. The results obtained for the 'unfractured' assumption show a faster arrival time but a lower impact on the upper aquifer than those obtained for the 'fractured' assumption. This represents an awkward situation in that one assumption implies the impact may be just acceptable while the other implies it is not acceptable (assuming that an increase in chloride of 100 mg/L is acceptable based on 'reasonable use' guidelines and a background concentration of 50 mg/L). In a case such as this, judgement would be required to assess what is reasonable. In this case, the 'truth' probably lies between the two results and hence would be marginally unacceptable unless there was a downgradient attenuation zone.

The cases considered to this point have considered variation in the properties of the aquitards and have assumed constant properties of the aquifer. Clearly, the hydraulic conductivity of the aquifers is also important. To illustrate this, case 7 was considered. Here the aquitards have the same properties as considered for case 6 but the hydraulic conductivity of the upper aquifer is increased (from 10^{-4} cm/s to 10^{-3} cm/s) and the hydraulic conductivity of the lower aquifer is decreased (from 10^{-3} to 10^{-5}). Thus in case 6 the lower aquifer is a significant drain for the system (taking almost 50% of the flow from the landfill) whereas in case 7 the upper aquifer is the major drain (taking almost all the flow). As might be anticipated, increasing the hydraulic conductivity of the upper aquifer increases the flow from the landfill (all other factors being equal). Results were obtained assuming migration through fractures (for the fractured till) and migration through the matrix in a similar fashion to that already discussed for case 6 and the results are shown in Figure 9.

Comparison of the results obtained for cases 6 and 7 shows that the higher Darcy velocity in the upper aquitard associated with case 7 gives rise to earlier impacts compared to case 6. For the case of migration through the intact soil, the peak impact in the upper aquifer is also greater, however this is not the case when one considers migration through the fractures. This situation arises because of the buffering effect that results from matrix diffusion from the fractures into the intact material (and subsequently from the matrix material to the fractures). Thus (as has previously been discussed by the authors Rowe & Booker, 1989, 1990), there is a complex symbiotic relationship between the effects of Darcy velocity and matrix diffusion and a higher Darcy velocity does not necessarily result in higher impact at a monitoring point when dealing with fractured media. When modelling contaminant migration in fractured media, it is important to examine a range of realistic possible situations in order to determine which combination of parameters provides the peak impact. This combination is not always obvious.

Assuming that regulatory requirements specify that one cannot increase chloride in the aquifer by more than about 100 mg/L, it is evident that for case 7 this requirement would be met in the lower aquifer but not in the upper aquifer at the edge of the landfill. One means of allowing additional reduction in impact is to acquire a natural attenuation zone. Figure 10 shows the variation in chloride concentration (with time) in the upper aquifer at "monitoring points" beneath the edge of the landfill, 30 m downgradient and 100 m downgradient of the landfill.

Inspection of Fig. 10 shows that the impact of the landfill on the upper aquifer decreases with distance away from the landfill and that one could meet the 100 mg/L maximum increase in impact postulated here at a little over 100 m from the landfill.

It is worth emphasizing again that when landfills are constructed on relatively 'tight' (i.e. hydraulic conductivity less than 10^{-7} cm/s) tills or clays, the impact of that landfill upon underlying aquifers may not occur until a time long after the leachate has reached benign levels (e.g. see Fig. 4). The clay acts as a buffer, slowing movement into the underlying soil but the impact, although long in coming, can be significant and can last for significant periods of time. For example, in case 7, if the site boundaries were 30 m from the edge of the landfill, the increased impact would exceed 100 mg/L (the MOE limit (MOE, 1986) if the background concentration is 50 mg/L) for a period of about 225 years.

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APPENDIX A

Rowe and Booker (1990a) have established that in the Laplace domain the source term q appearing in Eq. 4 can be expressed in the form

[A1] $$\bar{q} = s\,\bar{\eta}\,\bar{c}_f$$

in which η is a known function. They also give a detailed derivation of the specific expression for a general blocky system. For the sake of brevity, the derivation is omitted here and only the final results are presented.

Case 1:

Suppose that there is a single set of fractures parallel to the z axis at a spacing $2H_x$, then

[A2] $$\bar{\eta} = n_m R_m \frac{\tanh \mu H_x}{\mu H_x}$$

where

$$\mu^2 = \frac{R_m s}{D_m}$$

$$R_m = 1 + \frac{\rho K_m}{D_m}$$

ρ = dry density of the matrix soil; K_m is the distribution coefficient of the matrix soil; n_m is the matrix porosity; and D_m is the diffusion coefficient in the matrix.

Case 2:

Suppose there are two orthogonal sets of fractures parallel to the z axis at spacings $2H_x$, $2H_y$ (see Figure 1), then

$$[A3] \quad \bar{\eta} = n_m R_m \left[1-4 \sum_{j,k} \frac{s}{[s + \frac{D_m}{R_m} (\alpha_j^2 + \beta_j^2)]} \cdot \frac{1}{(\alpha_j H_x)^2} \cdot \frac{1}{(\beta_k H_y)^2} \right]$$

where

$$\alpha_j = (j - 0.5) \frac{\pi}{H_x}$$

$$\beta_k = (k - 0.5) \frac{\pi}{H_y}$$

and all other terms are as previously defined.

It is possible to find a reasonably accurate approximation to Eq. A3, for the case in which the spacing between the sets of orthogonal fissures is equal viz. $H_x = H_y$. In this case it seems reasonable to approximate diffusion into the square prism between adjacent fissures by diffusion into a cylinder of radius 'a' having an equal cross-sectional area, that is one for which

$$[A4] \qquad a = \frac{2H_x}{\sqrt{\pi}}$$

giving

$$\bar{\eta} = 2n_m R_m \frac{I_1(\mu a)}{\mu a}$$

where I_1 is a modified Bessel function of order one.

