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Effect of interface transmissivity and hydraulic conductivity on contaminant migration through composite liners with wrinkles or failed seams

R. Kerry Rowe¹* and A.Y. AbdelRazek²

Abstract:
The leakage and the peak chloride concentration in an aquifer for a single composite liner facility is modelled for: (i) a hole in a geomembrane wrinkle, and (ii) a failed seam. A method using a closed form solution to calculate leakage together with a 1½D semi-analytic contaminant transport model is proposed, and the results compared with those obtained from 2D finite element modelling (FEM). Leakage is shown to be highly dependent on the interaction between the interface transmissivity (θ) and hydraulic conductivity beneath the wrinkle (k_b). Similar leakages arising from different combinations of transmissivity and hydraulic conductivity are shown to have significantly different impacts on an underlying aquifer. Contaminant transport modelling is needed to assess this effect for the likely range of uncertainty regarding interface transmissivity (θ) and hydraulic conductivity. The 2D FEM model is conceptually more comprehensive, however using conventional software only a very limited size of problem could be accurately modeled given the greatly different scales that must be modelled. In contrast, the semi-analytic 1½D approach readily allowed consideration of the highly variable scales, and gave results at the downgradient edge sufficiently similar to the 2D.

Keywords: Geosynthetics, Contaminant migration, Landfill, Interface transmissivity, Composite liner

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1. Introduction
Geomembranes (GMBs) are often used in conjunction with a geosynthetic clay liner (GCL), a compacted clay liner (CCL), or both to form a composite liner for a wide range of hydraulic containment applications (Shackelford et al. 2009; Rowe 1998, 2004, 2006, 2012a; Bouazza 2002). When a GMB is exposed to heating due to solar exposure it experiences wrinkling (e.g., Giroud and Peggs 1990; Giroud and Morel 1992; Pelte et al. 1994; Giroud 1995; Rowe 1998; Koerner et al. 1999; Touze-Foltz et al. 2001; Chappel et al. 2012a,b; Rowe et al. 2012). Wrinkles can vary in length; smaller wrinkles can also interconnect with larger ones forming a continuous network of wrinkles. If a hole develops in, or adjacent, to a wrinkle in a GMB, it then becomes a major conduit for leakage through a composite liner (e.g., Brachman et al. 2007; Rowe 2012a). GMB panels are welded together, often using the dual wedge technique. These welds, or the heat affected zone immediately adjacent to the weld, represent a particularly vulnerable point due to a combination of magnified tensile strains at this location (relative to the sheet away from the weld; e.g., Giroud 2005; Peggs et al. 2014; Kavazanjian et al. 2017) and due to accelerated aging of some welds relative to the sheet (Rowe and Shoaib 2017). Since there is likely to be more than 1500 m/ha of welds between panels, a failure in seams can be an important source of long-term leakage through composite liners.

In a composite liner with a GMB over a GCL, the hydraulic conductivity \( (k) \) of GCL, and interface transmissivity \( (\theta) \) of GCL/GMB are the two key hydraulic parameters required for calculating leakage through the liner system and hence the contaminant impact on an underlying aquifer (e.g., Rowe 2012a). The hydraulic conductivity \( (k) \) is affected by confining stress, hydration before contact with leachate (Acikel et al. 2018; Rowe 2018a), bentonite type and particle size, and the ionic strength of permeant (among others: Rad et al. 1994; Petrov et al. 1997; Petrov and Rowe 1997; Rowe 1998; Shackelford et al. 2000,2010; Benson et al. 2008; Scalia et al. 2013; Bradshaw and Benson 2013; Bouazza and Gates 2014; Liu et al. 2015; Tian et al. 2016; Setz et al. 2017; Rowe 2018a,b; Chen et al. 2018; Gates et al. 2019).
Similarly, interface transmissivity ($\theta$) may depend on normal stress, bentonite type and gradation, geotextile (GTX) in contact with the GMB, GMB texture, and permeant chemistry (Harpur et al. 1993; Barroso et al. 2008, 2010; Mendes et al. 2010; Bannour and Touze-Foltz 2010; AbdelRazek et al. 2016; AbdelRazek and Rowe 2018; AbdelRazek 2018).

