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From the Past to the Future of Landfill Engineering Through Case Histories

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FROM THE PAST TO THE FUTURE OF LANDFILL ENGINEERING THROUGH CASE HISTORIES

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ABSTRACT

The advances in landfill engineering are outlined based on a number of case histories illustrating past problems, hydraulic performance of clay liners, diffusive transport through liners, hydraulic containment and clogging of leachate collection systems. The importance of conventional geotechnical considerations (e.g. stability) will also be highlighted with reference to a number of cases. Finally, the recent advances in landfill operations are illustrated with respect to a modern landfill. It is concluded that, provided all key failure mechanisms are considered in the design, construction and operation of the facility, modern landfills should provide environmental protection both today and well into the future.

KEYWORDS

Landfills, contaminant transport, liners, hydraulic conductivity, diffusion, hydraulic containment, leachate collection, clogging, geosynthetics, stability, landfill operations, case histories.

INTRODUCTION

From the earliest human tribes to today, mankind has generated and disposed of waste. With increases in population, population density and industrialization, there has been a dramatic increase in both the amount and toxicity of waste that requires disposal. Historically, a waste disposal site was typically a convenient hole in the ground that could be filled with waste. In small enough quantities, this waste can be assimilated by nature without negative impact. However, as small dumps grew, so did the problems until it was recognized that a simple hole in the ground was generally not good enough. This led to the development of the modern engineered landfill. By examination of a number of cases, it is the objective of this paper to highlight some of the problems that arose from uncontrolled dumping (the past), discuss some of the engineered solutions and some additional challenges that accompany these solutions (the present), and discuss how future impacts of modern landfills can be minimized and some emerging trends in landfill design , operation and closure (the future). The reader particularly interested in the use of geosynthetics in landfills is also referred to Koerner and Soong (1998) and Rowe (1998) for details regarding cover stability and barrier design respectively.

LOVE CANAL

Media attention given to Love Canal in the late 1970s, and the identification of a large number of similar sites heightened public awareness and concern regarding waste disposal. This concern ultimately led to the development of restrictions on waste disposal, new techniques for disposal, increased interest in the three Rs (reduce, reuse, recycle) and the modern engineered landfill. Cohen et al. (1987) provide an excellent description of the problem of clean up of Love Canal as summarized in the following three paragraphs.

Love Canal had originally been planned to move water from the Niagara River to a proposed hydroelectric plant. However, the project failed in 1896 after about 10% of the proposed canal had been excavated. The resulting unneeded "hole in the ground" was about 900 m long, 12-30 m wide and 2.4-4.6 m deep. Between 1942 and 1953, the Hooker Chemicals and Plastics Corporation widened and deepened portions of the canal and excavated pits outside the canal and then filled the hole with approximately 20,000 tonnes of chemical wastes which included numerous chlorinated hydrocarbons and, in particular, 2,3,7,8 tetrachlorodibenzo-p-dioxin (2,3,7,8-TCDD) which is a highly toxic byproduct of 2,4,5-trichlorophenol production.

Concern about Love Canal began to develop in about 1976 due to (a) waste subsidence and exposure of drums; (b) contaminated water ponding in backyards adjacent to the dump; (c) unpleasant chemical odors; (d) movement of contaminants into the basements of houses close to the landfill; and movement of contaminants into and through the local sewer system. These factors and related health problems (e.g. greater than statistical norms for spontaneous abortions and low birthweight infants in the area) resulted in President Carter declaring a State-of-Emergency at Love Canal in 1978. This involved the evacuation of 236 families from the homes around the landfill, closure of the local school, implementation of a containment plan for part of the site, and further investigations. Preliminary results of studies of 36 Love Canal residents indicated that 11 of the 36 had chromosomal abnormalities. Publication of these results was followed by the declaration of a second State-of-Emergency in 1980.

The geologic cross-section through Love Canal is shown in Fig. 1. Hydrogeologic studies indicated that a leachate

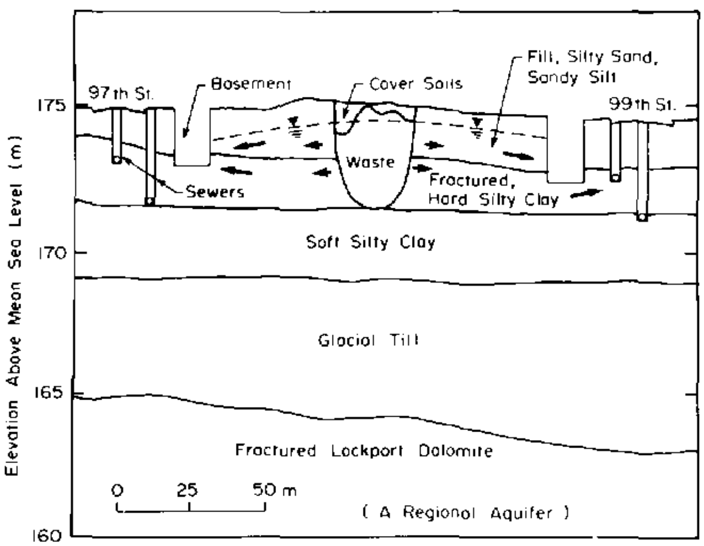

Fig. 1 A schematic geological cross-section through Love Canal showing water table and flow directions prior to remediation (modified from Cohen et al., 1987).

mound that had developed in the dump (Love Canal) was giving rise to radial groundwater flow through the overburden soils and downwards towards the bedrock. Dense non-aqueous phase liquids (DNAPLs) were found in the fractured silty clay layer. It appears that sump pumps had contributed to the movement of contaminants towards the basements. Likewise, the presence of the sewers below the water table may have caused a gradient towards the sewers that contributed to the movement of contaminant into the sewers.

IMPLICATION FOR MODERN LANDFILLS

The Love Canal case highlights a number of key concerns that had a significant influence on the development of engineered landfills including:

1. The risks associated of human contact with contaminated water and air from hazardous waste disposal sites. By inference, this concern has also been attributed to Municipal Solid Waste although clearly the risks are much less. These risks can be minimized by appropriate siting, design and operations of an engineered landfill facility.

2. The high risk associated with the disposal of barrels of liquid hazardous waste in a landfill. Modern engineered landfills usually will not accept liquid hazardous wastes and separate municipal waste landfills have significant restrictions on the acceptance of any concentrated hazardous wastes. This led to the development of special hazardous waste landfills which generally require a higher level of hydrogeologic predictability and protection and/or higher levels of engineering than municipal solid waste sites. Also, it has lead to the development of processes for reduction in the amount of liquid waste generated, techniques for solidifying liquid waste and alternative techniques for destroying (rather than landfilling) certain hazardous wastes (e.g. PCBs).

3. The potential for ground and surface water contamination that arises from placing waste in an unlined dump with no leachate control and hydrogeology unsuitable for controlling contaminant migration. This led to a requirement for appropriate hydrogeologic investigations prior to siting a landfill and often the requirement for either a suitable natural hydrogeologic barrier (e.g. thick intact clay) or one or more engineered liners (e.g. clay or composite with a geomembrane over clay).

4. The potential for rainwater infiltrating a waste mound and subsequently causing leachate migration to both surface and groundwater. Modern landfills typically include a leachate collection system to collect and remove leachate, thereby controlling the head acting on any liner system and minimizing the risk of leachate seeps. Landfills are also now typically designed with an engineered cover. The cover will usually be designed to minimize the risk of: (a) erosion exposing waste; and (b) leachate seeps contaminating surface water. The cover may also be designed to aid in gas collection and to control the amount of leachate generated.

5. The potential for contaminant migration through macrostructures (e.g. fractures) in clay layers. Generally, waste is no longer placed in fractured clay or rock without providing a liner system and/or hydraulic control to minimize the potential for contaminants migrating away from the site through fractures.

6. The potential for urban development (in this case basement and sewer construction) encouraging the movement of contaminants towards a potential receptor. This indicates the need to consider not only the existing but foreseeable future conditions when evaluating the suitability of a proposed site and design. When planning new developments, there is also a need to consider how the development may impact on the performance of any existing waste disposal site.

7. The difficulties and costs of remediation after contaminants have escaped from a dump and hence the economic benefit of spending more on selecting, designing and operating a site that would control contaminant escape to a trivial level.

As a consequence of Love Canal and numerous similar but less publicized other problems arising from old dumps, modern landfills often include a natural and/or engineered liner system, a leachate collection system, an engineered final cover and operation procedures that minimize and mitigate the potential impacts. Cases like Love Canal focussed minds on the potential for advective contaminant transport and DNAPLs causing major problems. While it is indeed important to eliminate concentrated DNAPLs from landfills and to minimize advective transport through the use of natural or engineered liners, it does not follow that if there are no DNAPLs and there is negligible advective flow then there will be negligible impact. As will be discussed in later sections, the effect of diffusion of dissolved organic and inorganic contaminants is often overlooked and can be significant.

While most landfills now incorporate leachate collection systems, it is far from clear that many of these systems have been designed to function long enough to adequately control impact for the contaminating lifespan of the landfill (i.e. the period of time during which the landfill could have an unacceptable impact if contaminants escaped). The clogging of leachate collection systems becomes a key issue here and will also be discussed in a later section.

The increased cost associated with the siting, design and construction of a modern engineered landfill combined with public concern about landfills has tended to cause an increase in the size of landfills (both in areal extent and thickness of waste). This has three potential effects. Firstly, it increases the contaminating lifespan (Rowe, 1991) of the landfill and hence increases the period of time that the engineered system is required to function. Secondly, it increases the potential mass loading on the environment and consequently increases the risk of unacceptable impact unless higher levels of engineering and natural protection are required to match the greater mass loading (MoEE, 1997). Thirdly, the construction of larger landfills with more sophisticated liner systems increases the risk of conventional geotechnical failures (e.g. stability problems) as discussed later.

Suitable engineered covers can greatly improve the performance of an engineered landfill. However, covers involving various drainage layers, while reducing leachate generation and gas collection, can also cause problems with respect to cover stability. Of particular concern is the development of seepage forces which can cause instability in landfill covers. A number of failures have occurred involving sand (or silty sand) over clay with 2.5:1 or 3:1 side slopes. The failures are typically related to (a) low initial hydraulic conductivity of the soil; (b) accumulation of fines in stone near a drainage pipe; or (c) clogging of a filter geotextile placed around a drainage pipe. The issue of cover stability is discussed in detail by Koerner and Soong (1998) and will not be discussed further in this paper.

CLAY LINERS

Modern landfills are frequently lined with either a natural or engineered clay liner. There are two primary contaminant transport mechanisms through clayey barriers: advection and diffusion. Advection (the movement of contaminants with flowing water) is controlled by the bulk hydraulic conductivity and hydraulic gradient across the clay liner/deposit. Diffusion (movement of contaminants from high concentration to low concentration) is controlled by the effective diffusion coefficient and the concentration gradient. The importance of advection is generally well recognized; the importance of diffusion is often totally overlooked. Both will be discussed with respect to a number of field cases in the following sections.