Case 3:

Consider three orthogonal sets of fractures, one parallel to the y-z plane, one parallel to the x-z plane and one parallel to the x-y plane at spacings $2H_x$, $2H_y$ and $2H_z$ then

$$[A5] \quad \bar{\eta} = n_m R_m \left[1 - 8 \sum_{j,k,\ell} \frac{s}{[s + \frac{D_m}{R_m} (\alpha_j^2 + \beta_k^2 + \gamma_\ell^2)]} \cdot \frac{1}{(\alpha_j H_x)^2} \cdot \frac{1}{(\beta_k H_y)^2} \cdot \frac{1}{(\gamma_\ell H_z)^2} \right]$$

where $\alpha_j = (j - 0.5) \frac{\pi}{H_x}$

$\beta_k = (k - 0.5) \frac{\pi}{H_y}$

$\gamma_\ell = (\ell - 0.5) \frac{\pi}{H_z}$

and all other terms are as previously defined.

APPENDIX B

The relationship between transformed nodal fluxes and concentrations given by Eq. [22] is fully specified by the following:

$$\begin{bmatrix} \bar{F}_{zj} \\ \\ -\bar{F}_{zk} \end{bmatrix} = \begin{bmatrix} Q_k & R_k \\ \\ S_k & T_k \end{bmatrix} \begin{bmatrix} \bar{C}_j \\ \\ \bar{C}_k \end{bmatrix}$$

in which

$$Q_k = \frac{\beta e^{\alpha H_k} - \alpha e^{\beta H_k}}{e^{\beta H_k} - e^{\alpha H_k}}$$

$$R_k = \frac{(\alpha - \beta)}{e^{-\alpha H_k} - e^{-\beta H_k}}$$

$$S_k = \frac{(\beta - \alpha)}{e^{\alpha H_k} - e^{\beta H_k}}$$

$$T_k = \frac{\beta e^{-\alpha H_k} - \alpha e^{-\beta H_k}}{e^{-\alpha H_k} - e^{-\beta H_k}}$$

where $H_k = z_k - z_j$ is the thickness of layer k

α, β are given by the solution to Eq. [20].

TABLE 3 COMMON PARAMETERS USED IN TRANSPORT MODEL

Quantity	
Reference Height of Leachate H_r (m)	10
Volume of Leachate Collected/Area q_c (m/a)	0.15
Initial Concentration c_0 (mg/L)	1000
Porosity of Till/Clay Matrix (-)	0.4
Diffusion Coefficient in Matrix (m^2/a)	0.02
Distribution Coefficient (m^3/kg)	0
Fracture Spacing $2H_x = 2H_y$ (m)	1
Fracture Opening Size $2h_x = 2h_y$ (μm)	8.5
Coefficient of Hydrodynamic Dispersion Along Fracture (m^2/a)	0.06
Fracture Distribution Coefficient (m)	0
Porosity of Aquifers (-)	0.3
Longitudinal Dispersivity (m)	1
Transverse Dispersivity (m)	0.1
Darcy Velocity(s) (m/a)	see Table 2