Leakage through composite liners is associated with contaminant mass transport to the underlying soil and receptor aquifer (Rowe et al. 2004). The significance of wrinkles in terms of increasing leakage has been recognized for 20 years (e.g., Rowe 1998) and the significant extent of wrinkle development in a facility has been documented (Chappel et al. 2012a, b; Rowe et al. 2012; Rowe 2012a). There has been, however, paucity of data on interface transmissivity of composite liners when permeated by a fluid other than water until recent work (AbdelRazek and Rowe 2018; AbdelRazek 2018). Also, there has been no assessment, prior to this study, of the effectiveness of composite liners and the impact of GMB wrinkles with a hole or a possible seam failure on contaminant transport when impact is not controlled by diffusion.

The primary objective of this paper is to explore the effect of the hydraulic conductivity ($k$) of GCL and interface transmissivity ($\theta$) between GCL and GMB on leakage and contaminant transport from a hypothetical containment facility for two types of defect: (i) a hole in a 0.1 m-wide (after compression by the weight of the waste) GMB wrinkle, and (ii) a failed seam. A secondary objective is to assess how well a hand calculation of leakage using the Rowe (1998) equation combined with a new means of addressing leaking wrinkles of a failed seam using a 1½D semi-analytical contaminant transport model performed in estimating the contaminant concentration in an aquifer for the same problem as examined by performing a full 2D coupled finite element analysis of leakage and the corresponding contaminant transport to an aquifer. The findings of the analyses and the practical implications will be assessed in the context of the regulatory framework of Ontario Regulation 232 (MoE 1998).
2. Problem examined and model details

2.1 The base problem and parameters

Except as otherwise noted, consideration was given (Figure 1) to a composite liner comprised of a 1.5 mm-thick GMB over a GCL of thickness $H_L = 0.007$ m overlying an attenuation layer of thickness $H_A = 3.74$ m such that the minimum layer thickness ($D = 3.75$ m) above the aquifer level met the requirements of MoE (1998). The thickness of the GCL was assumed constant (i.e. at the received condition) and swelling upon hydration/thinning under stress was not accounted for. The containment facility to be considered was taken as 1000 m long in the direction of groundwater flow in a $H_b= 3.0$ m-thick aquifer. The pressure head at the top of the aquifer was taken to be $h_a = 3$ m. The leachate head on the GMB was taken as $h_w = 0.3$ m, giving a net head loss, $h_{ds}$, across the composite liner and attenuation layer of 1.05 m. The hydraulic conductivity of the attenuation layer was taken to be the maximum permitted by MoE (1998) of $1\times10^{-7}$ m/s while that of the aquifer was taken to be $1\times10^{-3}$ m/s.

Rowe et al. (2012) demonstrated that periodicity is a feature of wrinkle distributions. Thus, in this paper, the GMB was assumed to have periodically distributed 100 m-long (i.e., length $L_w = 100$m perpendicular to the direction of groundwater flow) wrinkles with a hole (referred to herein as a holed-wrinkle or simply a wrinkle with the fact it has a hole implied by context) at a spacing of 50 m in the direction of groundwater flow. This corresponded to 200 m of holed wrinkle/ha. To put this in context, Chappel et al. (2012a) reported that at a landfill just north of Toronto (44°23' N 79°43' W) on a sunny day in early June there were 440 wrinkles/ha with the longest interconnected wrinkle of 80 m/ha at 08:45am (ambient temperature 21°C) and 3000 wrinkles/ha with the longest interconnected wrinkle of 4200 m/ha at 11:10am (ambient temperature 25°C). The wrinkle length reached about 200 m/ha at about 9 am. An interconnected wrinkle length of 6600 m/ha was reached at 1:45pm (ambient temperature 26°C). The wrinkles were assumed to have an initial width of 0.2 m (Rowe et al. 2012; Chappel et al. 2012a,b) but...
to compress to a width $2b = 0.1$ m under the weight of the overlying waste (Brachman and Gudina 2008).