HYDRAULIC PERFORMANCE OF CLAY LINERS

Intact Clay Deposits and Compacted Clay Liners

Natural clay deposits have the potential to provide low hydraulic conductivity and a high level of natural attenuation. Two excellent examples include the La Salle Road Landfill in a thick silty clay till deposit with a hydraulic conductivity of about $2x10^{-10}$ m/s (Barone, 1990; Barone et al., 1991) and the Confederation Road Landfill constructed in a thick clay deposit with a hydraulic conductivity of approximately $1x10^{-10}$ m/s (Goodall & Quigley, 1977; Quigley & Rowe, 1986; Rowe et al., 1995b). There was no fracturing below the weathered crust in either case and the hydraulic conductivity of the matrix represented the bulk hydraulic conductivity. As will be demonstrated in the following section, in both cases contaminant transport was controlled by diffusion and advection had negligible effect on contaminant migration at these two sites.

One common concern regarding the hydraulic conductivity of clay is the potential effect of clay-leachate interaction and the potential for an increase in hydraulic conductivity with time due to this interaction (see Rowe et al., 1995b, Chapter 4 for a detailed discussion). However, while this is a valid question and should be considered, the writer is not aware of any cases where there has been a measured increase in hydraulic conductivity of a low activity clay liner (under actual field conditions) in contact with municipal solid waste leachate. To the contrary, studies conducted at the Confederation Road Landfill site provide a suggestion that the hydraulic conductivity near the interface between the waste and the clay actually decreased (Fig. 2). These profiles indicate contaminant migration to a depth of about 1 m and a hydraulic conductivity in the upper 0.2 m that is less than for the rest of the deposit despite the fact that the water content at the stress ranges used for hydraulic determination is essentially constant. This suggests that the slight drop in hydraulic conductivity at the interface is chemically controlled. As discussed by Rowe et al. (1995b), there was a decrease in pore size near the interface which is supportive of a decrease in hydraulic conductivity (e.g. due to heavy metal precipitation) but is not definitive since there are other possible explanations for the reduction in pore size. In any event, the key observation is that the hydraulic conductivity did not increase.

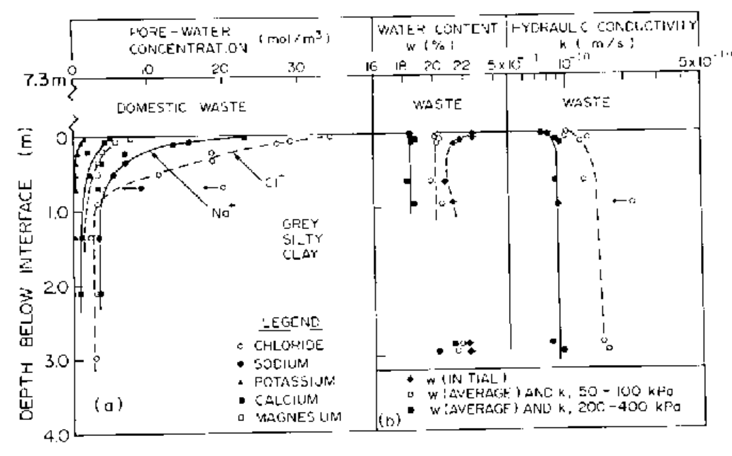

Fig. 2 Clay-leachate interaction at interface, BH 83-2, t=15 years. (a) Pore water chemistry; (b) Hydraulic conductivity and bulk water contents calculated from oedometer tests at pressures just below and above the pre-consolidation pressure of 150 kPa (after Rowe et al., 1995b).

The performance of a compacted clay liner at the Keele Valley Landfill has been described by Reades et al. (1989) and King et al. (1993) and the lysimeter data from King et al. (1993) is replotted in Fig. 3. This data (see also Table 1) shows a decrease in hydraulic conductivity with time to below $1x10^{-10}$ m/s (likely due to consolidation of the compacted clay liner) and certainly does not show an increase due to interaction with leachate. Again, as will be discussed later, contaminant transport through this liner is predominantly by diffusion and not advection.

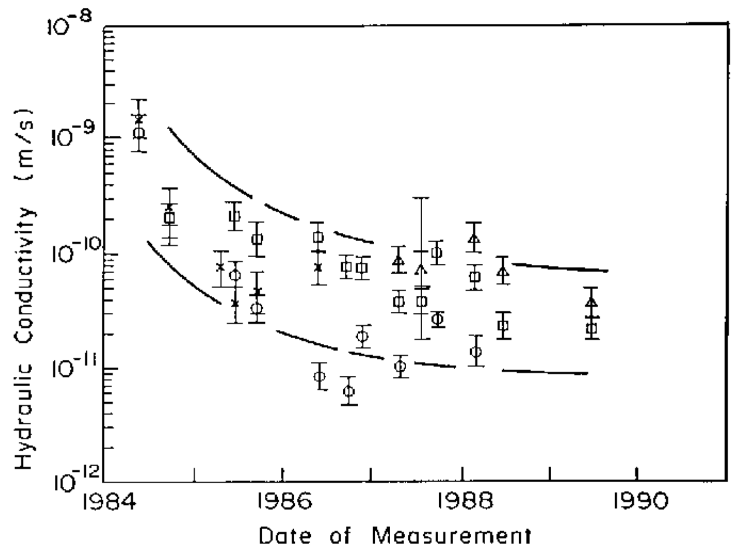

Fig. 3 Estimated hydraulic conductivity based on shallow lysimeter effluent flow rates; Keele Valley Landfill. Range bars represent possible error limits; squares, circles and triangles represent mean values within range limits. (Modified from King et al., 1993; after Rowe et al., 1995b.)

As a third and final example, Gordon et al. (1989) have reported that the hydraulic conductivity of clay liners at three Wisconsin landfills have significantly reduced with time (based on lysimeter data) with hydraulic ocnductivity values being in the range shown in Table 1.

These examples suggest that intact low activity clay liners can provide low hydraulic conductivity and to the extent that the hydraulic conductivity changes with time, it appears to decrease. For compacted clay liners the decrease appears to be primarily due to consolidation of the liner under the weight of the overlying waste although there may also be a contribution due to "clogging of pores" due to precipitation and biological activity. However, notwithstanding this good performance and the absence of negative clay-leachate interaction effects in this case, the potential for clay-leachate interaction should be considered for each leachate and clay proposed for a landfill liner.

Table 1 Hydraulic conductivity of clay liners based on lysimeter measurements.

	k-value (m/s)
Three Wisconsin landfill liners (specified $k=10^{-9}$ m/s) (Gordon, Huebner & Miazga, 1989)	
'Sauk'	$0.6\text{-}4\times10^{-10}$
'Marathon'	$0.5\text{-}2\times10^{-10}$
'Portage'*	$1\text{-}9\times10^{-10}$
*Stabilized (Huebner & Gordon, 1995)	1×10^{-11}
Keele Valley liner (specified $k=10^{-10}$ m/s) (King et al., 1993)	
1984 (stabilizing)	$\sim5\times10^{-10}$
1988 (stabilized)	$\sim5\times10^{-11}$

Fractured Clay Deposits

The bulk hydraulic conductivity of a clayey deposit can be significantly increased by even a relatively thin fractures at spacings of the order of 1 m (or more). These fractures may occur well below the weathered crust and may be difficult to detect by conventional vertical boreholes and even with angled boreholes. For example, in the investigation at the Halton Waste Management Site (Rowe et al., 1996b, 1997a), conventional borehole and laboratory tests gave a hydraulic conductivity value of $1\text{-}4\times10^{-10}$ m/s however test pits revealed the presence of vertical fractures at a spacing as small as 0.5 m through the entire thickness of the primary aquitard. A pumping test on an adjacent aquifer was used to infer the bulk hydraulic conductivity of the aquitard and yielded a value of about 1.4×10^{-9} m/s. This did not meet regulatory requirements and it was necessary to construct a 1.2 m thick (1 m design effective thickness) compacted clay liner. Data collected from tests conducted during construction of the first cell indicate that the liner has a hydraulic conductivity of 1×10^{-10} m/s.

The Halton case is not unique. Table 2 summarizes a number of cases where fracturing of low permeability soil extended well below the obviously weathered and fractured zone and where boreholes generally did not provide an indication of the presence or extent of fracturing in this zone. The important lesson from these cases is that either a pumping test on an adjacent aquifer and/or deep test pits are required to establish the extent of fracturing of unweathered aquitards.

Table 2 Summary of a number of cases where fracturing was encountered below the obviously weathered zone in clayey till deposits.

		Approximate depth of weathered zone (m)	Approximate observed depth of fractures (m)
Site	1	4-5	10.5-11
	2	4-5	7
	3	4-5	10
	4	4	7
	5	4-5	>13
	6	7	>12
	7	4-6	9
	8	4-6	8-8.5
	9	4-6	10
	10	5-6	~15

Empirical Correlations

Benson et al. (1994) developed a correlation between compactor weight, plasticity index, percent gravel, percent clay and initial saturation, with hydraulic conductivity based on data from 67 landfills in North America viz:

$$\ell nk = -22.96 + \frac{894}{w} - 0.08PI - 2.87S_i + 0.32\sqrt{G} + 0.02C \quad (1)$$

where:
- k = hydraulic conductivity (m/s)
- w = compactor weight (kN)
- PI = Plasticity Index (%)
- S_i = initial saturation (decimal form e.g. $S_i = 0.95$)
- G = percent gravel (%)
- C = percent clay (%)

The authors clearly caution that the equation is in no way a replacement for measurement of hydraulic conductivity, but does provide a quick means of checking hydraulic conductivity of a compacted clay liner during quality control operations provided that the liner is compacted wet of the line of optimums and there are proper construction techniques (lift thickness, hydration time, etc.). Since both criteria are met for the Halton and Keele Valley Landfills, it is of some interest to compare the measured values with those based on equation (1). Table 3 shows that fairly reasonable results are obtained. For these two liners the correlation provides a somewhat high (by a factor of 2.5 to 4) estimate of the hydraulic conductivity under field stress conditions.

Table 3 Calculated values of hydraulic conductivity (based on Benson et al., 1994) and measured values for two Ontario landfills.

	Soil	Compactor Weight (kN)	Hydraulic Conductivity (m/s) Predicted	Observed @25kPa	@100kPa
Halton	Till				
Land-	WUT	180	$2x10^{-9}$	$1.4x10^{-10}$	$0.8x10^{-10}$
fill	WUT	300	$3x10^{-10}$	$1.1x10^{-10}$	$0.8x10^{-10}$
	UUT	300	$3x10^{10}$	$1.4x10^{-10}$	$0.8x10^{-10}$
				Field	Triaxial @145kPa
Keele	Till	300	$3x10^{-10}$	$0.7x10^{-10}$	$0.8x10^{-10}$

DIFFUSION THROUGH CLAY

The following subsection examines three long term (10,000-15,000 year) natural diffusion profiles and three shorter term (4-15 year) diffusion profiles from landfill sites through clayey barriers. In each case, the hydraulic conductivity and gradient are such that advective transport is negligible and contaminant transport is governed by diffusion.