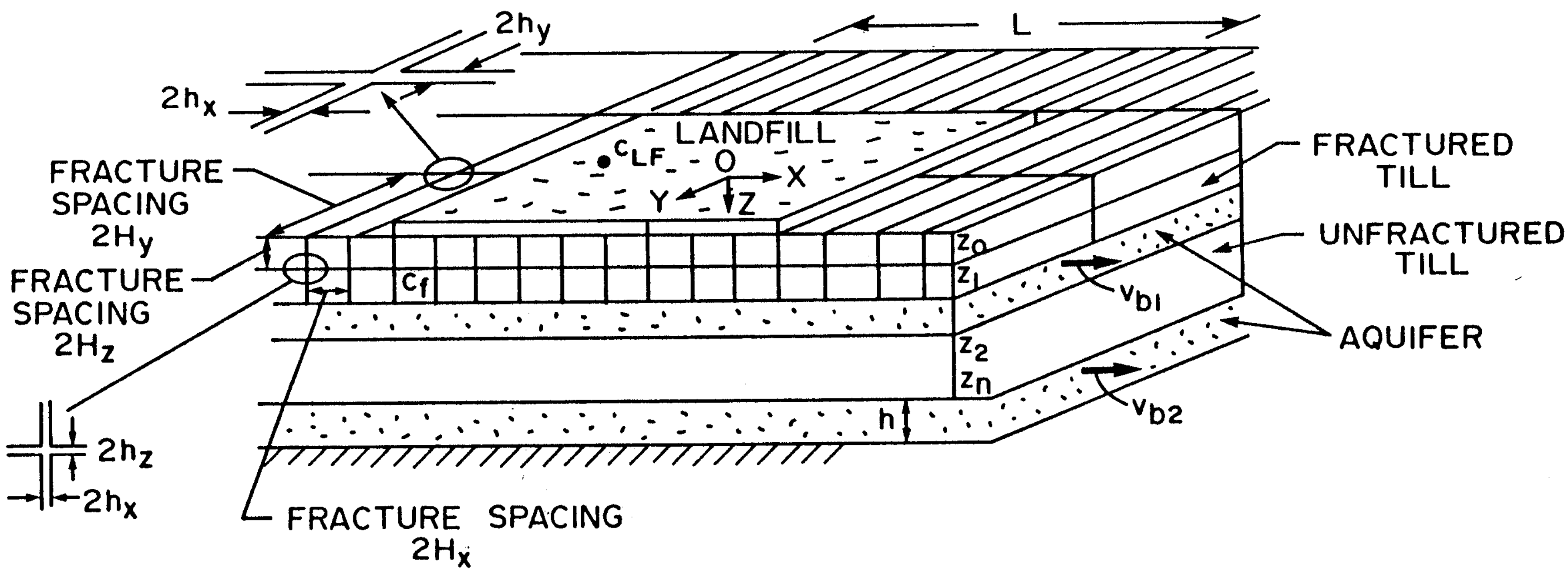

FIGURE 1 PROBLEM CONFIGURATION - FRACTURED TILL OVERLYING AN AQUIFER

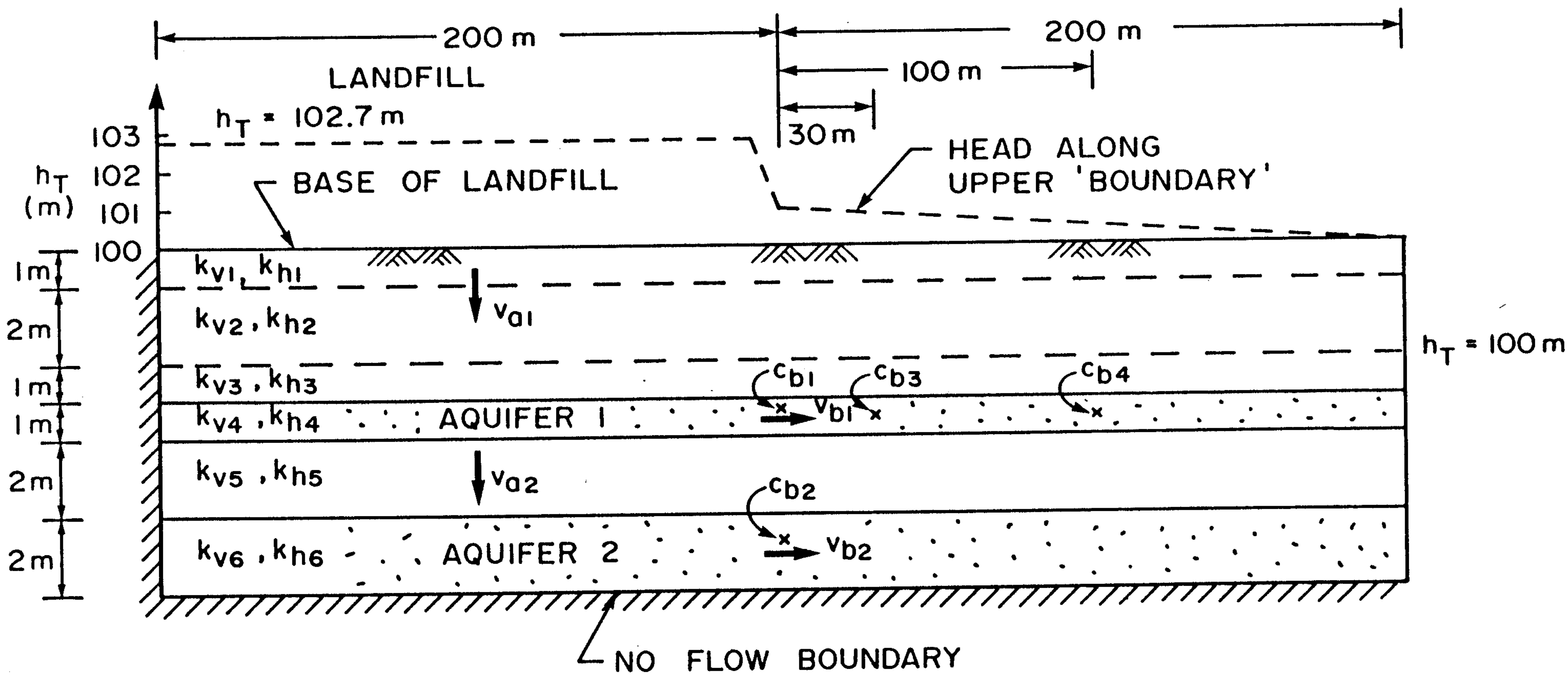

FIGURE 2 SCHEMATIC OF GENERAL HYDROSTRATIGRAPHY CONSIDERED

