RECENT ADVANCES IN MODELLING OF CONTAMINANT IMPACT DUE TO CLOGGING

R.K. Rowe

Geotechnical Research Centre
Department of Civil and Environmental Engineering
The University of Western Ontario, London, Ontario

and

J.R. Booker

School of Civil Engineering
University of Sydney, Sydney, Australia

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R. Kerry Rowe
Geotechnical Research Centre, Dept. of Civil Engineering, University of Western Ontario, London, Canada

John R. Booker
School of Civil Engineering, University of Sydney, Sydney, Australia

ABSTRACT: Recent advances in the development of finite layer theory allow the modelling of changes in the operation of an engineered barrier system for landfills. Factors that can be considered include changes in the operation of the system as the primary leachate system clogs, changes in the operation of secondary leachate collection and hydraulic control layers, and changes in the diffusive and hydraulic characteristics of geomembranes. The application of the theory is illustrated with reference to a number of standard barrier system designs and it is shown that for a large landfill some of these may eventually give rise to unacceptable impact when consideration is given to the finite service life of the engineered systems for the conditions examined.

1 INTRODUCTION

The finite layer technique (e.g. Rowe & Booker, 1985; Rowe et al., 1995a) is now widely used around the world for modelling the potential impact due to proposed new and expanded landfills. However, the needs for modelling landfill systems have expanded from the relatively simple need to be able to model advective-diffusive transport through clay liners to the need to be able to model quite complex systems involving composite liners with a geomembrane (GM) overlying a compacted clay liner (CCL) and/or a geosynthetic clay liner (GCL), multiple composite liners, primary leachate collection system (PLCS) performance, secondary leachate collection system (SLCS) performance, the effect of the presence or absence of a natural low permeability "attenuation layer" (AL) and the effect of the location, and change in location, of the potentiometric surface in an underlying aquifer relative to leachate levels in any primary or secondary collection system. For example, Figure 1 shows three barrier system designs (to be described in detail in Section 4.3.1) that may need to be considered in different regulatory environments.

Of particular interest is the ability to model changes in the operation of the system as the primary leachate collection system clogs (see Rowe et al., 1995a for a discussion of clogging), changes in the operation of a secondary leachate collection or hydraulic control layer, and, eventually, changes in the diffusive and hydraulic characteristics of geomembranes. The objective of this paper is to outline recent advances in the development of finite layer theory to allow modelling of these events and to illustrate the application of the theory for some practical cases.

2 FINITE LAYER THEORY FOR MODELLING TIME HISTORY

For both 1D (Rowe & Booker, 1995a) and 2D (Rowe & Booker, 1997) conditions, the modelling of a time history where there is a change in boundary conditions or layer properties (e.g. diffusion coefficient of a contaminant through an HDPE geomembrane as the geomembrane ages, change in advective velocity as a leachate collection system clogs and a leachate mound develops) involves modelling one set of conditions for a period of time t^*, storing the concentration history at time t^* and then using this as initial conditions for modelling the next period of time, $t' = t-t^*$, with the changed conditions.

Thus, in general terms, we seek a solution to the advection-dispersion equation

$$nD_{xx}\frac{\partial^2 c}{\partial x^2}+nD_{zz}\frac{\partial_c}{\partial z^2}-nv_x\frac{\partial c}{\partial x}-nv_z\frac{\partial c}{\partial z} = (n+\rho K_d)\frac{\partial c}{\partial t}+n\lambda c \quad (1)$$

where

D_{xx}, D_{zz}	are the coefficient of hydrodynamic dispersion in the x and z direction
v_x, v_z	are the groundwater velocities in the x and z direction
n	is the porosity
ρ	is the dry density
K_d	is the partitioning/distribution coefficient
λ	is the first order decay coefficient, and
c	is the concentration at position (x,z) at time t

such that we can change boundary conditions and layer properties at discreet time t*.

2.1 *1D conditions*

The modelling of changed conditions is most readily illustrated by considering the 1D migration of contaminant through a layer. For a given set of boundary conditions and layer properties, the modelling of contaminant migration through a layer is as described by Rowe and Booker (1985). However, if at some time t* these conditions change, then the concentrations in the layer at time t* just prior to the change can be approximated by the relationship

$$c^* = \chi e^{\Omega z} \quad (2a)$$

and represented in terms of the concentrations at the top (c_p^*) and bottom (c_q^*) of the layer such that:

$$\Omega = \frac{\ell n(c_q^*) - \ell n(c_p^*)}{z_q - z_p} \quad (2b)$$

and

$$\ell n\chi = \frac{z_q \ell n(c_p^*) - z_p \ell n(c_q^*)}{z_q - z_p} \quad (2c)$$

where z_p, z_q are the co-ordinates at the top and bottom of the layer respectively ($z_p \leq z \leq z_q$).

It is useful to introduce the time t′ which is the time that has elapsed after a change in condition (at time t*) has occurred such that