Long Term Diffusion Profiles

Rowe et al. (1995b) summarize three cases involving the development of natural diffusion profiles through thick (30-40 m) deposits of clay during the 10,000-15,000 years since the last glacial period. In each case, advection can be shown to be negligibly small (0.0004 m/a or less).

The diffusion coefficient for chloride was deduced to range between $2x10^{-10}$ and $3.8x10^{-10}$ m^2/s (Table 4) for these three

Table 4 Chloride diffusion coefficients for three natural diffusion profiles over 10,000-15,000 years.

Site	Chloride Diffusion Coefficient (m^2/s)	Comment
Hawkesbury Leda Clay	$2x10^{-10}$	Diffusion from brackish pore water to surface Quigley et al., 1983)
Sarnia Water-Laid Till	$3x10^{-10}$	Diffusion from saline bedrock (Desaulniers et al., 1981)
Freshwater Clay	$3.8x10^{-10}$	Diffusion from saline bedrock (Rowe & Sawicki, 1992)

cases. These values correspond to diffusion at typical groundwater temperatures for the region of 7-10°C. The fact that the diffusion profiles could be so well matched to theoretical expectations is a sign of the predictability of the phenomenon of diffusion over a period of more than 10,000 years. The difference in diffusion coefficients is relatively small compared to typical variability of hydraulic conductivity. In part, the difference is attributed to different tortuosity of the three clayey soils and in part due to the need to maintain an ion balance and hence the dependence of the rate of chloride migration on the rate of migration of associated cations.

Diffusion Below the Confederation Road Landfill, Sarnia, Ontario, Canada

The diffusion of contaminant from the Confederation Landfill has been extensively studied by a series of researchers at the University of Western Ontario over a 20 year period (the first study by Goodall and Quigley being published in 1977). Figure 2 shows that over the first 15 years, chloride migrated about 1 m. An effective diffusion coefficient of $6.3x10^{-10}$ m^2/s based on short term (1 week) laboratory tests (Table 5) provides a good prediction of the observed profile (Quigley & Rowe, 1986). As shown in Fig. 2, most cations moved much less than chloride. For example, potassium experienced significant sorption (due to ion exchange) and migrated less than half the distance of chloride. Heavy metals (lead, copper, zinc, iron and manganese) migrated less than 0.2 m (likely only 0.1 m) in the same time period and had largely been removed from solution by precipitation (Yanful et al., 1988a,b).

Table 5 Chloride diffusion coefficients at three landfill sites in Ontario.

Site	Chloride Diffusion Coefficient (m^2/s)	Reference
Keele Valley	6.5×10^{-10}	Reades et al., 1989
Confederation Road	6.3×10^{-10}	Rowe et al., 1995b
La Salle Rd.	2.0×10^{-10}	Barone et al., 1991

Diffusion at the Keele Valley Landfill, Maple, Ontario, Canada

The diffusion of contaminant through the compacted clay liner at the Keele Valley Landfill has been reported by Reades et al. (1989). Over a period of 4.25 years, chloride had diffused about 0.75 m and could be well predicted using a diffusion coefficient of 6.5×10^{-10} m^2/s obtained from short term diffusion tests (Table 5). This is very consistent with the diffusion coefficient obtained for the Confederation Landfill. However, it should also be noted that these two cases illustrate how diffusion causes a rapid contaminant migration over the first few years, but the rate of advance of the contaminant front gets smaller with time. Note that at Keele Valley it had migrated 0.7-0.75 m in 4.25 years while at Confederation Road it had only migrated about 1 m in 15 years.

Of particular note at the Keele Valley Landfill was the fact that the 0.3 m thick sand blanket had experienced biological clogging (to be discussed later) and was acting as part of the diffusion barrier rather than as part of the drainage system. Also of note was the fact that organic contaminant (especially toluene) may have migrated almost 0.6 m (see Fig. 4). Due to the difficulty of getting good field concentration profiles for VOCs, the results in Fig. 4 should be interpreted with caution, however, the results do suggest that more investigation of the migration of organic components through clay liners is required.

Diffusion at the La Salle Road Landfill, Sarnia, Ontario, Canada

In contrast to the two municipal solid waste landfills examined above, the third example involves an industrial landfill where the waste stream consisted mostly of insulation materials, insulation packaging materials, rock wool, plastics and rubber. This landfill, known as the La Salle Road Landfill, is located only 6 km from the Confederation Road Landfill in the same geological deposit (the St. Clair clay plain) with the base of both landfills being located in medium to stiff, grey, silty clay till. The mineralogy of the two sites is very similar (see Table 6) and in both cases the hydraulic conductivity ($1\text{-}2 \times 10^{-10}$ m/s) and gradient (0.2) are low.

The primary difference between the two sites is the nature of the waste and the consequent leachate. Table 7 summarizes the measured range of leachate concentration for chloride, sodium and potassium. Unfortunately, these landfills were "developed" without a regular monitoring programme and hence only limited data is available. The data for Confederation Road is from two analyses performed 6 years apart. The La Salle Road data represents two locations in the waste at one point in time. It should be noted that due to the nature of the waste at La Salle, the concentration of ammonia-N (2,200-2,640 mg/L), sulphate (3,300-5,200 mg/L) and pH (9) were much higher than in normal municipal leachate.

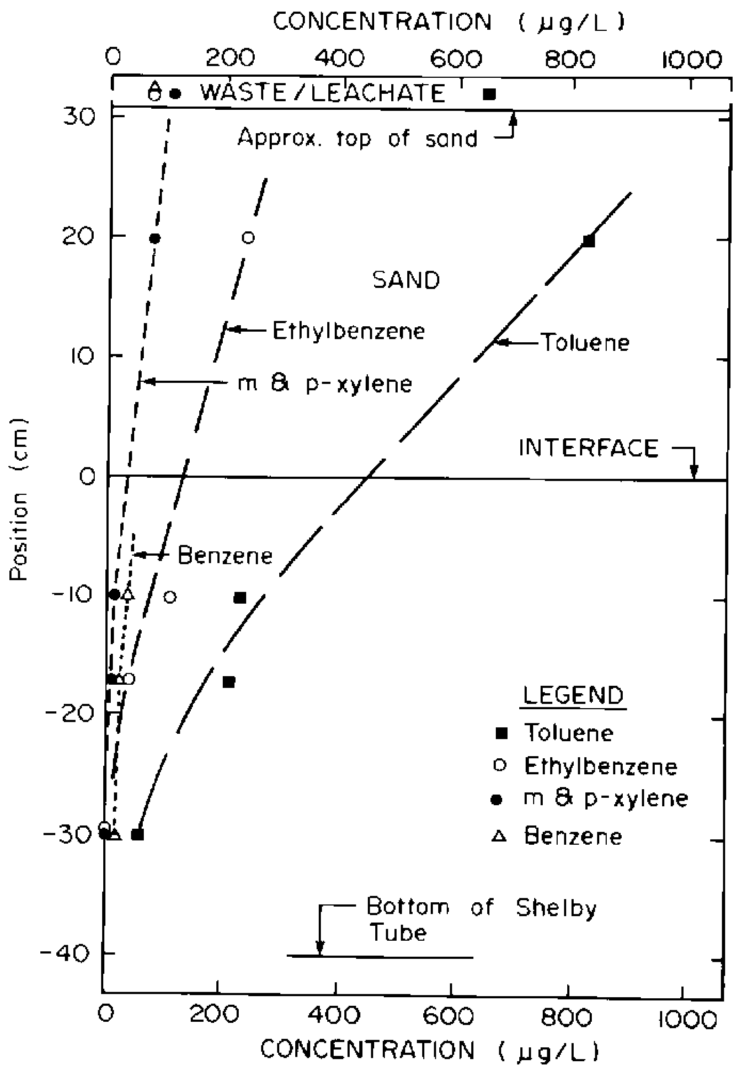

Fig. 4 Pore water organic profiles at t=4.25 years in the Keele Valley Landfill liner. (Modified from Barone et al., 1993.)

Table 6 Components of mineralogy of soils below two Sarnia landfills.

	Confederation Road	La Salle Road
Carbonates	34%	30-33%
Illite	25%	25-28%
Chlorite	24%	15-19%
Vermiculite	0%	2-3%
Quartz & Feldspar	15%	18-23%
Smectite	2%	1%
CEC meq/100g	12-15%	14-18%
Reference	Quigley et al. 1987b	Barone, 1990

Table 7 Comparison of leachate concentration; diffusion and distribution coefficients for migration through silty clay till below two Sarnia landfills. (1. Rowe et al., 1995b; Quigley & Rowe, 1986; 2. Barone et al., 1991; 3. limited data)

	Chloride	Sodium	Potassium
Measured range of leachate concentration[3] (mg/L)			
Confederation Road[1]	520-3000	525-2890	~200
La Salle Rd.[2]	970-1400	3600-5800	260-395
Diffusion Coefficient, D (m^2/s)			
Confederation Road[1]	$6.0\text{-}6.3 \times 10^{-10}$	$3.5\text{-}3.8 \times 10^{-10}$	$4.8\text{-}5.0 \times 10^{-10}$
La Salle Rd.[2]	2.0×10^{-10}	3.7×10^{-10}	5.0×10^{-10}
Distribution Coefficient, K_d (mL/a)			
Confederation Road[1]	0	0.16	3.2
La Salle Rd.[2]	0	0.3	1.0

The diffusion profile at the Confederation Road Landfill could be reasonably well interpreted using the diffusion coefficient and distribution coefficient given in Table 7 (see Quigley & Rowe, 1986; Rowe et al., 1995b for a detailed discussion). However, as will be discussed below, matching the concentration profile through the deposit at La Salle Road presented additional challenges.

Figures 5 and 6 show the observed concentration profiles with depth below the waste for chloride, sodium, sulphate and potassium after 11 years. It appears that chloride and sodium (Fig. 5) have both migrated between 1.1-1.4 m. Sulphate and potassium (Fig. 6) have migrated between 1.1-0.4 m over the same period. These figures also show the diffusion profiles calculated based on diffusion coefficients, D, and distribution coefficients, K_d (Table 7) obtained from laboratory diffusion tests (at 7°C) using leachate from the La Salle Landfill. Several observations can be made from Figs. 5 and 6 and Table 7. Firstly, in each case, the contaminant plume is overpredicted with the exception of the leading edge of sodium and chloride plumes. Secondly, although the front has reached the distance noted above, most of the contaminant is in the upper 0.4 m even for chloride and sulphate which do not interact with the soil (i.e. no sorption). Thirdly, inspecting Table 7, it can be seen that the diffusion coefficients for potassium and sodium are essentially the same for Confederation Road and La Salle Road but that there is a factor of three difference in the diffusion coefficient for chloride. Finally, the distribution coefficients for sodium and potassium are quite different at the two sites with more sorption of sodium and less sorption of potassium at La Salle Road. These differences require an explanation.