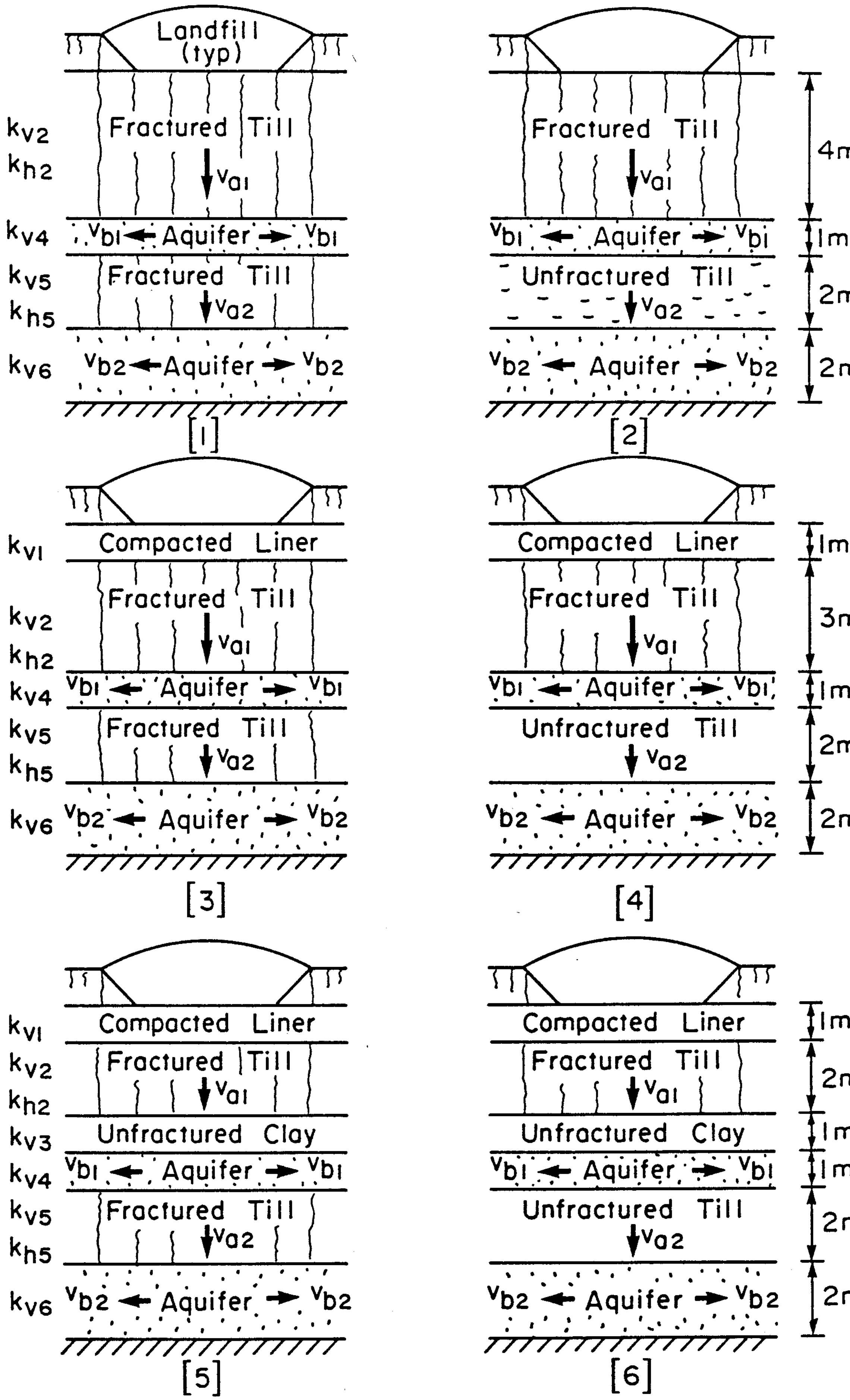

FIGURE 3 SCHEMATIC OF THE HYDROSTRATIGRAPHY FOR THE CASES CONSIDERED

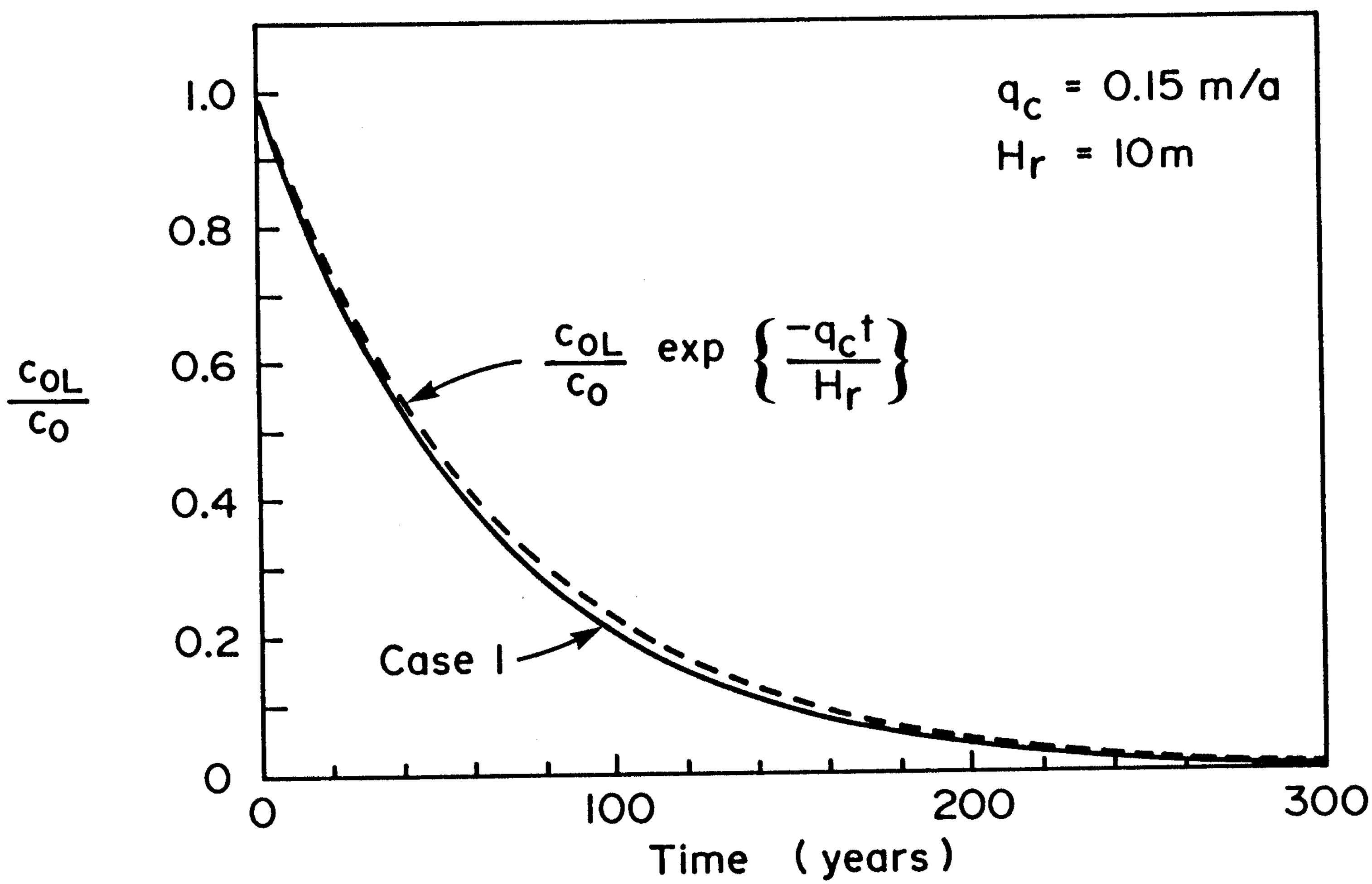