$$t' = t - t^* \quad (3a)$$

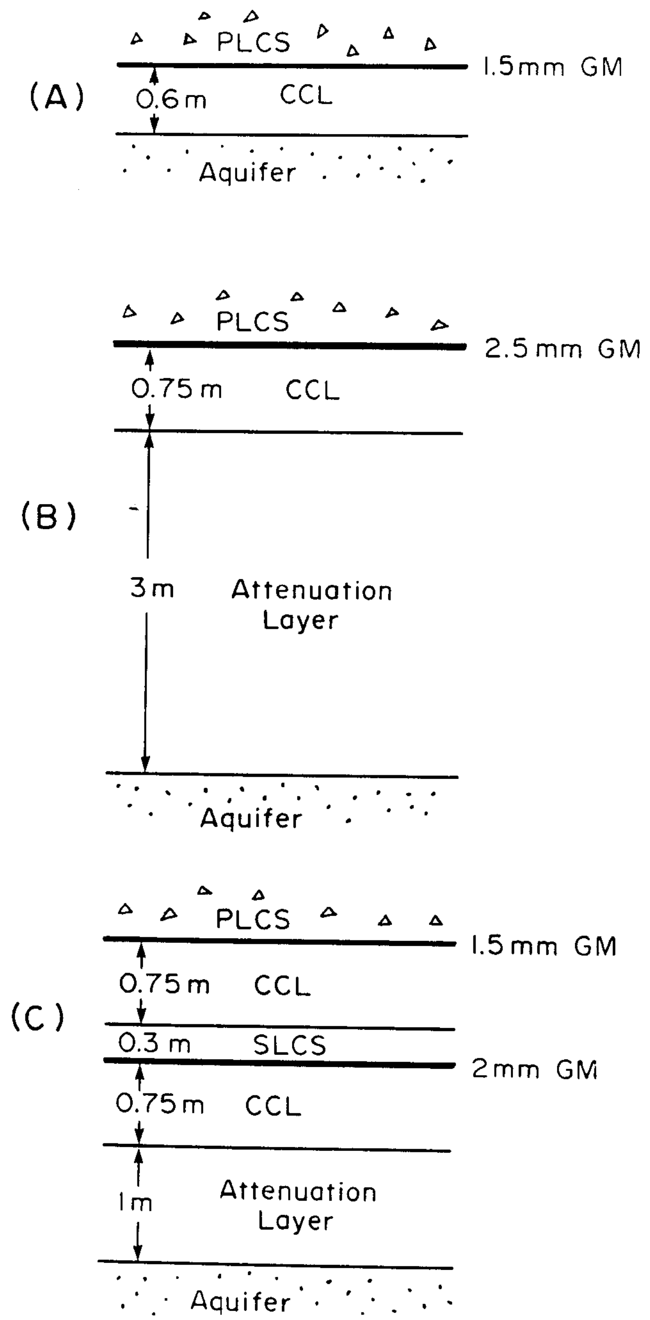

Figure 1. Schematics of three barrier systems examined.

and then introducing the notation

$$c'(t') = c(t) = c(t'+t^*) \tag{3b}$$

$$f'(t') = f(t) = f(t'+t^*) \tag{3c}$$

where c(t), f(t) are the concentration and mass flux at some depth z at time t.

Taking a Laplace transform of the 1D version of Eq. 1, it is then possible to establish a relationship between the transformed nodal fluxes $\bar{f}'_p$, $\bar{f}'_q$ and the transformed nodal concentration $\bar{c}'_p$, $\bar{c}'_q$ for any layer p

$$\begin{bmatrix} \bar{f}'_p \\ -\bar{f}'_q \end{bmatrix} = -\begin{bmatrix} A_p \\ B_p \end{bmatrix} + \begin{bmatrix} Q_p & R_p \\ S_p & T_p \end{bmatrix} \begin{bmatrix} \bar{c}'_p \\ \bar{c}'_q \end{bmatrix} \tag{4}$$

where the superior bar ($\bar{c}$, $\bar{f}$) denotes that this is a transformed function; Q_p, R_p, S_p and T_p are functions of the current layer properties (see Rowe & Booker, 1985; Rowe et al., 1995a) for $t \geq t^*$; and A_p and B_p are functions of both the current layer properties and the hereditary data (c_p^* , c_q^*) and can be derived for both unfractured and fractured layers as described by Rowe and Booker (1995a).

Considering finite mass boundary conditions which may change at time t* we have

$$c_1'(t') = c_1(t^*) - \frac{1}{H_r'}\int_0^{t'} f_1'(t')dt' - \frac{q_c'}{H_r'}\int_0^{t'} c_1'(t')dt' \tag{5a}$$

where H_r' is the current "reference height of leachate" and is a measure of the leachable mass of contaminant per unit area of landfill (see Rowe et al., 1995a) and q_c' is the volume of leachate collected per unit area ($t^* < t$) and $c_1(t^*)$ is the concentration of contaminant in the landfill at time t*.

Taking the Laplace transform of Eq. 5a, it can be rearranged into the form

$$-\bar{f}_1' = T_o\bar{c}_1' + B_o \tag{5b}$$

where

$$T_o = sH_r' + q_o' \tag{5c}$$

$$B_o = -H_r'c(t^*) \tag{5d}$$

and s is the Laplace transform parameter.

Likewise, the base boundary conditions for a landfill of length L parallel to the direction of groundwater flow and an aquifer of thickness h_b having a porosity n_b and horizontal Darcy velocity v_b' at $t > t^*$ can be written in the form

$$\bar{f}'_{m+1} = Q_{m+1}\bar{c}'_{m+1} - A_{m+1} \tag{6a}$$

where

$$Q_{m+1} = n_b h_b[s + \frac{v_b'}{L}] \tag{6b}$$

$$A_{m+1} = n_b h_b c_{m+1}(t^*) \tag{6c}$$

Thus for a multilayered deposit, consideration of continuity of flux and concentration at the layer boundaries and incorporating the boundary conditions given by Eqs. 5 and 6 gives

$$\begin{bmatrix} T_o+Q_1 & R_1 & 0 & & \\ S_1 & T_1+Q_1 & R_1 & & \\ & & \cdot & & \\ & & \cdot & & \\ & & \cdot & & \\ & & & T_m+Q_m & R_m \\ & & & S_m & T_m+Q_{m+1} \end{bmatrix} \begin{bmatrix} \bar{c}_1' \\ \bar{c}_2' \\ \cdot \\ \cdot \\ \cdot \\ \cdot \\ \bar{c}'_{m+1} \end{bmatrix} = \begin{bmatrix} B_o+A_1 \\ B_1+A_2 \\ \\ B_m+A_{m+1} \end{bmatrix} \tag{7}$$

where the coefficients T_k, Q_k, S_k, R_k, A_k, B_k (k = 1,...,m) can be determined as described by Rowe and Booker (1995a) for either fractured or unfractured layers, T_o, B_o are obtained from Eq. 5 and A_{m+1}, Q_{m+1} are obtained from Eq. 6.

2.2 2D conditions

The approach described above can be generalized to two-dimensional conditions (Rowe & Booker, 1997). In this case, the concentration at time t* in a layer can be described by a generalized form of Eq. 2 as:

$$\Phi = Ee^{\epsilon z} \tag{8a}$$

where
$$\Phi = R\int_{-\infty}^{\infty} c^*(x,z)^{i\xi x}dx \tag{8b}$$

$$R = 1 + \rho K_d/n \tag{8c}$$

$$\epsilon = \frac{\ell n(\Phi_q) - \ell n(\Phi_p)}{z_q - z_p}$$

$$\ell n(E) = \frac{z_q \ell n(\Phi_p) - z_p \ell n(\Phi_q)}{z_q - z_p} \tag{8d}$$

and
$$\Phi_p = \Phi(z_p)$$
$$\Phi_q = \Phi(z_q).$$

By taking Laplace and Fourier transforms of Eq. 1, the relationship between concentration and vertical flux for any layer can be written in a form similar to Eq. 3, replacing $\bar{f}_p^{\prime}$, $\bar{f}_q^{\prime}$, $\bar{c}_p^{\prime}$ and $\bar{c}_q^{\prime}$ by terms that reflect that a Fourier transform has also been used ($\bar{F}_p^{\prime}$, $\bar{F}_q^{\prime}$, $\bar{C}_p^{\prime}$, $\bar{C}_q^{\prime}$) and where A_k, B_k, Q_k, R_k, S_k and T_k are a function of the layer properties (D_{zz}, D_{xx} etc.) and A_k, B_k are also functions of the hereditary information at time t* (Φ_{k-1}, Φ_k, ϵ) (see Rowe & Booker, 1997 for details).