Only one GCL initial “representative” thickness (in this case 0.007 m) can be explicitly modelled and the method of analysis does not permit explicit consideration of a change GCL thickness to reflect swelling (especially below a wrinkle) or any reduction in voids ratio due to applied stress and chemical interaction when in direct contact with the GMB. However, the effect can be, and was, implicitly considered in the selection of the hydraulic conductivity values based on this representative thickness when deducing it from the measured permittivity in laboratory tests. Thus, while only one thickness can be explicitly considered the net effect of a change on permittivity due to a change in voids ratio (thickness) can be considered in the selection of the hydraulic conductivity to give the correct hydraulic resistance (permittivity). Specifically, two hydraulic conductivity values were used to simulate the GCL (Figure 1) under saturated conditions. For the “stress-free” zone beneath the wrinkle, a hydraulic conductivity ($k_b$) was assumed while a second hydraulic conductivity ($k_a$) was assumed where the GMB and GCL were in direct contact and there was a substantial vertical stress to compress the GCL and reduce its hydraulic conductivity. The interface transmissivity ($\theta$) between the GMB and GCL, and the GCL hydraulic conductivities ($k_b$ and $k_a$), are variables that will be discussed later.

A typical facility with 6.8 m-wide GMB rolls will have more than 1500 m/ha of dual wedge welded seam. Recent research (Rowe and Shoaib 2017, 2018) has shown that this is a critical location with respect to the service-life of geomembranes and that, provided the GMB has adequate protection from overlying drainage gravel (Rowe et al. 2013; Ewais et al. 2014; Abdelaal et al. 2014), GMB failure is likely to first occur at the welds. To explore the implication of seam failure, assuming no other holes, on leakage and contaminant impact on the underlying aquifer, the failure of 1500 m/ha of seam was modelled for the extreme situation where they all failed simultaneously at 0, 75 and 140 years after the
waste was placed. The failed seams were periodically distributed at a spacing of 6.8 m (typical roll with between welds) in the direction of groundwater flow and were each modelled as a crack 0.001 m wide perpendicular at the edge of each GMB roll (i.e., assuming the rolls were predominantly placed perpendicular to the direction of groundwater flow).

Dissolved salts form a major constituent of municipal solid waste (MSW) landfill leachates along with volatile fatty acids and low concentrations of heavy metals and volatile organic compounds (e.g., benzene, toluene, and dichloromethane; Rowe et al. 2004). Of the salt constituents, chloride is a conservative contaminant. Chloride’s significant attenuation mechanism is dilution, it doesn’t attenuate due to precipitation or sorption or biodegradation (Rowe et al. 2004). Thus, chloride contaminant was considered in this study, the chloride concentration \(c_o\) was assumed to vary linearly with the mass of waste per unit area from 1500 mg/l (for 150,000 t/ha) to 2500 mg/l (for 250,000 t/ha) as per Ontario Regulation 232 (MoE 1998, Table 1). The chloride concentration \(c_o\) was assumed to be at the maximum concentration at the start of the analysis (with negligible mass removal prior to reaching that maximum; a conservative assumption consistent with MoE 1998) and corresponded to a total chloride mass (mg) per unit (kg) mass of as compacted waste \(m_{TC} = 1800\) mg/kg as per MoE 1998, Table 1). It can be shown that with infiltration of water through the landfill cover \(q_o\) per unit area, and leachate collection that maintains a constant head in the landfill, the chloride will decrease with time (due to dilution) according to a first order relationship (Rowe 1991; Rowe et al 2004):

\[
c(t) = c_o \cdot e^{-\lambda t}
\]

[1]

where, \(c(t)\) is the concentration at time \(t\), \(c_o\) is the initial concentration in the source,

\[
\lambda = \frac{q_o \cdot c_o \cdot A_o}{m_{TC}}
\]

is the first order decay constant corresponding to dilution.

### 2.2 2D finite element [FE] analysis
Given the nature of the problem being examined, it would seem appropriate to use either 2D finite element software or 1½ D finite layer software in common industrial use to assess the potential impact of the wrinkles and failed seams on contaminant impact at the downgradient edge and monitoring points outside the landfill. As will become evident each approach has its strengths and weaknesses.

In the first instance, the finite element (FE) software (GeoStudio 2018: SEEP/W) was used to estimate the steady state leakage \( Q \) through a composite liner and (GeoStudio 2018: CTRAN/W) was used to assess the migration of chloride to the underlying aquifer. However, it soon became evident that finite element software running on even powerful PCs has difficulty modelling the range of scales that exist in the problem to be analyzed. The scales range from the extremely thin layer needed for the GMB-GCL interface (in reality probably < 0.0001m), the GCL (~0.007 m), the crack in a seam (0.001 mm), the 0.1m width of the wrinkle with a hole (“holed-wrinkle”) in the GMB while modelling the full scale of the attenuation layers 3.743m thickness), aquifer (3 m thickness), and length of landfill (1000m). This challenge was exacerbated by the many orders of magnitude difference in the saturated hydraulic conductivities of the various units (from \( 6 \times 10^{-12} \) to \( 1 \times 10^{-3} \) m/s) and coefficients of hydrodynamic dispersion.