FIGURE 4 VARIATION IN LEACHATE STRENGTH WITH TIME

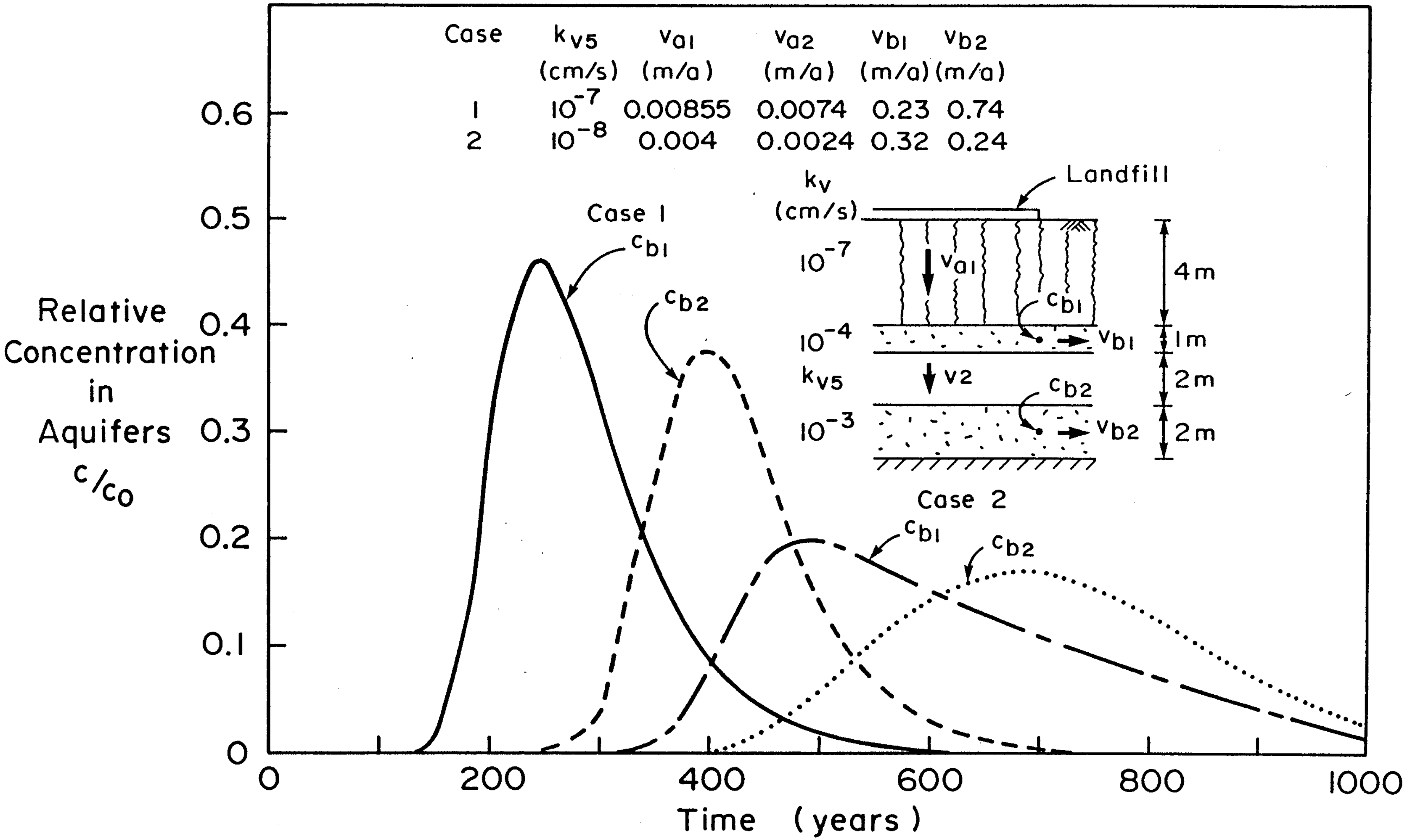

FIGURE 5 CALCULATED VARIATION IN CONCENTRATION WITH TIME AT TWO MONITORING POINTS CASES 1 AND 2

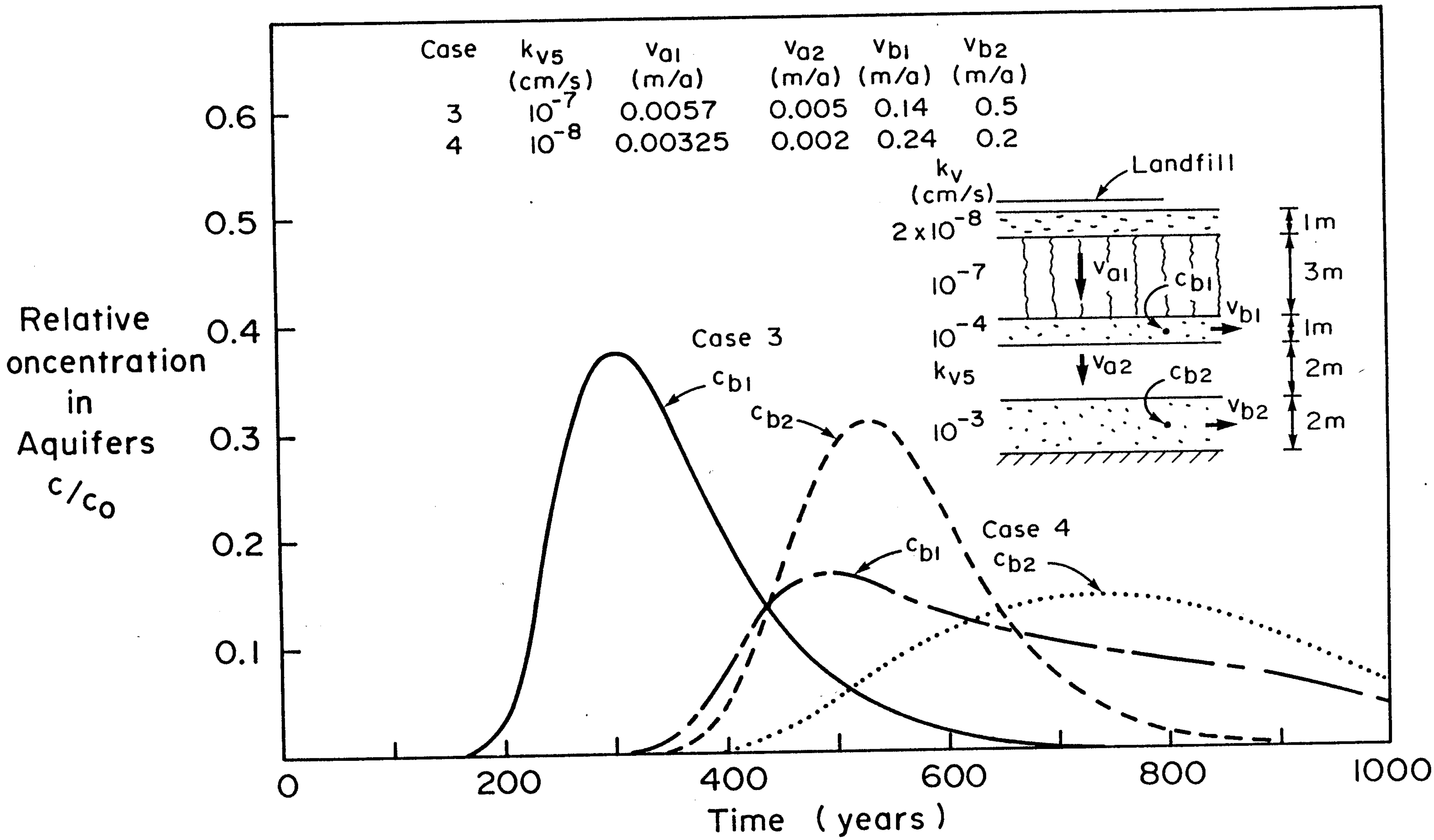