The discrepancy between the theory and observations shown on Fig. 5 can not be rectified by "adjusting" the diffusion coefficient. For example, Fig. 7 shows that the calculated concentration profile for chloride using a diffusion coefficient of 0.5×10^{-10} m^2/s provides a good fit to the upper concentration profile but fails to predict the extent of contaminant migration correctly. In contrast, the calculated

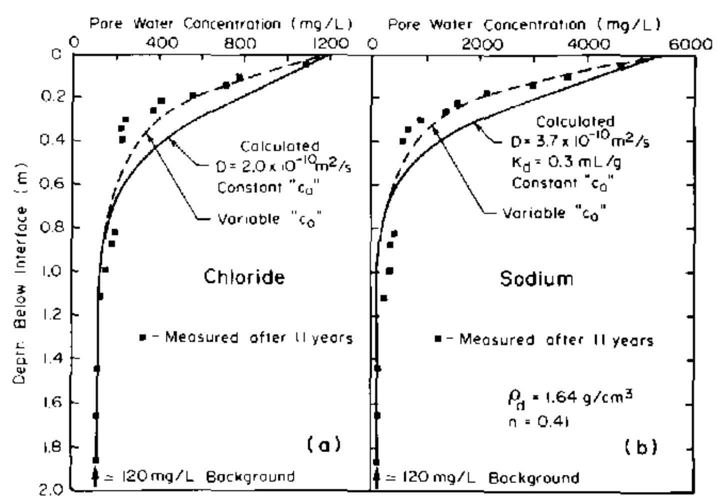

Fig. 5 Observed and calculated (a) chloride and (b) sodium migration below the La Salle Road Landfill after 11 years. (Modified from Barone et al., 1991.)

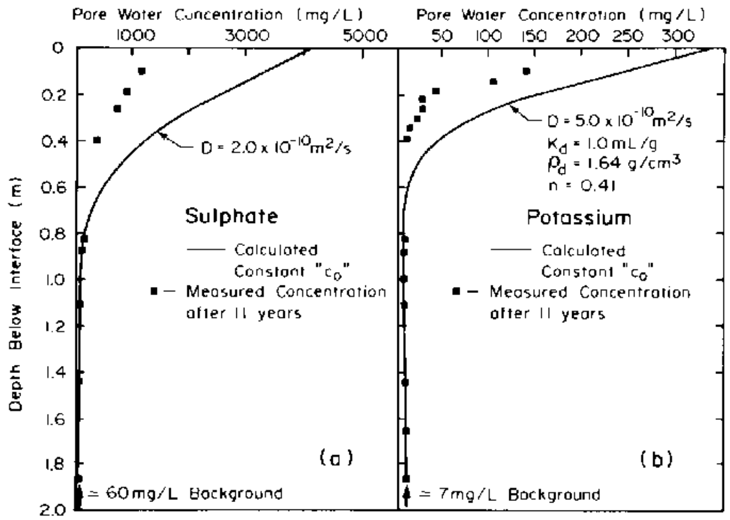

Fig. 6 Observed and calculated (a) sulphate and (b) potassium migration below the La Salle Road Landfill after 11 years. (Modified from Barone, 1990.)

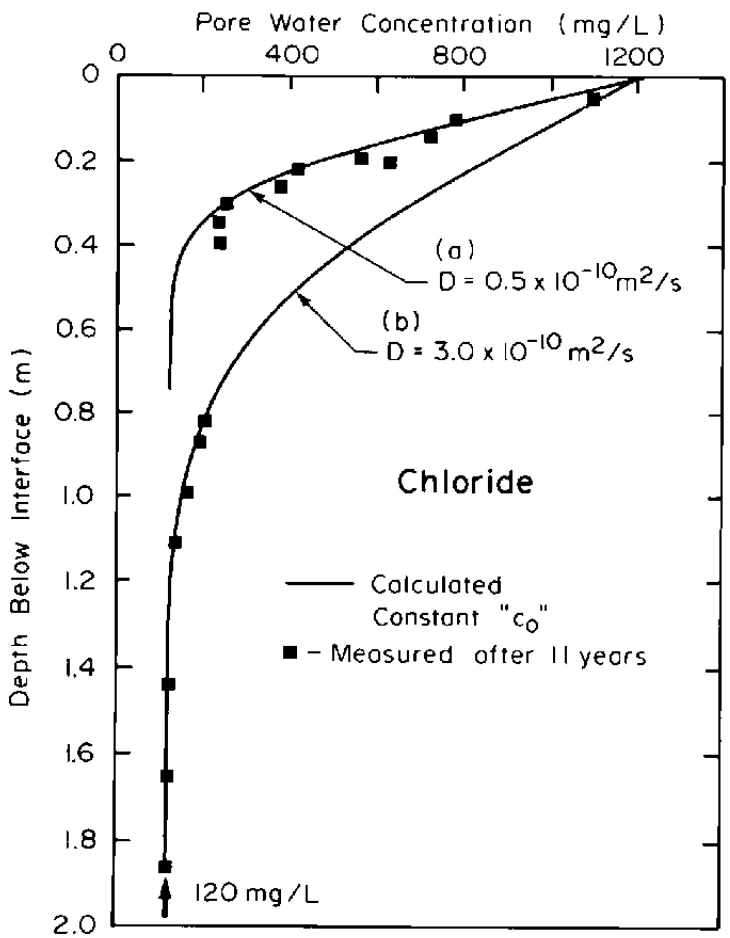

Fig. 7 Effect of diffusion coefficient on the calculated migration of chloride at La Salle Road Landfill. (Modified from Barone et al., 1991.)

profile using D=3x10^{-10} m^2/s predicts the extent of the plume but greatly overpredicts the concentration between 0.1 and 0.4 m. Although the diffusion coefficients for potassium are essentially identical for both sites and differ by a factor of three for chloride, it was found that one could get a good fit to the Confederation Road data but there was a consistent over-prediction at La Salle Road for both chloride and potassium. These observations lead one to suspect that there are different reasons for the discrepancy between chloride diffusion coefficients for sites and the discrepancy between observed and calculated profiles at La Salle Road.

The leachate data used to model La Salle Road was obtained at 11 years and the prior leachate history is not known. However, it is known (Barone et al., 1991) that the hole (which was originally excavated to provide borrow material for road construction) was pumped free of water and filled with waste starting at the west end in 1977. The cell was completed in 1980. At the location monitored, the waste was probably placed in 1978.

The cell has no leachate collection system. While the waste was placed at this location, the leachate level was generally low (less than about 2 m above the base of the cell) and was impounded by a clay dyke located about a third the cell length away from the west end. During this time, dissolution of some of the more soluble waste materials coupled with the low leachate volume can be expected to have resulted in relatively concentrated leachate. As the filling progressed beyond the dyke, the leachate level was noted to gradually rise due to accumulating rainwater and infiltrating groundwater. Eventually, when the waste had reached the east end in 1980, the leachate level was near the top of the cell and surface trenches had to be dug to route any overflow into an adjacent unfilled cell. At this time, the high water levels may have diluted the concentrations relative to the concentrations thought to have been present when the waste was first placed. Thus, it is hypothesized that high concentrations in the waste leachate over about 1 year could have been followed by a drop to relatively low levels as shown on Fig. 8. The subsequent increase in species concentrations at about 8 years is more difficult to explain. This increase may be due to slower degradation of more stable waste materials.

Calculations were performed for the concentration histories shown in Fig. 8 and the corresponding concentration profiles with depth are shown as "variable c_o" in Fig. 5. The modelling of the variable source concentrations provides a reasonable fit to the data between the bottom of the waste and 0.4 m using the diffusion coefficients and distribution coefficients obtained from laboratory tests on the soil. The fit in this region could be improved by adjusting the concentration time history, however, there is little point in doing this given the hypothetical nature of the history. The key point is that the source history can have a significant effect on the shape of the calculated concentration profile. This provides a plausible explanation for that shape of the concentration profile observed for all the contaminants and also explains the consistent discrepancy between the field observations and the profile calculated assuming a constant source concentration equal to that observed in the leachate at 11 years.

Variation in Diffusion and Sorption Parameters

What the variable concentration hypothesis for La Salle Road does not address is the under-prediction of the extent of the plume for both chloride and sodium and the discrepancy between the values of D and K_d at La Salle Road and Confederation Road.

Barone et al. (1989) conducted a series of diffusion tests using background soil from near the Confederation Road Landfill and demonstrated that for a given soil the diffusion and sorption parameters could vary substantially depending on the chemical composition of the leachate. For the range of conditions examined, they demonstrated that the diffusion coefficient of chloride varied from $5.6x10^{-10}$ to $7.5x10^{-10}$ m^2/s (for tests conducted at 10°C) while the diffusion coefficient for sodium varied from $4.6x10^{-10}$ to $5.6x10^{-10}$ m^2/s and the distribution coefficients for sodium varied from 0.15 to 0.45 mL/g. The values given in Table 7 which explain the observed diffusion profile are close to or within the range from these laboratory tests. However, the range of values obtained for the same soil for different leachates clearly indicates that the diffusion coefficient is not a fundamental soil parameter and can vary substantially due to competition for sorption sites and the need to maintain an ion balance.

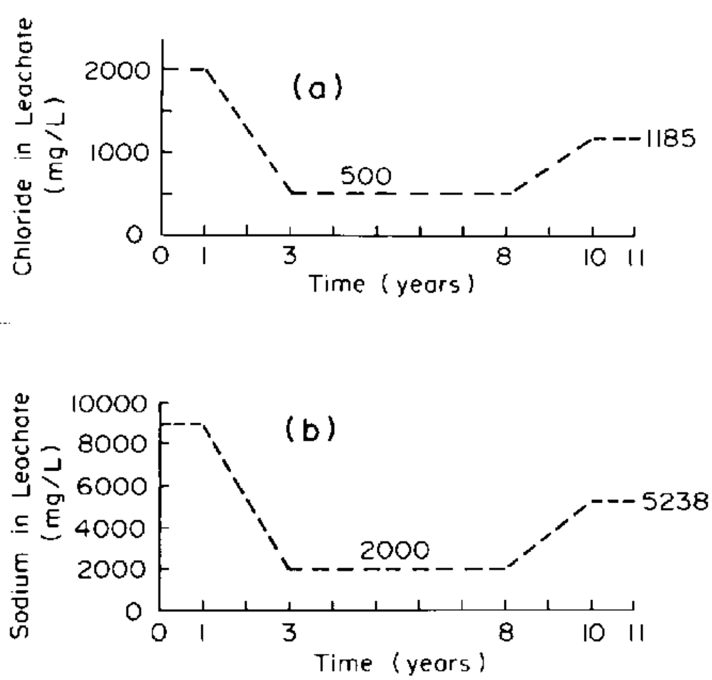

Fig. 8 Assumed concentration variation with time in the waste leachate for (a) chloride, and (b) sodium for La Salle Road Landfill with known values at 11 years. (Modified from Barone, 1990; Barone et al., 1991.)