For the two-dimensional case, an aquifer can be modelled in a manner similar to that for 1D (Eq. 6a) viz.

$$\bar{F}_{m+1}^{\prime} = Q_{m+1}\bar{C}_{m+1}^{\prime} - A_{m+1} \tag{9a}$$

where

$$Q_{m+1} = hn_b\left(s + D_H\xi^2 + \frac{iv_b\xi}{n_b} + \lambda_b\right) \tag{9b}$$

and

$$A_{m+1} = hn_b\int_{-\infty}^{\infty} c_{b_o}(x)e^{-i\xi x}dx \tag{9c}$$

and ξ is the Fourier transform parameter. By invoking continuity of flux and concentration and applying Eq. 9, one can establish a system of equations:

$$\begin{bmatrix} Q_1 & R_1 & 0 & \cdots \\ S_T & & & \\ 0 & & K & \\ \cdot & & & \\ \cdot & & & \\ \cdot & & & \end{bmatrix} \begin{bmatrix} \bar{C}_0^{\prime} \\ \bar{C}_1^{\prime} \\ \cdot \\ \cdot \\ \cdot \\ \bar{C}_n^{\prime} \end{bmatrix} = \begin{bmatrix} A_1 + \bar{F}_T \\ w \end{bmatrix} \tag{10a}$$

where

$$K = \begin{bmatrix} Q_2+T_1 & R_2 & 0 & \cdots & 0 \\ S_2 & Q_3+T_2 & R_3 & \cdots & 0 \\ 0 & S_3 & Q_4+T_3 & \cdots & 0 \\ \cdot & & & & \\ \cdot & & & & \\ 0 & & Q_n+T_{n-1} & & R_n \\ 0 & & S_n & & T_n+Q_{m+1} \end{bmatrix} \tag{10b}$$

$$w^T = [A_2+B_1, \ldots\ldots, A_{n+1}+B_n] \tag{10c}$$

The surface flux $\bar{F}_T$ can be written in terms of the surface concentration (Rowe & Booker, 1997) as

$$\bar{F}_T = \psi\bar{C}_o^{\prime} - \Theta \tag{11a}$$

where
$$\psi = P_1 - Q_1R_1[e_1^TK^{-1}e_1] \tag{11b}$$
$$\Theta = A_1 - Q_1[e_1^TK^{-1}w] \tag{11c}$$

and
$$e_1^T = [1\ 0\ \ldots\ldots 0\ 1] \tag{11d}$$

Equation 11 and the Fourier inversion theorem can then be used to calculate the flux distribution at the base of the landfill viz.

$$\bar{f}_T = \frac{1}{2\pi}\int_{-\infty}^{\infty}[\psi\bar{C}_o^{\prime} - \Theta]e^{ix\xi}d\xi \tag{12}$$

and hence the average concentration in the landfill can be deduced from the mass balance equation

$$(s+\lambda+\frac{q_c'}{H_r'})\bar{c}_o^*(s)=c_o^*-\frac{1}{LH_r}\int_{-L/2}^{L/2}\bar{f}_T(x,s)dx \qquad (13)$$

3 BARRIER SYSTEMS FOR LANDFILLS

Barrier systems for landfills often consist of a leachate collection system and a composite liner involving a geomembrane over clay. Figure 1 illustrates three such systems (US, Germany and Ontario, Canada). The clogging of leachate collection systems is a primary consideration in the assessment of potential impact (see Brune et al., 1991; Rowe et al., 1995a). Rowe et al. (1997) have outlined a numerical procedure that can be used to predict the rate of clogging of leachate collection systems. MoEE (1996) have defined the service life of two different primary leachate collection systems as 75 years and 100 years, and the service life of a primary geomembrane as 150 years. Thus it is of considerable interest to examine the potential for contaminant impact based on consideration of both diffusion during the service life of the engineered system and diffusive-advective transport when the collection system and geomembrane fail. The 1D and 2D results reported in the following were obtained using the programs POLLUTE (Rowe & Booker, 1994) and MIGRATE (Rowe & Booker, 1995b).

3.1 *Operation and failure of leachate collection system*

The primary objective of a leachate collection system is to control (limit) the leachate mound and the head acting on the liner, thereby minimizing the driving forces for outward advective contaminant transport. Under normal operating conditions, a well designed and operated collection system will keep the head on the liner to less than a few centimeters over the majority of the base of a landfill. However, the design head is usually about 0.3 m.

A leachate collection system can be regarded as performing according to design even as it begins to clog and as a leachate mound begins to develop. Once the leachate mound exceeds the design value, it is no longer adequately performing its primary design function and is regarded as having "failed" or "clogged". However, this does not mean that it has become impermeable or that it is not collecting any leachate. On the contrary, even a heavily clogged leachate collection system is likely to be much more permeable than the underlying liner and it will continue to collect some leachate long after it has been judged to be clogged. But being clogged does mean that a leachate mound exceeding the design value has developed and consequently the advective transport through the liner is increased.

As a leachate collection system continues to clog, the hydraulic conductivity is further reduced and the leachate mound continues to develop until, in the limit, the magnitude of the leachate mound is controlled by one or more of (a) the percolation through the cover; (b) the escape of leachate through the liner; and (c) the escape of leachate as "side seeps". The escape of leachate through the liner can itself vary with time depending on the condition of the liner. For example, while one has a good geomembrane liner it will be much less than when the geomembrane liner fails.

3.2 *Operation and failure of a geomembrane*

A well constructed HDPE geomembrane may provide an excellent barrier to water flow (including vapour transport through the geomembrane; see Giroud et al., 1992) and diffusion of charged inorganic contaminants such as chloride (Rowe et al., 1995b). On the other hand, organic contaminants such as Dichloromethane, 1,1 Dichloroethane, 1,2 Dichloroethane etc. can readily diffuse through a well designed and constructed geomembrane (Rowe et al., 1996). Their migration will be controlled by the clay liner.

A geomembrane can be expected to have a finite service life. When this life is reached, one can expect increased diffusion and advection through the geomembrane. The factors affecting the service life of geomembranes have been discussed by Koch et al. (1988), Koerner et al. (1990), Rowe et al. (1994) and depends on a number of factors including the quantity of antioxidant in the geomembrane when placed, the presence of heavy metals, oxygen and fluid adjacent to the geomembrane, and temperature.