Given the challenges noted above, to get an even reasonable representation with the available computational capacity, it was necessary to limit the modelling to a single 100 m long cell in the containment facility. The intact areas of the GMB layer were modelled only as a boundary condition, a non-porous boundary (zero mass and advective flux; Figure 2) was used to simulate the effect of intact GMB where neither flow nor diffusion is permitted. The two 100 m-long holed GMB wrinkles with width \( 2b \) of 0.1m were modelled as a gap of 0.1m width and appropriate flow \( h_w \) and mass transport \( c_0 \) boundaries were assigned (Figure 2). To avoid modeling problems, it was necessary to increase the
thickness of the transmissive layer modelled to a 0.25 mm layer and the hydraulic conductivity of the transmissive layer was adjusted to get the desired transmissivity for a layer of this thickness.

The FE model was extended 100 m downgradient of the cell (a buffer zone) to enable the assessment of chloride migration away from the facility and the chloride concentration was evaluated at hypothetical monitoring wells sampling the 3m thick aquifer and located: (i) at the edge of the landfill, (ii) 30 m (minimum buffer as per MoE 1998), and (iii) 100 m (preferred buffer; MoE 1998) downgradient of the cell. Since the problem to be considered had periodically distributed holed-wrinkles at a spacing of 50 m over the 100 m length of the landfill, the two wrinkles were located 25 m upgradient and downgradient of the center of the cell (Figure 2).

The seam crack (width $2b = 0.001$ m) was only modelled using the FE software for seepage analysis since symmetry could be invoked for the seepage but it could not be invoked for transport modelling and even at this scale it was not practical to model contaminant transport with cracks 0.001m wide at a 6.8 m spacing over the 200 m needed for model the last cell and the 100m distance to the site boundary.

For the FE seepage and contaminant transport calculations, the finite element mesh was graded from finest (0.005 m in the refinement zones of GCL and interface layers) near the holed-wrinkle and getting coarser (up to 0.5 m) away from the wrinkle/seam crack. The mesh was proportioned to satisfy Peclet and Courant criterion. The mesh refinement in the attenuation layer and aquifer was mainly governed by the Peclet number resulting in element size of 0.4 m. The Courant number criteria was satisfied by adopting 1000-time steps for the contaminant analysis. The FE mesh used for the seepage and contaminant transport model had a total of 29 300 triangular and quadrilateral elements with 28 900 nodes. The effect of mesh refinement was examined and will be discussed in the context of the comparisons with the Rowe (1998) equation and the semi-analytical POLLUTE analyses.

1. **FE Flow and Contaminant transport model boundary conditions for full cell**
A specified pressure head, $h_w=0.3$ m, was applied to the GCL where there was a wrinkle/seam crack, and specified total heads were applied at the up-gradient and down-gradient edges of the aquifer such that horizontal Darcy flux ($v_{hout}$) through the aquifer fulfilled the continuity requirements and a horizontal Darcy flux 1 m/a (i.e., a horizontal gradient of 0.001) prior to the landfill leaking. A zero-flux seepage boundary condition was used to model the intact surface of the GMB, and it was also applied on the model geometric boundaries not otherwise specified.

Since a GMB is essentially impermeable to both advective and diffusive transport of chloride where it is intact (Rowe 2005, 2012b), the GMB was modelled as a zero mass and advective flux boundary. A (chloride) concentration defined boundary condition given by Equation 1 (Rowe 1991; Rowe et al. 2004) was applied at the wrinkle location. Advective-diffusive-dispersive chloride migration through the underlying soil down to the aquifer was modelled for a range of GCL parameters $k_a$, $k_b$ and interface transmissivity $\theta$ and dispersivity values. Given the four orders of magnitude difference in the hydraulic conductivity of the aquifer compared to the overlying attenuation layer, the water flow in the aquifer is predominantly horizontal and controlled by the difference in head between the upgradient and downgradient edges of the landfill. A zero-mass flux boundary was assumed on all the boundaries except the wrinkle (initial concentration) and the down-gradient edge of the aquifer (free exit boundary, referred to as “exit review ($Q_d > 0$) boundary” in CTRAN/w literature; GeoStudio 2018) to account for advective-dispersive transport at the down-gradient edge of aquifers (Figure 2).