FIGURE 6 CALCULATED VARIATION IN CONCENTRATION WITH TIME AT TWO MONITORING POINTS - CASES 3 AND 4

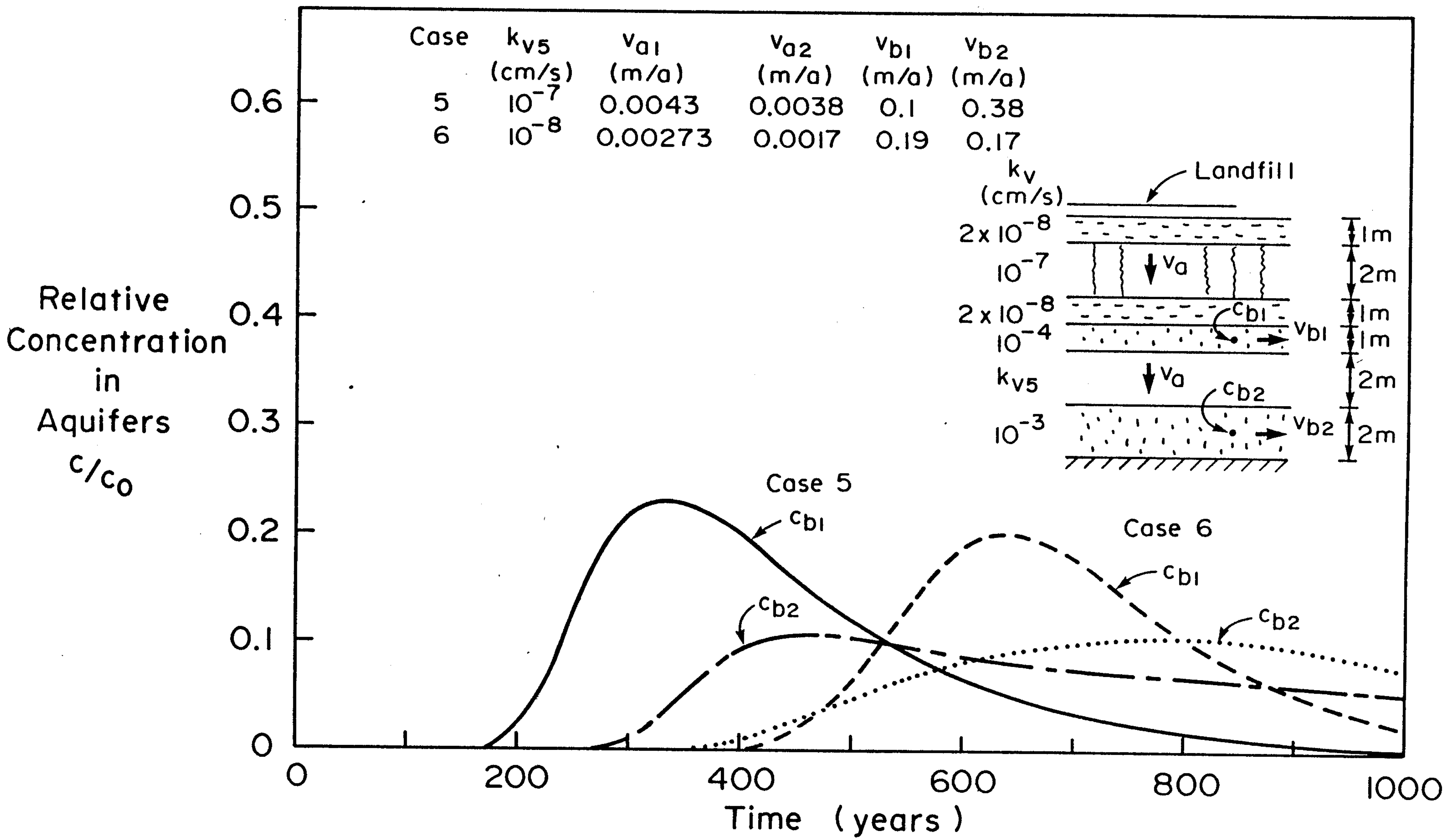

FIGURE 7 CALCULATED VARIATION IN