Based on these considerations, MoEE (1996) assign a service life of 150 years to a 1.5 mm thick HDPE primary geomembrane and 350 years to a 2 mm thick HDPE secondary geomembrane. At this time, the geomembrane can be judged to no longer perform its design function and is no longer considered in impact calculations.

4 APPLICATION OF 1D THEORY TO MODELLING OF LANDFILL IMPACT

To illustrate the application of the theory described in Section 2 and to examine some implications, consideration will be given to the predicted impact of two landfills (a "small" and "large" landfill) for the three barrier systems shown in Figure 1. Attention will be focussed on two contaminants: chloride and dichloromethane.

4.1 *Landfill characteristics*

The small landfill will be considered to have a length of 300 m in the direction of groundwater flow and 80,000 tonnes/ha of waste (on average). The large landfill will be considered to have a length of 1000 m in the direction of groundwater flow and an average waste mass of 250,000 tonnes/ha. In both cases, the infiltration through the landfill cover will be taken to be 0.2 m/a and attention will be focussed on contaminant transport in the upper 3m of an aquifer where the horizontal Darcy velocity under operating conditions is 1 m/a and the porosity is 0.3. Unless otherwise noted, it is assumed that the potentiometric surface in the aquifer is 0.6 m below the top of the liner system for all cases.

The initial concentration and mass of contaminant are based on MoEE (1996) and are given in Table 1. The assumed diffusion coefficients through geomembranes and clay and sorption characteristics of DCM in compacted clay are also given in Table 1.

The barrier systems examined (see Figure 1) were:

A. US Subtitle D Minimum Design consisting of a 1.5 mm thick HDPE geomembrane (GM) over a 0.6 m compacted clay liner (CCL) with a hydraulic conductivity of $1x10^{-9}$ m/s and no other required attenuation layer.

B. German minimum design involving a 2.5 mm thick geomembrane (GM) over a 0.75 m thick compacted clay liner (CCL) with a hydraulic conductivity of $5x10^{-10}$ m/s over a "geological barrier" or "attenuation layer" with a hydraulic conductivity of 10^{-7} m/s.

C. Double lined system similar to that proposed by MoEE (1996, as modified) for large landfills consisting of a 1.5 mm thick HDPE geomembrane (GM) over a 0.75 m thick compacted clay liner (CCL) over a 0.3 m thick secondary leachate collection layer (SLCS) over a 2 mm thick HDPE secondary geomembrane (GM) and 0.75 m of compacted clay liner (CCL) over a 1 m thick attenuation layer.

The leakage through the geomembrane was calculated based on Giroud et al. (1992) for a well constructed geomembrane with small 2.5 holes/ha and was negligible for each case while the geomembrane and leachate collection system maintain their design functionality.

In modelling impact, it was assumed that the leachate mound in all cases would build up over a 40 year period and that the fluid available for transport or removal was reduced by that required to build the leachate mound during this period. It should be noted that this assumption (and the details of how the mound build up is modelled) will affect the results of these calculations but not the general trends which are the focus of this paper.

Following MoEE (1996), all service lives are measured relative to the midpoint of the operation of the landfill (i.e. year 10 in this case) so that for service life of the PLCS of 100 years and primary geomembrane of 150 years, these are assumed to fail at year 110 and 160 respectively.

4.2 *Biodegradation of Dichloromethane*

The concentration of dichloromethane (DCM) in MSW leachate has been found (Rowe, 1995) to have a high peak concentration (see Table 1) but to experience a significant decrease with time (attributed to biodegradation). Based on MoEE (1996), the half-life of DCM in leachate was taken to be 10 years. However, there is evidence (Rowe, 1995) that the half-life in leachate could be as low as 2 years and so this was also considered. The half-life of DCM as it migrates through soil is more questionable. Three values were examined: 10 years, 50 years and infinity. The 10 year value assumes the same basic rate as in leachate (MoEE, 1996). However, the biodegradation of DCM may require other substrates (e.g. acetic acid or other volatile fatty acids) to be present (this is the subject of current research at The University of Western Ontario). Rowe et al. (1996) showed that

Table 1. Assumed contaminant characteristics

	Chloride	Dichloromethane
Initial Source Concentration[1]		
Small Landfill (mg/L)	1500	3.3
Large Landfill (mg/L)	2500	3.3
Leachable Mass of Contaminant[1] (g/kg)	1.8	0.0023
Diffusion Coefficient Through Geomembranes[2] (m^2/a)	1×10^{-7}	7×10^{-5}
Diffusion Coefficient Through Clay[3] (m^2/a)	0.018	0.018
Sorption in Compacted Clay Liner (ρK_d)	-	1.5

Notes:
1. Based on MoEE (1996).
2. Based on Rowe et al. (1996).
3. Based on Rowe et al. (1995a).

acetic acid diffuses very slowly through a geomembrane ($D \approx 9 \times 10^{-7}$ m^2/a). This could retard the degradation of DCM by an, as present, unknown amount and hence the half-lives of 10 and 50 years and infinity were modelled to assess the sensitivity of the designs to the assumed half-lives (ranging from the optimistic to most pessimistic case).

4.3 *Migration of chloride*

4.3.1 Barrier design A

Figure 2 shows the calculated variation in chloride concentration with time in the aquifer for design A barrier system for both a small and large landfill. This figure illustrates two points. Firstly, even with failure of the leachate collection system and geomembrane (year 110 and 160 respectively) the peak impact of the small landfill is less than 60 mg/L and hence would likely be acceptable in most regulatory environments. Thus, allowing for reasonable finite service lives, barrier design A is reasonable with respect to chloride for a "small" landfill.