The concentration of potassium at both the Confederation and La Salle Road sites appears to have been similar and the diffusion coefficients are similar. Potassium is a relatively heavily sorbed species compared to sodium. At La Salle Road there would have been strong competition for sorption (cation exchange) sites due to the very high concentration of ammonium (NH_4^+) and sodium (Na^+) in the La Salle Road industrial waste leachate compared to typical MSW leachate. This would explain lower sorption of potassium (lower K_d) at La Salle Road than Confederation Road. The much higher than usual sodium concentration relative to potassium likely also increased sorption of Na^+ at La Salle (K_d=0.3 mL/g) relative to Confederation Road (K_d=0.16 mL/g). Note that both values fall in the range 0.15-0.45 mL/g obtained by Barone et al. (1991) for Confederation Road soil and two different leachates.

It is hypothesized that the low diffusion coefficient for chloride at La Salle Road would appear to be related to the high concentration of sulphate (SO_4^{2-}) and competition with sulphate for cations to maintain ion balance. Examination of Fig. 7 suggests that the mobilized diffusion coefficients in the field may have been $3x10^{-10}$ m^2/s with the anomaly evident in Fig. 7 at depths of 0.1-0.4 being explained by a variable source concentration as already discussed.

More research is required to verify the hypotheses presented above to explain the difference between the diffusion parameters at Confederation and La Salle Road and the diffusion profiles at La Salle Road. However, what is very clear is the fact that (a) diffusion and sorption parameters for a given species may vary depending on soil but especially depending on the leachate composition; (b) even though chloride is considered a conservative contaminant (i.e. not reacting with soil) its migration is affected by the chemical composition of the leachate; (c) source concentration variations with time can change the shape of a calculated contaminant plume; (d) although diffusion coefficients vary from soil to soil and leachate to leachate, the range of variation in the six cases considered is between $2x10^{-10}$ m^2/s and $7x10^{-10}$ m^2/s for a wide range of conditions and diffusion periods between 4 and 15,000 years; thus, compared to other geoenvironmental parameters, it is reasonably predictable and consistent.

HYDRAULIC CONTAINMENT

Advective contaminant transport from a landfill needs to be controlled and this can be achieved by providing a low permeability barrier (e.g. a clay liner or composite geomembrane over a clay liner) and/or by controlling the hydraulic gradient. In an appropriate hydrogeologic environment, it may in fact be possible to ensure no advective transport from a landfill by designing it such that there is an inward gradient to the landfill. This is most readily achieved in areas where there is a low permeability aquitard underlain by an aquifer with a potentiometric surface close to (or above) ground level, but can be achieved for a much wider range of conditions.

In the 1960s and 1970s there was a move to ensure that landfill waste did not come into contact with groundwater. For example, many U.S. states specified a minimum separation distance between the waste and the groundwater level. The separation distances and siting rules were arbitrary and had little, if any, scientific basis (Glebs, 1980). This approach may have reduced contamination of groundwater for unlined gravel/sand pits, but in general did not prevent groundwater contamination. On the contrary, the approach had the potential of discouraging the construction of landfills in locations with ideal hydrogeologic conditions.

Studies in the mid to late 1960s showed that many landfills in clay soils, below the zone of saturation, were operating without causing problems while others in more permeable soils, above the water table, were contaminating groundwater. By 1968, this led the Illinois Geological Survey and others to the conclusion that landfills in clay sites could, with proper leachate management and engineering design, provide adequate environmental protection, even though waste was located below the groundwater table (Glebs, 1980).

In the mid 1970s, engineers and hydrogeologists in Illinois and Wisconsin (U.S.A.), and Ontario (Canada) began developing designs and design criteria for landfills that were intentionally located such that the bottom of the landfill was below the groundwater table but where the leachate head in the landfill was controlled to a level below the groundwater level (and potentiometric surface in any underlying aquifer). Under these conditions, there is a hydraulic gradient and flow into the landfill (see Fig. 9).

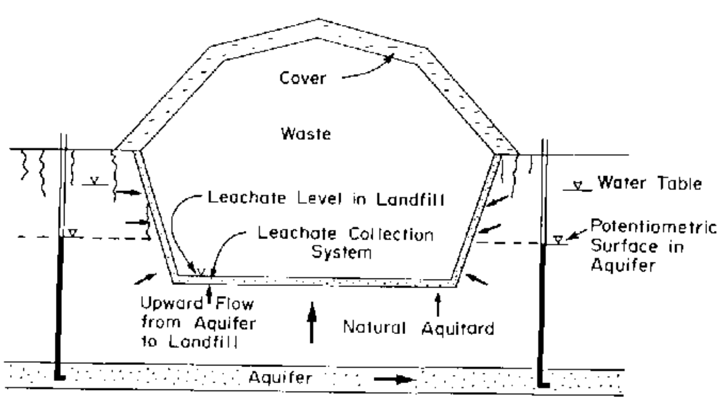

Fig. 9 Barrier design involving a leachate collection system, a natural clayey deposit, upward advection and downward diffusion - a hydraulic trap.

These conditions prevent outward advection of leachate. To minimize the amount of leachate requiring collection it is desirable that either the landfill be in a natural low hydraulic conductivity deposit or a clay liner. This type of design provides "hydraulic containment" provided that there is an inward gradient and has been referred to by some as a "hydraulic trap". By the end of the 1970s, a number of hydraulic containment landfills had been constructed in Wisconsin (Glebs, 1980). Others followed in Illinois (Burke & Haubert, 1991) and in February 1989 the first "hydraulic trap" design (the Halton Waste Management Site) was approved in Ontario. This was a benchmark decision and the approval of this "hydraulic trap" was followed by the proposal and approval of a number of other hydraulic containment landfills (e.g. Grimsby, North Simcoe, and Essex-Windsor Landfills) in Ontario.

Like all modern landfill designs, there is more to a hydraulic trap design than simply digging a hole below the groundwater table, filling it with waste and pumping leachate. Furthermore, while the inward flow of groundwater tends to inhibit outward diffusion of contaminants, there is still the potential for impact on groundwater even with a "hydraulic trap".

As noted by Rowe (1992) and Rowe et al. (1995b), the design of hydraulic containment sites involves a number of conflicting criteria and particular attention must be paid to the integration of engineering and hydrogeological considerations. For example, lowering the landfill base elevations increases the inward hydraulic gradient and hence inward advective flow. This reduces the amount of outward diffusion (which is good); however, by lowering the base contours, one also reduces the thickness of the barrier which is separating the waste from the underlying aquifer (which is not so good). Furthermore, the lower the base of the landfill the greater the potential for opening of fractures in the clay due to uplift pressure and the greater the potential for blowout of the base of the landfill. One can reduce the likelihood of blowout by depressurizing the aquifer during construction; however, this option requires careful evaluation of the number of wells required to gain adequate drawdown (i.e. lowering) of water levels in the aquifer over large areas and the potential effect on off-site water users.

In order to provide long term environmental protection, it is important that the design of all engineered landfills provides a system likely to have a service life that exceeds the contaminating lifespan of the landfill (which may be centuries for large landfills). In the case of hydraulic containment sites, the key long term consideration is maintaining the "hydraulic trap" for the contaminating lifespan. The "hydraulic trap" depends on the groundwater levels being above the leachate level in the landfill. An increase in leachate levels and/or drop in groundwater levels could both cause a loss of the hydraulic containment.

In order to maintain the hydraulic trap, it will be necessary to have some form of leachate collection to control the leachate head and so particular consideration must be given to (a) the service life of the leachate collection systems and (b) how to maintain the "hydraulic trap" if the service life of the primary leachate collection system is less than the contaminating lifespan. A second consideration is the level of confidence in the groundwater levels and an assessment of the probability that the water level will not drop. A drop could occur due to climate change, development activities upgradient of the landfill, or due to construction of the landfill itself. The third of these, the so called "shadow effect", is the result of the construction of the landfill (a) reducing recharge to the groundwater system (if located in a recharge zone) and (b) removing water from the groundwater system due to the operation of the hydraulic trap.

All of the factors discussed above were considered in the final design of the Halton Waste Management Facility to be discussed below.

Halton Waste Management Facility

The Halton Waste Management Facility was approved in February 1989 following a public hearing that ran from 5 May 1987 to 8 November 1988. Approval in principle was followed by a detailed hydrogeologic investigation and design study (Rowe et al., 1993, 1996a, 1997a). Construction began in 1991 and the first load of waste was accepted in 1992. The landfill may be regarded as one aspect of the "future" of landfilling, both because of the nature of its hydraulic containment design and its operations procedures (Rowe et al., 1996a). The following provides a summary of the containment design based on Rowe et al. (1996a,1997a).

A cross-section showing the stratigraphy and landfill is given in Fig. 10. The detailed (1990-1991) hydrogeologic investigation revealed that (i) the unweathered upper till contained some fractures through its entire thickness (typically to a depth of about 8 m from ground surface); (ii) the hydraulic conductivity of the unweathered upper till may exceed 1.4×10^{-9} m/s in some locations; and (iii) water levels in the upper granular unit (Fig. 10) had dropped by up to 2 m between the original pre-hearing investigation conducted in 1986 and the subsequent investigation in 1990-1991. Due to the fractured nature of the unweathered upper till, the conditions of approval required the construction of a 1 m thick clayey till liner constructed to the lowest practical permeability by excavating and recompacting the unweathered upper till. The drop in water levels had a significant impact on the landfill design since the leachate level was required to be at least 0.4 m below the potentiometric surface in the aquifer.