CONCENTRATION WITH TIME AT TWO MONITORING POINTS - CASES 5 AND 6

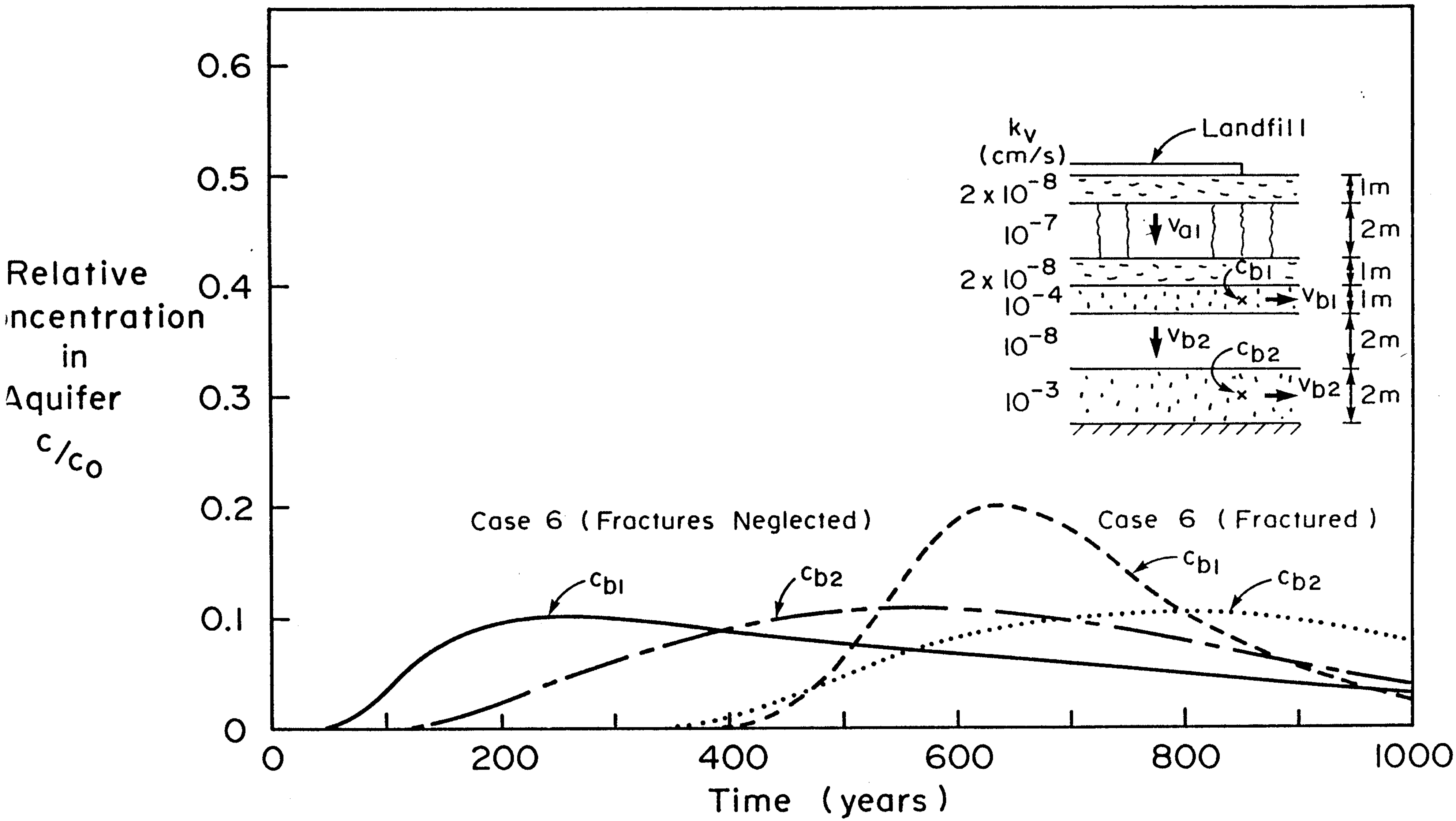

FIGURE 8 EFFECT OF MODELLING STRATEGY ON CALCULATED RESULTS - CASE 6

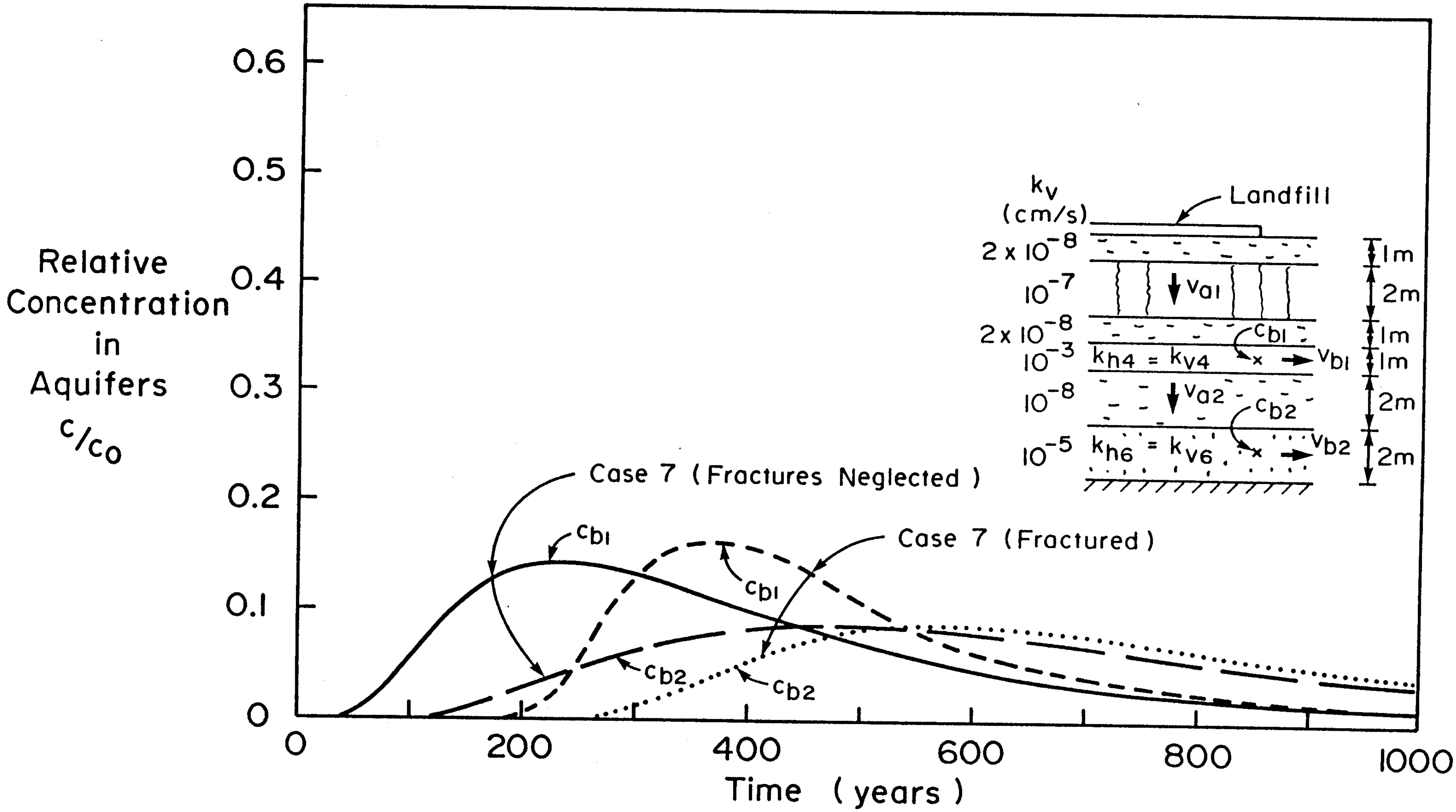