The second observation that can be made from Fig. 2 is that for a "large" landfill, even if the geomembrane never fails, failure of the primary leachate collection system (and the consequent leachate mound - if not controlled by other means) would result in a significant increase in chloride concentration in the aquifer and a peak impact just below 200 mg/L. When consideration is given to the finite service life of the geomembrane (assumed here to fail at year 160), there is a very significant impact on the aquifer (about 430 mg/L) shortly after failure of the geomembrane. For this reason, barrier design A would not be considered

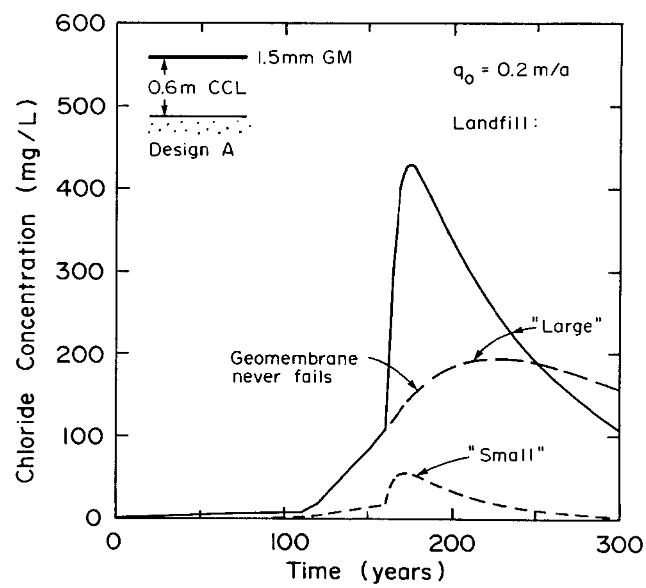

Figure 2. Variation in chloride concentration with time in the aquifer; Design A, large and small landfill.

acceptable in the Province of Ontario, Canada, where regulations (MoEE, 1994, 1996) require that consideration be given to both the finite service life of engineered components (such as leachate collection systems and geomembrane liners) for the "contaminating lifespan of the landfill" (i.e. for as long as the engineering is required to protect the environment).

4.3.2 Barrier design B

Figure 3 presents results for barrier design B corresponding to the cases considered in Figure 2 for barrier design A. Designs A and B differ in the thickness of both the geomembrane and attenuation layer thickness, however, if one considers the finite service life of the geomembrane, the peak impact occurs shortly after failure of the geomembrane and is, to all practical purposes, the same for both barrier designs for the conditions considered. However, if one assumes an infinite service life of the geomembrane then there is an effect of the thicker liners and the peak impact for barrier design B is about 25% less than that for design A.

Recognizing that geomembranes do not have an infinite service life, barrier design B gives an impact that would likely be acceptable for a small landfill but not for a large landfill in Ontario.

4.3.3 Barrier design C

Recognizing the limitations of barrier designs A and B with respect to impacts for large landfills when reasonable service lives of engineered components of the primary system (PLCS and primary GM) are considered, the Ontario draft Landfill Design Standard involves a double barrier system for large landfills, as shown schematically in Figure 4. Figure 4 also shows the calculated impact is negligible (less than 2 mg/L) for a large landfill even when the finite service life of the leachate collection systems and geomembranes is considered.

Comparing the results presented in Figures 2, 3 and 4 for a large landfill (and noting the 300 fold difference in scale) it can be seen that barrier system C has the potential to provide far greater environmental protection than designs A and B. This is primarily due to the presence of the secondary leachate collection system which serves to remove most of the contaminant that escapes through the primary liner when the PLCS and primary GM fail. This can be approached from Figure 5 which shows that the concentrations reaching the SLCS are slightly greater than the concentrations calculated in the aquifer for designs A and B (see Figures 2 and 3). However, all this

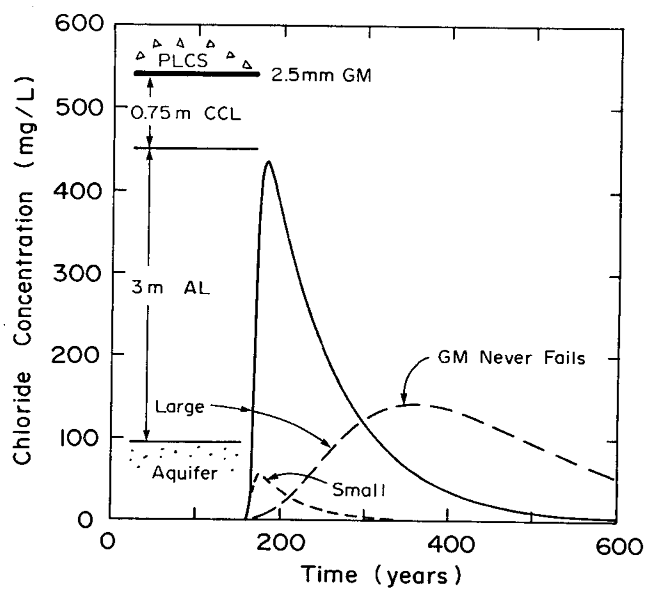

Figure 3. Variation in calculated chloride concentration in aquifer with time: Design B; small and large landfill.

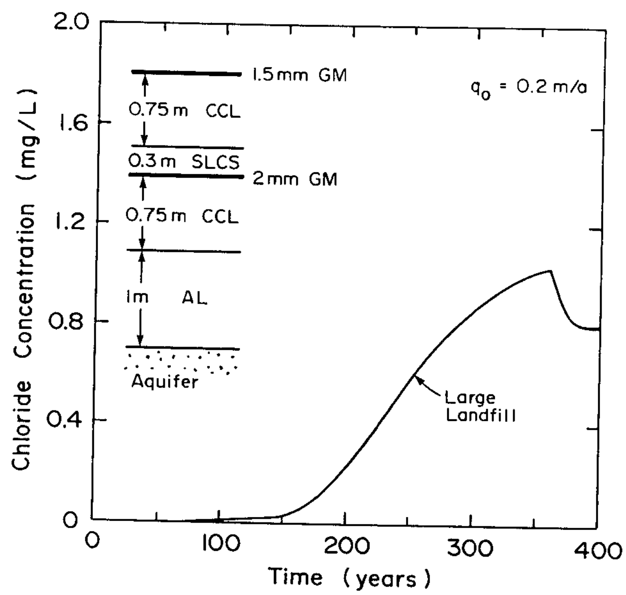

Figure 4. Variation in calculated chloride concentration in aquifer with time: Design C; large landfill.

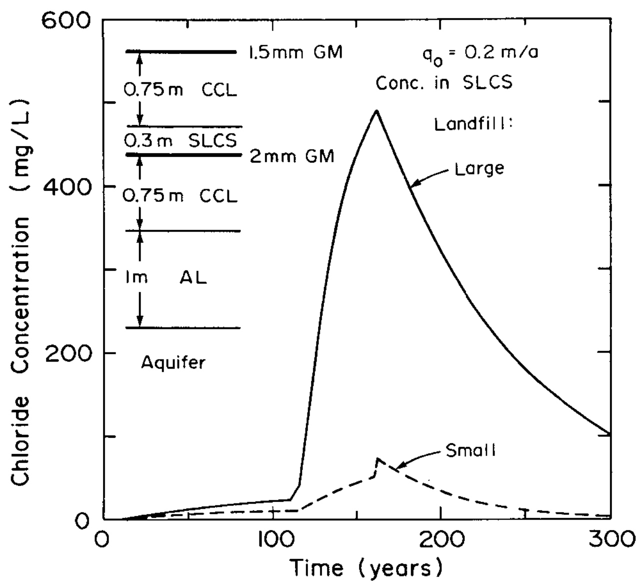

Figure 5. Variation in the calculated chloride concentration in the SLCS with time: Design C; small and large landfills.

case where there is an inward flow and hence a "hydraulic trap" after failure of the geomembrane and the case where there is outward flow corresponding to a unit gradient after failure of the geomembrane. As can be seen, the gradient does have an effect with a much larger (but still small - less than 40 mg/L) impact being calculated for the outward gradient case than for the inward gradient case after geomembrane failure. The impact for the outward gradient case would have been much greater if there had been no secondary geomembrane; this can be inferred from the case considered in the next paragraph.