The constraints placed by the location of the bottom of the Unweathered Upper Till, the thickness of the Lower Till

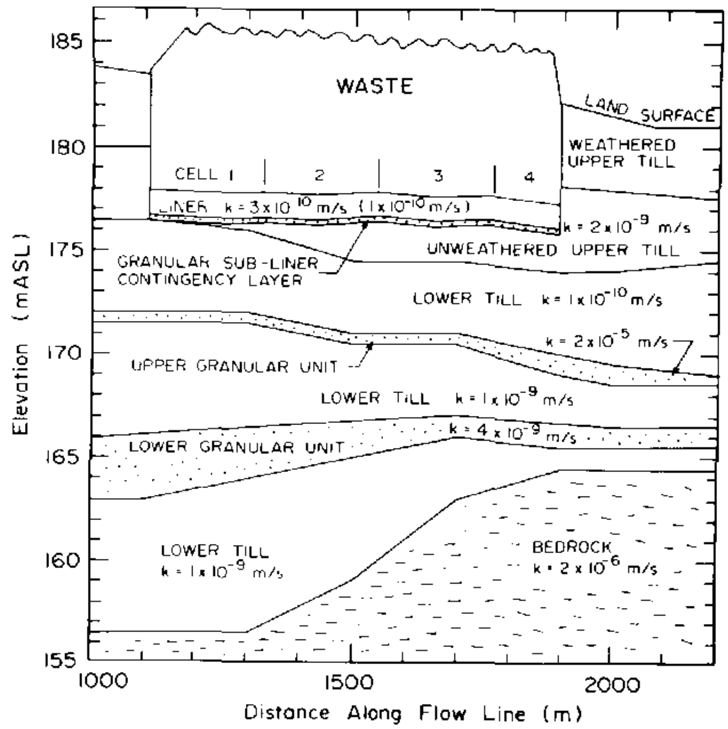

Fig. 10 Cross-section showing landfill base and stratigraphy along a critical flow line beneath the Halton Landfill. (Modified from Rowe et al., 1997b.)

and the need to maintain a "hydraulic trap" with respect to the relatively flat potentiometric surface in the Upper Granular Unit (UGU) placed severe restrictions on the development of the base contours and, in particular, on the slope of the landfill base.

The base contours were selected to (i) ensure at least 0.25% slope on the leachate collection pipes and more where practical; (ii) ensure that the lowest measured potentiometric surface in the UGU was at least 0.4 m above the design leachate mound; and (iii) minimize the excavation into the Lower Till. They were initially developed based on the hydrogeologic constraints discussed above with the objective of maintaining at least a 0.25% slope on the collection pipes. These base contours were then used in a flow analysis that was conducted to establish the potential change in head due to the "shadow effect" (see Rowe et al., 1995b, 1997a). This analysis did result in changes to base contours of the landfill and, in particular, to a lowering of the northern cells (Cells 1, 2 and 3) by up to 2 m relative to that required based on the lowest measured water level.

The final design cross-section of the barrier system, as shown in Fig. 11, involves a primary leachate collection system consisting of perforated pipes located in a 0.3-0.6 m thick stone layer over a 1.2-1.5 m thick recompacted clayey silt till liner which in turn overlies a 0.3 m thick stone layer and pipes which form the "Sub-Liner Contingency Layer" (SLCL).

The primary leachate collection system was designed to minimize the potential for clogging and to prolong its service life as discussed by Rowe et al. (1996a). To minimize the residency time of leachate within the stone drainage blanket, the liner was contoured to give a 3% slope towards the leachate collection pipes. Thus, the thickness of stone layer 2 (see Fig. 11) varied from 0.45 m at the pipes to 0.15 m midway between pipes. The specifications for the liner were developed based on a detailed trial liner investigation (Rowe et al., 1993) which demonstrated the feasibility of constructing a 1.2 m thick recompacted liner (using Unweathered Upper Till) over a stone layer (the Sub-Liner Contingency Layer - SLCL) and achieve a hydraulic conductivity of less than $3x10^{-10}$ m/s over more than 1 m of this thickness. The bottom 0.2 m of the liner was discounted due to the difficulty of obtaining good compaction of the lowermost layer which is in contact with the SLCL (see Rowe et al., 1993). Monitoring of the construction of the liner for Cell 1 indicated a harmonic mean hydraulic conductivity of approximately $1x10^{-10}$ m/s with a maximum test value of $1.6x10^{-10}$ m/s.

Due to the poor groundwater quality in the Upper Granular Unit and the consequent need to control an increase in chloride concentration to minimal levels, it was considered important to have some contingency that would allow control of contaminant impact in the event that unacceptable impact might otherwise be deemed likely. For example, if the stone drainage blanket in the primary leachate collection system were to clog and a leachate mound develops on the base of the clay liner during the contaminating lifespan, or if there is an unexpected drop in water levels in the Upper Granular Unit. The Sub-Liner Contingency Layer (SLCL) was installed to address this possibility. Its operation is discussed in detail by Rowe et al. (1996a, 1997a).

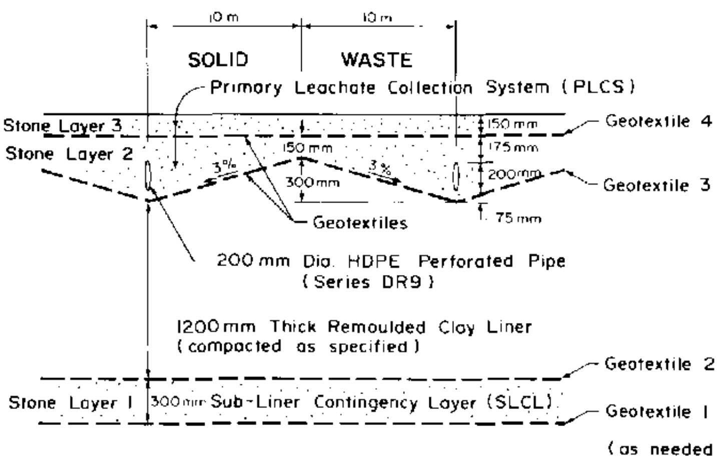

Fig. 11 Cross-section through the engineered barrier system for the Halton Landfill. (Modified from Rowe et al., 1996.)

The SLCL consists of a 0.3 m layer of clear stone with pipes connected to a manhole at the east and west of each cell. This layer is hydraulically isolated from the landfill by the compacted clay liner. It is intended that the sub-liner contingency layer below each cell will also be hydraulically isolated from that below the adjacent cells so that each can be independently monitored and controlled. Once a landfill cell is completed and sufficient waste has been placed to maintain basal stability, the SLCL is saturated by pumping water into a distribution manhole. Air is allowed to escape through air vents at strategic locations. Once the SLCL is saturated and the head in the unit is sufficient to induce an inward hydraulic gradient across the clay liner, the water level will be left to adjust itself until a hydraulic equilibrium is reached. This equilibrium is expected to involve a natural "hydraulic trap" with a small quantity of water flowing from the UGU into the SLCL and from there through the clay liner and into the Primary Leachate Collection System.

The Halton Landfill is subject to an extensive monitoring program and has been performing well since construction. As previously noted, the successful approval and operation of the Halton Waste Management Facility has resulted in the approval of a number of other hydraulic containment designs (e.g. Grimsby and Essex-Windsor Landfills, Canada) without the need for a public hearing.

GEOSYNTHETICS IN BARRIER SYSTEMS

The past decade has seen a substantial increase in the understanding of the role of geosynthetics in landfill barrier design, advances in design methods and the development of sound construction quality control and assurance procedures. This, combined with a regulatory preference for geomembranes in standard designs (e.g. U.S. EPA Subtitle D, German Standards, etc.) has resulted in geomembranes (usually High Density Polyethylene) now being used extensively as part of the liner system for landfills. Typically, the geomembrane is used in conjunction with either a compacted clay liner or geosynthetic clay liner to minimize leakage through any holes/defects in the geomembrane. Geotextiles are used extensively as filters and separators (e.g. see Fig. 11) and to provide protection to geomembranes. Geonets may be used to provide leachate drainage, especially on steep side slopes, and to provide protection to geomembranes.

Geosynthetics are a very major part of the future for landfill barrier systems. Key issues are discussed by Rowe (1998) and are not repeated here.

LEACHATE COLLECTION SYSTEMS

The percolation of rainwater through a landfill cover and subsequently through the waste results in the generation of leachate. In order to minimize detrimental impact due to the escape of leachate, most modern landfills have a leachate collection system that is intended to (a) collect most, if not all, of the leachate generated by the landfill and (b) minimize the buildup of leachate in the waste which, if allowed to occur, would increase the driving force for the advective movement of contaminants from the landfill out into the groundwater or surface water. Leachate typically flows down through the waste and into a granular layer (sand, gravel or crushed stone). It is usually intended that it will then flow laterally through the void space between the solid particles in the granular media to plastic (usually High Density Polyethylene, HDPE) collector pipes. These pipes are an essential component of the collection system and are heavily perforated to allow leachate entry. The pipes conduct the leachate to pumps which are used to remove the leachate from the landfill for treatment.

Leachate contains nutrients which will encourage bacterial growth within the waste, in geotextile filters, in granular drainage layers and around the perforations in the leachate collection pipes. Clogging of the leachate collection system involves the filling of the void space between solid particles (e.g. crushed stone) as a result of a combination of biological, chemical and physical events. There is a growing body of evidence indicating that a major component in the clogging process is microbiological (Brune et al., 1991; Cullimore, 1993). The reduction in void space caused by biofilm growth (Brune et al., 1991; Vandevivere & Baveye, 1992; Rowe et al., 1997b,c) results in a concurrent reduction in the hydraulic conductivity of these drainage systems and hence a reduction in their capacity to laterally transmit leachate. This results in the buildup of a leachate mound within the landfill and can subsequently result in impact on surface water by leachate seepage from the sideslopes of the landfill as well as increased contaminant migration through the barrier system and into the groundwater. Examples of clogging of leachate collection systems can be found in a number of existing landfills including Toronto's large Brock West Landfill where a 20 m high leachate mound built up during the first 11 years of operation and the Keele Valley Landfill where the void space has been significantly reduced by clogging after only a few years operation (Rowe et al., 1995a; Fleming et al., 1997).

The clog material filling the void space between solid particles consists of both biotic and abiotic components. The biotic components are formed by the biofilms which may be differentiated into cellular (viable) and extracellular polymeric substances (EPS). This EPS retains water and accumulates various recalcitrant materials. Intermixed or stratified with the biotic fraction are the abiotic components which may be amorphous or crystalline in form and may, or may not, be enmeshed into the EPS.

A limited amount of research has been conducted into the mechanisms affecting the clogging of primary leachate collection systems. Notably, Brune et al. (1991) performed a field investigation of a number of landfills in Germany where significant clogging had been observed. They also conducted a laboratory study in which they performed column tests using various drainage materials over a period of 16 months. Rowe et al. (1997b,c) have completed five years of a seven year study which has involved field exhumation at the Keele Valley Landfill, large scale mesocosm experiments (full scale real time collection system components), anaerobic column studies and development of a clogging model (Rowe et al., 1997d). The following subsections will focus on several field case histories (Rowe et al., 1994).

<u>German Experience as Reported by Brune et al. (1991)</u>

Brune et al. (1991) reported the findings from a number of field investigations directed at understanding the failure of leachate collection systems due to clogging. In the course of their survey of 29 German landfills with collection systems, Brune et al. reported evidence of encrustation material in more than half of the cases investigated. Impairment of the drainage system varied from moderate deposits on the pipe bottom to extensive encrustation of the drainage layer in the vicinity of the pipe and in the whole drainage layer (Brune et al., 1991). At seven of the sites, exhumation of the collection systems was performed and it was found that layers of waste above the drainage systems had become "consolidated" and relatively impermeable. This indicates that clogging is not restricted to the drainage layers but can also occur in the waste near the bottom of the landfill.