FIGURE 9 EFFECT OF MODELLING STRATEGY ON CALCULATED RESULTS - CASE 7

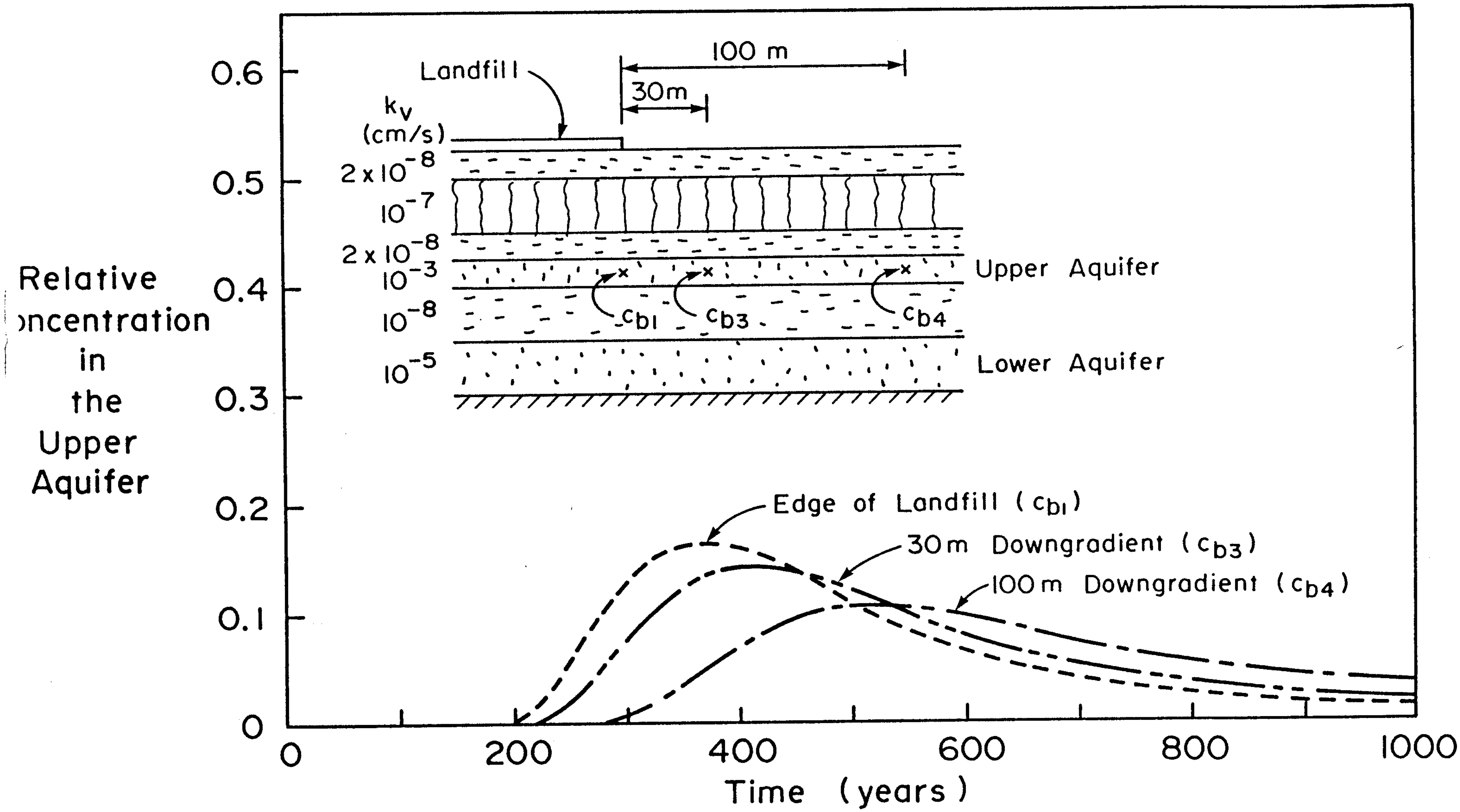

FIGURE 10 ATTENUATION OF CONCENTRATION IN THE AQUIFER (CASE 7)