The positive effect of the inward gradient evident from Figures 4 and 6 begs the question as to whether the impact would be less if there were no geomembrane. To examine this, Figure 7 shows the calculated impact for the case of an inward flow with no secondary geomembrane as well as the basic case of no flow with a secondary geomembrane present (until it fails). The impact without the secondary geomembrane is larger than that obtained with it indicating that at early times when there is a high chloride concentration in the leachate is removed and the chloride that does diffuse through the secondary liner system has negligible impact (see Figure 4).

While it is generally true that design C has the potential to give better environmental protection than designs A and B, an additional factor aiding design C in this case is the assumption that the potentiometric surface is 0.6 m below the top of the primary liner in all three cases.

While the secondary geomembrane is functioning there will be negligible flow across the secondary composite liner and contaminant transport across this liner is purely by diffusion. However, when the geomembrane fails, there will be an inward flow of water from the aquifer across the secondary clay liner and into the SLCS. This "hydraulic trap" will resist the outward diffusion of contaminants and is responsible for the drop in calculated impact evident in Figure 4 after 360 years (when the geomembrane is assumed to fail). This raises two questions: (1) what difference would there be if there were outward gradient after the geomembrane fails and (2) if there are inward gradients, would it be better to eliminate the secondary geomembrane completely.

Figure 6 shows the calculated impacts for the

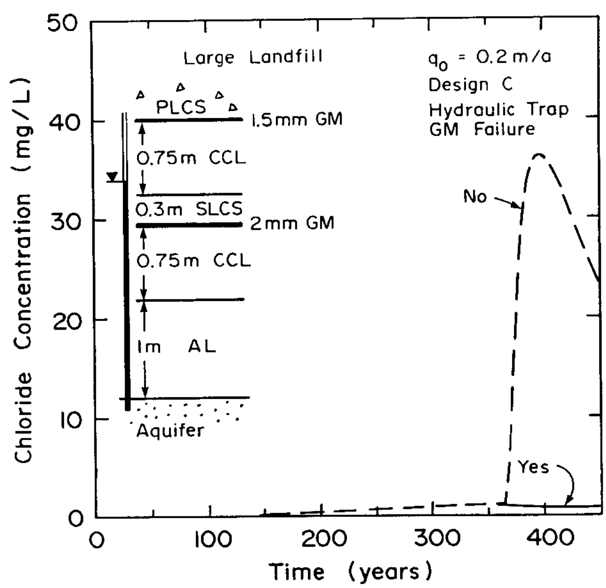

Figure 6. Effect of presence of hydraulic trap after failure of secondary geomembrane on impact in aquifer: Design C.

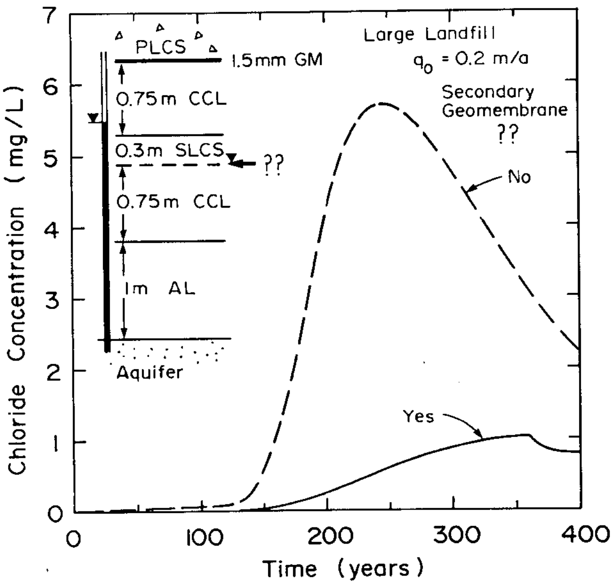

Figure 7. Effect of presence of secondary geomembrane if a hydraulic trap could be developed across the secondary liner without a geomembrane.

SLCS the secondary geomembrane provides greater resistance to outward diffusion of chloride than does the inward gradient that develops when no secondary geomembrane is present at early times. For this particular case, where there is an inward gradient, there is, however, a reasonable argument that the secondary geomembrane is not essential provided that the inward gradient can be maintained for hundreds of years. This conclusion should not be generalized and may not be valid here there are outward gradients across the secondary liner if there were no secondary geomembrane.

4.4 *Migration of dichloromethane*

As seen in the previous subsection, a HDPE geomembrane provides an excellent diffusion barrier to chloride and impacts were negligible until failure of the PLCS and the consequent build up of the leachate mound forced contaminants through the holes in the geomembrane. However, dichloromethane (DCM) can readily diffuse through a geomembrane while both the leachate collection system and geomembrane are still functioning as designed. Thus the impact due to DCM may be expected to be highly dependent on the attenuation that can occur due to sorption in the clay and biodegradation in the leachate and clay. This will become evident from the following results.

Figure 8 shows the calculated increase in DCM concentration in the aquifer for barrier design A (small and large landfills), B (large) and C (large) assuming a half-life in the leachate and soil of 10 and 50 years respectively. As might be expected, the impact for the "small" landfill was less than for the "large" landfill although by a much lesser amount than was the case for chloride. This finding is, in part, due to the fact that the assumed source concentration of chloride was lower for the small landfill than for the large landfill but the same value was used for the source DCM concentration for both landfills.