Mechanical damage to pipes was reported in one third of the sites surveyed. It appears that the pipes most prone to damage were stoneware pipes. No specific reference was made to any other type of pipe.

At the Altwarmbuchen Landfill the rate of filling had been very rapid (about 10-20 m/a) and there was reported to have been an intensive acetogenic phase of decomposition. The E_h of between -150 and -100 mV and presence of sulphide in all the drains, together with gas analysis, indicated anaerobic conditions prevailed. Newer portions of the landfill had acidic leachate (e.g. pH=5.9, COD =51,000 mg/L, BOD_5=23,300 mg/L, BOD_5/COD=0.46, conductivity 135 mS/cm, calcium 3,530 mg/L and iron 1,150 mg/L, temperature in the drain: 25-40°C) while older portions were neutral to slightly alkaline (pH=7-8, COD =10,000 mg/L, BOD_5=1,000 mg/L, BOD_5/COD=0.1). There was considerable inorganic content in the leachate. At this landfill, clogging was particularly intense despite the fact that the leachate pipes were flushed at least once per year. The deposits in the pipes included deposits in the bottom, on the sides and even reaching from one wall to the other. This landfill accepted sewage sludge and some of the clogging was reported to be directly related to this sludge. The landfill also accepted salt slag.

An extensive excavation of the bottom liner system of the Geldern Pont sanitary landfill in Germany revealed that the whole drainage layer was affected by heavy and extensive encrustation (precipitation). It was reported that large areas of the sandy gravel (80% gravel: 2-9 mm and 20% sand: 0-2 mm) drainage layer were clogged ("incrusted and consolidated") to between 1/3 and 2/3 of its thickness. The area affected amounted to between 30% and 80% of the area excavated. The permeability was reduced by several orders of magnitude down to values as low as 10^{-8} m/s.

In contrast to the two examples of significant clogging noted above, the Venneberg Landfill exhibited very little clogging in the collection pipes between the annual flushing events. This landfill had been slowly filled (about 2 m/a) and was in the stable methane phase of decomposition. It was characterized by relatively low strength leachate (pH=7, COD=1,000 mg/L, BOD_5=40 mg/L, BOD_5/COD<0.1, conductivity 10.5 mS/cm, calcium 132 mg/L and iron 28 mg/L, temperature in the drain: 14-20°C). The encrustation consisted of inorganic precipitate combined with the bacterial slime. The encrusted drainage material was reported to have ranged from a thin layer of fine material on grains of gravel, to complete filling of the pores between the gravel grains forming a structure like that of concrete. Where the granular drainage layer was clogged, it was usually consolidated to a large mass which extended over an area several meters in diameter. A chemical analysis indicated that the five primary constituents of the encrustation material (with the average percentages of total dry mass given in brackets) were calcium (21%), carbonate (34%), silica (16%), iron (8%) and sulphur, likely as sulphide, (3%). Thus these five components represent about 80% of the total dry mass. Organic carbon only constituted about 3% of the total mass.

The German investigators concluded, inter alia:

- The main components of encrustation material were the cations of calcium and iron combined with carbonate and sulphide.
- The highest concentrations of organic and inorganic substances in the leachate and the greatest annual amount of drain encrustation were associated with the landfill that was most rapidly filled (10-20 m/a).
- Once a landfill has reached the stable methane phase with its lightly loaded leachate, there is very little encrustation.
- "Excavation of a sewage sludge deposit revealed that over large areas the drainage system had become more or less impermeable due to massive encrustations".
- "Limestone gravel is absolutely unsuitable as a drainage material. It decomposes under the milieu conditions prevalent on the bottom of a sanitary landfill."

It is not clear how the German investigators arrived at the last conclusion since there is no real evidence given in the paper to substantiate such a conclusion.

Brock West Landfill

The Brock West Landfill is located in southern Ontario in the Town of Pickering and is operated by the Municipality of Metropolitan Toronto. It became operational in June 1975 and is currently at its maximum approved footprint of 64.4 ha (Dames & Moore Canada, 1992). It is estimated that there is about 17 Mt of waste that has been landfilled including approximately 2 Mt of sewage sludge (Bleiker, 1992). The highest refuse contours are reported to be about 45 m above the liner (ibid). The temperature in the refuse at about 15 to 20 m depth ranges from about 20°C to about 60°C (ibid). The leachate temperature has been measured (Golder Associates Ltd., September 1994, pers. comm.) to be 27°C to 32°C. It appears likely that these high temperatures are related to the disposal of significant quantities of sewage sludge. For example, Bleiker (1992) reports that the temperature in the sludge was 5.5°C higher than the temperature of the refuse immediately adjacent to it. The elevated temperature continued for a distance of 1 m below the sludge layer.

The landfill is reported to have an approximately 100-150 mm thick sand bentonite liner and a primary leachate collection system which consists of leachate collection pipes at spacings typically ranging between 50 m (newer portions of the landfill - 1987-88) and 200 m (older portions of the landfill). The collection system is reported to involve a perforated pipe with pea gravel (5-10 mm) for the pipe bedding (M.M. Dillon Ltd. & Mr. Lou Ciardullo, pers. comm.).

During 1987, "water levels measured in gas control wells located over much of the eastern two thirds of the landfill proved that fluid was mounded in the refuse to as much as 20 m above the liner" (Dames & Moore, 1992). It is uncertain when the leachate mound developed. Dames & Moore (1992) reported that it was apparently not present in 1985-86. Leachate buildup was initially observed in September 1986 (i.e. 11 years after the commencement of landfilling) in one of the two landfill monitors just above the bentonite liner. In 1987, inspection of the gas collection wells indicated a high leachate level of nearly 20 m above the liner. In 1988, 1989, 1990 and 1991 the average elevation of the mound was about 25 m, 24 m, 23 m and 22.5 m above the liner, respectively. Plugging of the leachate header was observed in 1988. In 1990 a by-pass system was installed to divert leachate around a plugged section of the perimeter drain. The volume of leachate collected by the collection system has been small since 1988.

In 1991 only 7,300 m^3 was collected compared with an estimated input of 129,300 m^3. Thus only about 6% of the estimated fluid input is collected by the primary leachate drains. The fluid input in 1991 was reported to be less than in 1990 partly due to a decrease in precipitation but mostly because sewage sludge disposal was discontinued in May 1991.

Dames & Moore (1992) state that "the leachate collection system is not significantly reducing the leachate head in the refuse. Bacterial activity and organic plugging of the drains is highly suspect as the cause. This factor in combination with the lower than expected refuse permeability and a higher than expected bottom density are believed to have caused the development of the high leachate mound."

Very little monitoring was conducted in the critical mound buildup period between 1985 and 1987. However, it is evident that between 1979 and 1981 the leachate had a relatively high organic content (annual average values ranging between 2,326-6,540 mg/L BOD_5, 3,879-8,313 mg/L COD, BOD_5/COD ~0.6-0.79, pH~7). These values dropped in 1982 and 1983. Between 1981 and 1983, the pH was close to 7 (yearly average 7.1 to 7.4) and the iron content was relatively high (yearly averages between 73 and 119). After 1988, the leachate at Brock West had similar characteristics to the German leachate characterized at "Low Strength" (annual mean values 341-742 mg/L BOD_5, 1637-3158 mg/L COD, BOD_5/COD~0.2-0.3, pH 7.5-7.7 for 1988-1991). Thus, although the data is limited, it appears that the clogging of the Brock West leachate collection system may have occurred during the early acetogenic phase of decomposition. The leachate now appears to be in the stable methanogenic phase.

Keele Valley Landfill

The Keele Valley Landfill (KVL) became operational in 1983 and has recently been constructed to its maximum approved footprint of approximately 99 ha. It has an original estimated mass capacity in excess of 20 Mt. It is understood that no sewage sludge has been disposed of at Keele Valley. The highest refuse contours are about 60 m above the liner and the proposed average thickness when the landfill is about 30 m for the currently approved final contours (Golder Associates Ltd., June 1994 pers. comm.).

As previously discussed, the landfill has an approximately 1.2 m thick compacted clayey till liner with a hydraulic conductivity of less than 10^{-10} m/s (King et al., 1993). The landfill was constructed in four stages. In each stage, the liner is covered by about 0.3 m of sand which is intended to provide desiccation protection to the liner. In Stages 1 and 2, the primary leachate collection system consists of lateral French drains (50 mm, nominal diameter, stone) at spacings of about 65 m sloping towards the main collection pipes (spacing 200 m). In Stages 3 and 4, there is a 0.3 m thick continuous stone drainage blanket of 50 mm clear dolomitic limestone over an approximately 0.3 m thick sand protection layer and clayey liner. A woven geotextile is placed between the stone and the sand. The waste is placed directly on the stone drainage layer. The heads in the collection system and waste are being monitored and there is evidence of increasing levels of leachate mounding and temperature, especially in Stages 1 and 2 (Barone et al., 1997).

An exhumation in Stage 1 (which did not include the 0.3 m thick blanket of 50 mm stone) was performed to examine the performance of the liner after 4.25 years of landfilling (King et al., 1993). As discussed earlier, it showed that the diffusion profile started at the top of the sand blanket. This implies that there is negligible horizontal or vertical flow in the sand. The blanket appears to have experienced chemical/biological clogging with about the top 5 cm being black (iron sulphide) and the next 10 cm being reduced grey sand.

A field investigation has been carried out by Rowe et al. (1995a) to examine the leachate collection stone (in Stage 4) which had been in place for 1 year, 3 years and 4 years at the Keele Valley Landfill. The clogging was generally greater in areas where there were larger leachate flows or ponding, and in the older portions of the collection blanket. The stone still readily allowed the transmission of leachate; although there was about a three order of magnitude decrease in hydraulic conductivity in the lower saturated portion of the stone layer (Fleming et al., 1997). It was evident that some of the clogging of the stone could be directly attributed to the placing of the waste directly over the stone (i.e. without any filter between the waste and stone). This appeared to have allowed the movement of soil particles into the stone.

A chemical analysis of the encrustation material (mineral deposit) from the Keele Valley drainage layer indicated that it is approximately 19-23% calcium, 31-32% carbonate, 1-3% iron, 0.2-0.9% sulphur and 20-23% silica. Adjusting for the encrustation component excluding silica, calcium represents about 24% to 30% of the encrustation. Laboratory analysis of the encrustation formed in pipes from KVL leachate shows it to be about 30% calcium. The measured density of the encrustation ranged from 1.5 to 2.3 g/cm^3 (average 1.9 g/cm^3) (Rowe et al., 1995a; Fleming et al., 1997).