It would be desirable to be able to identify the effects of landfill size on source DCM concentrations but at present this is not possible due to

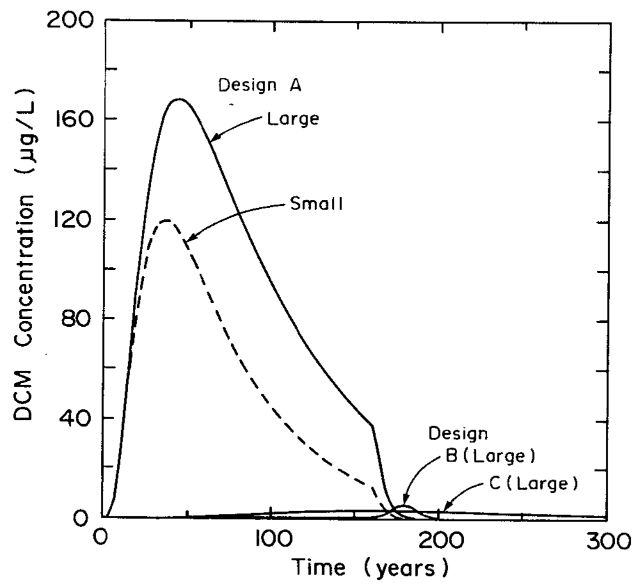

Figure 8. Calculated variation in dichloromethane (DCM) concentrations with time in aquifer: q_o=0.2 m/a; half-lives: 10 years in leachate, 50 years below geomembranes.

the paucity of good DCM data for small landfills. However, if the impact for a small landfill is less than for a large landfill based on the same source concentration, it may be expected that if the calculations performed for a large landfill meet the regulatory criteria, a smaller landfill would also meet the regulatory criteria.

Comparing the results from designs A, B and C (large landfills) in Figure 8, it is evident that the design of the landfill barrier system has a profound effect on the DCM impact. In particular, there is a much greater difference between the results for Cases A and B for DCM than was evident for chloride (where both designs gave very similar results). Each of the barrier systems will be discussed in more detail below.

4.4.1 Design A

Figure 9 shows the calculated DCM impacts for four different combinations of half-life; allowing an assessment of the performance of the barrier to be made in the context of uncertainty regarding half-life. Also shown is the maximum acceptable concentration (MAC) for drinking water (based on US EPA) and the maximum allowable concentration in

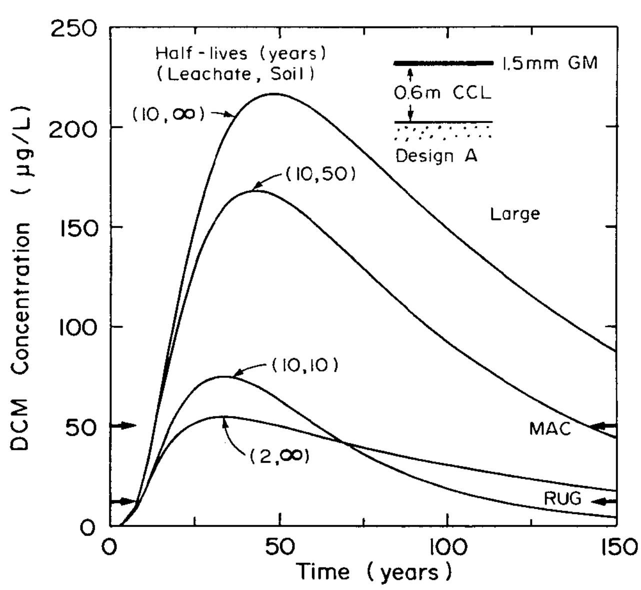

Figure 9. Calculated variation in DCM concentrations with time in aquifer: Design A, large landfill. Effect of half-lives.

drinking water based on Ontario's "Reasonable Use Guideline" (RUG) for health related contaminants.

Since design A involves a relatively thin clay liner (0.6 m) there is potential for relatively rapid migration through the clay and impact on the aquifer. In all four cases, the calculated impact exceeded the MAC within about 30 years. Thus, irrespective of uncertainty regarding the half-life of DCM in leachate and soil, the impact would be judged unacceptable. For the most conservative case (10,∞) the peak impact was almost 220 µg/L compared with the MAC of 50 µg/L and Ontario's RUG value of 12.5 µg/L. Adopting the Ontario half-lives (MoEE, 1996) the peak impact is calculated to be about 75 µg/L. If one adopts the more optimistic half-life of 2 years in the leachate (and there is some limited evidence to support this; see Rowe, 1995) but neglects any biodegradation in the soil itself (which is conservative) then the calculated impact is about 55 µg/L. Thus with luck this design might give an impact that is close to the MAC however for the assumed conditions it is most unlikely that one would meet Ontario's RUG requirements. Note that the peak impacts occur during the period when all the engineering is assumed to be working as designed.

The time before the arrival of DCM at detectable levels in the aquifer would depend on the sorption in the clay (in this example $\rho K_d = 1.5$) and the thickness of the clay. A higher value of ρK_d would give later arrival times and lower impact; a lower value of ρK_d would give earlier arrival times and greater impact. The effect of clay thickness becomes evident when one compares the results for designs A and B (see Figure 8).

4.4.2 Design B

Figure 10 shows results for cases similar to those discussed above for design A but for design B. The primary difference between the two designs with respect to DCM migration is the thickness of soil (including the attenuation layer) above the receptor aquifer. For this design, there is negligible increase in DCM in the aquifer until after the PLCS fails. This is due to the long diffusive transport times in the thick soil layer. These long travel times also allow for much greater breakdown of DCM. As for design A, the most critical case involves half-lives (leachate; soil) of (10 years, ∞) however this worst case is below the

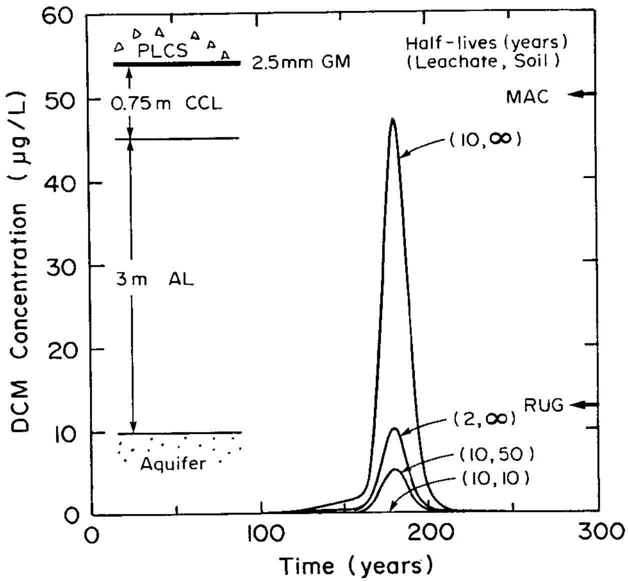

Figure 10. Calculated variation in DCM concentrations with time in aquifer: Design B, large landfill. Effects of half-lives.