Implications

It is evident from the foregoing that clogging is a major factor that can influence the service life of leachate collection systems and consequently can have a significant effect on the performance of the landfill. More research is required to fully identify the service life of these systems and improve the design service life. However, it is already evident that (a) minimizing movement of particulate material into the granular material, (b) large pore size, and (c) regular cleaning of perforated pipes are all important factors in extending the functional life of these systems.

SLOPE STABILITY

Although most of the focus on landfill design relates to controlling contaminants, it would be a mistake to overlook the importance of geotechnical considerations such as slope stability. This is particularly true with the growing trend to both larger new landfills and vertical and lateral expansion of existing landfills.

Examples of failures in the development of expansion of existing landfills include the Maine and Rumpke failures. The Maine failure (Reynolds, 1991) involved lateral expansion of an old MSW landfill on a marine clay deposit consisting of a 3 m crust over 12-18 m of soft clay. The landslide involved the movement of about 500,000 m^3 of material and was caused by (a) excavation through the stiff crust of a marine clay at the toe of an existing (old) landfill to allow installation of a leachate trench and construction of a composite liner in the adjacent expansion area, (b) excessive vertical height of the old waste for the strength of the foundation, and (c) stockpiling of cover soils near the crest of the old landfill.

The Rumpke failure (Stark & Evans, 1997) involved the lateral expansion of an old unlined MSW landfill on a colluvium deposit. About 1,300,000 m^3 of waste over an area of 8 ha moved into a 4.4 ha excavation formed to allow construction of the composite liner system in the expansion area. The toe of the slope moved 250-300 m. The failure was caused by a combination of (a) excessive landfilling (by about 12 m) above approved elevation in the old landfill, (b) excavation at the toe of the old slope to install the new collection and liner system, and (c) low mobilized strength (post peak) of the colluvium.

Both these cases indicate the need for (a) a proper geotechnical investigation of the subsoil properties, (b) control of waste slopes, (c) appropriate control on waste elevations (and assessment of waste density) and (d) geotechnical analysis and design o the proposed excavation and expansion of existing landfills. Excavation at the toe of existing slopes was an important factor in both failures.

LANDFILL OPERATIONS

There has been a significant improvement for landfill operations from the days of the old "town dump" when waste was dumped and often open burned without regard to leachate collection, odor, dust, litter, birds etc. The Halton Waste Management Facility provides a good example of how the problems associated with these factors can be mitigated. As described by Rowe et al. (1996a), it has received considerable attention from both environmental professionals and the public and has become a showplace where proponents of new landfills can demonstrate that if a landfill is operated properly, environmental impact can be minimal. A number of important features of the operation of the Halton Waste Management Facility are summarized below.

The tipping face is kept as small as practical to minimize potential problems due to litter, birds, and odour etc. After a truck has unloaded, the waste is inspected and unacceptable waste (e.g. propane tanks, paint, oil and other types of hazardous waste) are removed. If acceptable, the waste is compacted with a heavy (30 000 kg) steel wheel compactor to obtain maximum density.

A combination of geosynthetic cover sheets and conventional soil cover are used for daily cover. The geosynthetic cover sheets are applied on the sloping side of the tipping face thereby minimizing the volume lost to soil cover. They are weighted down with used tires. A soil cover is used on the horizontal top of the tipping area and this forms the floor area for future lifts. Since this clayey soil has the potential to cause trafficability problems after rain, a thin layer of waste wood chips is used to provide a non-slip surface and to reduce the trafficking of clay on the tires of the trucks.

The potential for leachate seeps is minimized by sloping the daily cover near the edges of the landfill downward away from the edges to encourage perched leachate to run into the landfill rather than escape through the final cover. Also, the final cover incorporates a drainage layer below the compacted clay. Both 150 mm of gravel and a geosynthetic drainage blanket have been used. The objective of this layer is to intercept any leachate that does migrate to the edge and direct it to the leachate collection system. These procedures have been very successful in avoiding leachate seeps throuh the final cover.

Two of the most significant concerns to those living near a landfill are birds and litter. Birds have been kept to negligible levels at the Halton Landfill by the use of a Harris Hawk which is well suited to the landfill site conditions and serves to scare away most of the birds. In addition to the Harris Hawk, a starter pistol is occasionally used to shoot pyrotechnic shells into the sky followed by playing a taped distress call through loudspeakers to suggest that a bird has been injured. The combination of these two techniques has been so successful that birds have largely given up even attempting to come to the site.

There are three levels of litter fence. The first level involves portable litter fences 3.6 m high and 7.6 m long (on wheels) that are lined up together near the working face to catch litter. The portable fences can be steered from both ends and have a braking system so that they can be used on slopes. They are also very stable and can withstand strong winds. The second level consists of a semi-permanent 3.5 m high fabric fence. Finally, there is a chainlink fence around the perimeter of the landfill site. These fences are augmented by a regular litter patrol to pick up any litter on-site and off-site that might be attributed to the landfill.

Waste (broken) concrete and asphalt are used to construct the access roads using a geotextile separator between this coarse material and the clayey subgrade.

An important aspect of landfill operation involves ensuring the landfill is operated according to the approved operating plan, including ensuring slopes do not exceed specified values and that waste elevations do not exceed the approved elevation.

To minimize clogging of the collection system, leachate must be regularly collected (and not allowed to back up in the collection system or the waste). The leachate collection pipes must be cleaned regularly and inspected. A monitoring program (leachate, surface and groundwater, and gas) should be implemented, reported and reviewed.

Adherence to the issues listed above combined with good engineering design and construction of the liner and cover systems distinguish the "dumps" of the past from the modern engineered sanitary landfill. With continued vigilance, the modern landfill if properly designed, constructed, operated, closed and maintained for the contamination lifespan should provide no hazard to either this or future generations - unlike some of the old dumps of the past.

THE FUTURE

Modern engineered landfills typically have a barrier system that includes, as a minimum, a primary leachate collection system and either a thick clay deposit as a "natural liner", a compacted clay liner, or a composite (geomembrane over clay) liner. In the future, there is likely to be increasing concern for issues such as the service life of the leachate collection system and the geomembrane (Rowe et al., 1994; Rowe & Fraser, 1994, 1995). Likewise, there is growing concern for the ability to monitor contaminant escape due to a defect in single composite liners. These two issues can be well addressed by using a double lined system with a secondary leachate collection/leak detection layer. The Halton Landfill already discussed illustrates this trend. Here, the "Sub-Liner Contingency Layer" (see Figure 11) provides a backup in the event of an unexpected problem and can be used to control the impact on the environment in the event of a failure of the primary leachate collection system. A double composite system can serve the same purpose and these landfills have been used for hazardous waste disposal for some time. There is now a move towards using double lined systems for municipal solid waste, especially for large landfills (e.g. see Rowe et al., 1995b; MoEE, 1997). This trend is likely to grow in the future.

The service life of the components of an engineered landfill is particularly important for large landfills with low permeability covers since these landfills have a large contaminating lifespan and are likely to require maintenance for hundreds of years if future impacts are to be avoided (Rowe, 1991; Rowe et al., 1995b). This concern is leading to the desire to stabilize the waste prior to placement or to accelerate degradation and stabilization after it has been placed. Waste stabilization prior to placement is now being mandated in some European countries. For example, in Germany waste material to be landfilled must have an organic carbon content of less than 5% by weight (TA Siedlungsabfall, 1993). This, in fact, means that all domestic waste must be treated in an incinerator. France is following the German lead and after 2002, no raw municipal solid waste can be landfilled. One solution being considered by the French is to use landfills to store blocks of chemically stabilized and solidified waste (Gourc, 1994).

In the U.S., the trend in landfill design has been to have a low permeability cover over raw waste. This trend is likely to increase since the U.S. Supreme Court has imposed limits on the amount of landfill gas a landfill can release to the atmosphere per year. In effect, this will mean the need for a low permeability cover and gas collection system for all but the smallest landfills. However, as noted above, these covers will extend the contaminating lifespan of the landfills unless other action is taken to stabilize the waste. This is leading to increased interest in the use of either moisture addition or leachate recirculation in landfills (EPA, 1995) and there is likely to be much greater use of landfills as bioreactors in the future.

A final challenge for the modern landfills in the future is their closure and afteruse. There is a need to provide an aesthetically pleasing closed landfill that contributes to the local community (e.g. as a park, golf course, wildlife habitat) while not negatively impacting on the long term performance, care and maintenance of the facility. MacKay (1996) describes three examples of different landfill closure designs. In the closed Key Largo Landfill (Florida, USA) has been developed as a nature reserve by removal of non-native species of plants and replanting with native species. Particular attention was paid to promoting the growth of shrubs whose leaves are food for an endangered butterfly native to the area. The Sanlando Landfill was developed for afteruse as part of a softball complex for the local community. The Dyer Boulevard Landfill was developed as a multi-purpose recreational park. A final example was the conversion of a landfill in northern New Jersey into a piece of art (Pinyan, 1987). In all such cases, particular care is required in the design of the final cover, and the gas and leachate collection system, where applicable, to meet the needs of the afteruse, provide long term performance and minimize potential dangers or damage to the public or the landfill itself.

CONCLUDING COMMENTS

The advances in landfill engineering have been outlined based on a number of field cases that cover the spectrum from the uncontrolled dumps of the past (e.g. Love Canal) to state-of-the-art facilities (Halton Landfill) which combine a highly engineered facility and state-of-the-art operations. Cases have been used to demonstrate that high quality, low permeability compacted clay ($k<10^{-10}$ m/s) liners have been constructed and perform well in the field. When operating under design conditions, the primary contaminant transport mechanism for modern barrier systems is diffusion. Diffusion of both inorganic and organic contaminants through clay deposits and compacted clay liners has been illustrated for a number of landfills. Although not discussed in detail in this paper, diffusion of organic contaminants can also be significant for geomembrane liners (Rowe et al., 1995b,c; 1996b). Contaminant transport through composite liner systems (geomembrane over clay) is discussed in detail by Rowe (1998).

Leachate collection systems represent a key component of modern landfills. As illustrated with respect to a number of cases, these systems can experience significant clogging. The amount of clogging can be reduced and the service life of these systems can be extended by appropriate design. However, the available cases suggest that careful consideration should be given to the service life of these systems in the design of modern landfills if they are to provide adequate protection of surface and groundwater into the future.

The trend to larger new landfills and expansion of existing landfills can create potential stability problems and conventional geotechnical issues should not be overlooked in the design of new or expanded landfills as illustrated by several cases cited herein.

Finally, no matter how good the barrier system and cover, the operation of the landfill will dictate whether the landfill does or does not have a significant environmental impact. Modern landfills have well developed operation plans and monitoring to ensure that those plans are followed. Those landfills which combine good design (considering all key failure mechanisms), construction, operation and long term maintenance may be expected to provide environmental protection both today and for hundreds of years into the future.

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