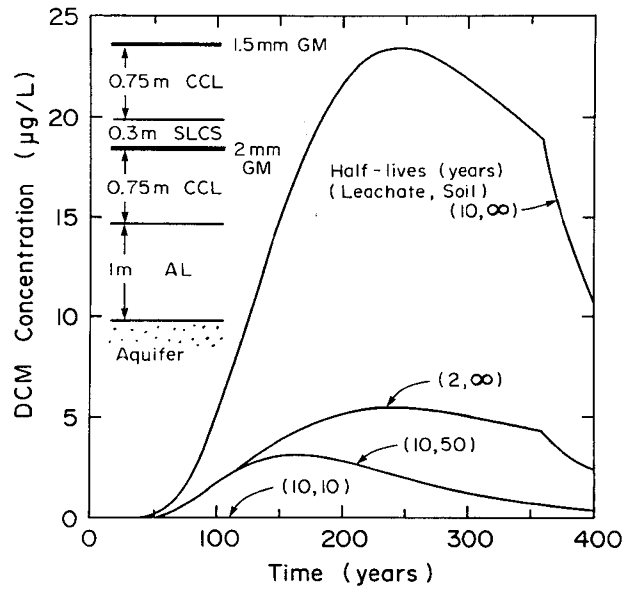

Figure 11. Calculated variations in DCM concentration with time in aquifer: Design C, large landfill. Effect of half-lives.

MAC (but only just). Considering the more likely cases of (10 years, 50 years) or (2 years, ∞) the calculated impacts are below the RUG. Note that although for design A the (2, ∞) case gave the lowest impact for design B it gives the second highest impact and cases (10, 50) and (10, ∞) give lower impacts. This is due to the greater degradation that can occur in the thicker clay if the half-life is not assumed to be infinite in the soil. For the parameters given by MoEE (1996), the calculated impact is not plottable and is well below typical method detection limits used to detect contaminants in groundwater.

4.4.3 Design C

As is evident from Figure 8, and comparing Figures 9, 10 and 11, design C (see Figure 11) tends to give the lowest DCM impacts. For the worst case half-lives considered (10, ∞) the calculated impact for design C is about half that for design B and almost an order of magnitude lower than for design A. However, due to the fact that design C has less soil than design B, DCM is detected earlier in the aquifer. When the half-life in the soil is assumed to be infinite (cases (10, ∞) and (2, ∞)) there is a change in the impact trend when the secondary geomembrane fails due to the hydraulic trap coming into effect at that time as already discussed with respect to chloride in Section 4.3.3.

For the likely range of cases (10, 10), (2, ∞), (10, 250), the calculated impact for design C is well below the MAC and below the RUG criteria. For the MoEE (1996) parameters, the calculated impact is negligible (below detection limits as was the case for design B).

5 APPLICATION OF 2D THEORY TO MODELLING OF LANDFILL IMPACT

Space does not permit a detailed examination of two-dimensional effects and the reader is referred to Rowe and Booker (1997) for details. However, some comments are warranted.

Many landfill problems do in fact closely approach 1D conditions due to the fact that liners are thin relative to the lateral dimensions and hence flow in low permeability layers tends to be vertical while flow in permeable layers is predominantly horizontal as modelled using 1D theory. This can be illustrated by re-analyzing design A examined in the previous section using 2D theory. Figure 12 shows the calculated variation in concentration with position at a number of different times. The concentration increases with position beneath the landfill as water in the aquifer moves from the upgradient to the downgradient edge of the landfill. Up to 160 years, the contaminant migration is advective-diffusive with only a small leakage through holes in the geomembrane. After 160 years, the geomembrane is assumed to fail and the contaminant movement is controlled by advection.

Consistent with previous findings (e.g. Rowe et al., 1995a), the peak impact predicted by the full two-dimensional analysis is very similar to, but slightly larger than, that predicted by the 1D analysis. In this case, the 2D analysis predicts a peak impact 16% higher and 5 years earlier than the 1D analysis. However, given the uncertainties in input data, the difference is not very significant and suggests that the 1D analysis would be adequate for this type of problem.

Two-dimensional analysis of landfill designs is most appropriate when one wishes to examine the effect of attenuation of contaminants with distance away from the landfill or if one wished to examine the effect of sequential development of landfill cells or the effect of expanding an existing landfill. This is discussed in more detail by Rowe and Booker (1997).

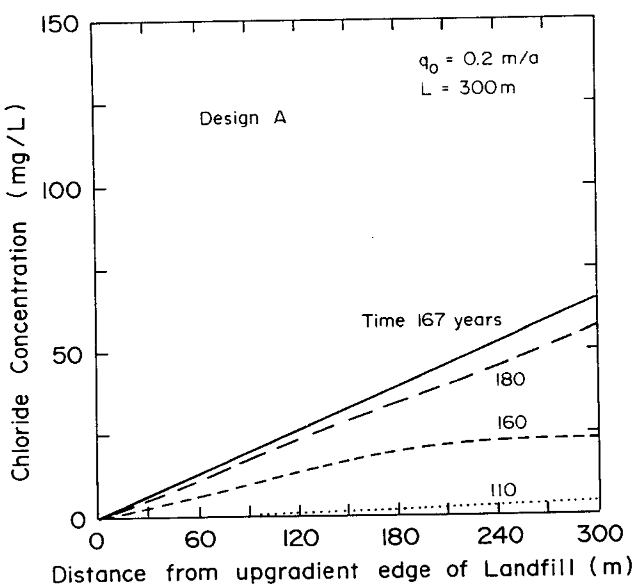

Figure 12. Development of 2D contaminant plume in aquifer beneath a small landfill - Design A.

6 CONCLUSION

Advances in the development of finite layer theory to allow modelling of changes in the operation of an engineered barrier system in landfills have been outlined. These advances allow consideration of the effects of clogging of leachate collection systems and changes in the diffusive and hydraulic characteristics of geomembrane liners.

The application of the theory has been illustrated with reference to a number of barrier system designs. The conclusions drawn from these examples relate to a specific set of assumed parameters and conditions and the specific results should not be generalized. However, they do indicate the need to carefully model each particular landfill case and not to assume that by adopting a "standard design" one is always providing adequate protection of groundwater from significant contamination.

For the specific conditions examined, it was found that a number of standard barrier designs eventually give rise to what would be regarded as unacceptable impacts in Ontario. These unacceptable impacts may arise due to diffusion of organic contaminants even when the engineered system is operating as designed. They can also arise due to the finite service of engineered systems (e.g. due to clogging of the primary collection system) unless backup engineering has been included in the barrier design.

7 ACKNOWLEDGEMENT

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