The porosity of GCL was taken to be 0.7 while that of the attenuation layer and aquifer was 0.3 (Rowe et al. 2004). Chloride diffusion coefficient of GCL was taken to be $5 \times 10^{-3}$ m$^2$/a while that in the attenuation layer and aquifer were each taken as $2 \times 10^{-2}$ m$^2$/a (Rowe et al. 2004). A base case value of longitudinal (i.e., vertical) dispersivity ($\alpha_L$) of 0.2 m was assumed for flow through the GCL and AL, whereas $\alpha_L$ (horizontal) of 3.0 m was used for the aquifer. Longitudinal dispersivity ($\alpha_L$) of aquifers and
the ratio of transverse to longitudinal dispersivity \( (\alpha_T/\alpha_L) \) are parameters of significant uncertainty (Rowe at al. 2004), and a sensitivity analysis was conducted to assess their effect on the contaminant concentration at the downgradient edge of the facility, and the two monitoring points (i.e., 30 m and 100 m) downgradient edge as will be discussed in Section 3.3.

ii. **FE Flow model boundary conditions and mesh properties for a single wrinkle/seam**

To assess leakage and the sensitivity of the FE flow calculations to mesh refinement, a much finer scale than the scale possible for flow and transport modeling of a full cell was adopted, thus a separate flow modelling (SEEP/W) was conducted for individual holed-wrinkles or failed/cracked seams. Assuming local symmetry of the flow at the wrinkle/failed seam, only half of the wrinkle/failed seam width was modelled, and the model only extended for 25 m from the wrinkle/failed seam centerline towards the downgradient edge of the aquifer. This enabled more mesh refinement at the interface layer and the area of GCL beneath the wrinkle half-width (b). The global element size for the mesh was 0.015 m, with reduced element size of 0.0075 m at the GCL beneath the wrinkle width, and element size of 0.00375 m at the interface layer up to 5 m away of the wrinkle/failed seam location and getting coarser away from the wrinkle/failed seam toward the lateral boundary (i.e., 25 m from the wrinkle centerline). The basic mesh had a total of 15 959 triangular and quadrilateral elements with 11 825 nodes (this increased on some refinement studies to be discusses later).

A specified pressure head, \( h_w = 0.3 \) m, was applied to the GCL where there was a wrinkle/seam crack, and specified total head \( (h_d = 3.0 \) m) was applied at the aquifer such that the head loss across the composite liner \( (h_d) \) is 1.05 m. A no flux seepage boundary condition was used to model the intact surface of the GMB and was also applied on the model geometric boundaries not otherwise specified.

2.3 **Analytical relationship for leakage below a wrinkle or long crack**
Rowe (1998) developed a closed form analytical equation to calculate the leakage ($Q$) through a GMB hole coincident with a wrinkle (Figure 1) which can also be used of a long crack. In its simplest form assuming wrinkles/cracks are far enough apart to not interact (the more general case with interaction is given by Rowe 1998), the Rowe (1998) equation can be written (Figure 1)) as:

$$Q = L_w \cdot [2b \cdot k_b + 2(k_a \cdot \theta \cdot D)^{0.5}] \frac{h_d}{(H_L + H_A)} \quad \text{(m}^3\text{/s)} \quad [2a]$$

where $Q$ is the leakage ($\text{m}^3/\text{s}$); $L_w$ is the wrinkle length (m); $2b$ is the wrinkle width (m); and for a GCL of thickness $H_L$ and hydraulic conductivities $k_{La}$ where the GMB and GCL are in intimate contact and $k_{Lb}$ where unstressed below the wrinkle, and an attenuation layer of thickness, $H_{AL}$, and hydraulic conductivity, $k_{AL}$, the quantities $k_a$ and $k_b$ represent the harmonic mean

$$k_a = \left[\frac{(H_L + H_{AL})}{(H_L / k_{La} + H_{AL} / k_{AL})}\right] \quad \text{(m/s) where the GMB and GCL are in intimate contact} \quad [2b]$$

$$k_b = \left[\frac{(H_L + H_{AL})}{(H_L / k_{Lb} + H_{AL} / k_{AL})}\right] \quad \text{(m/s) for the unstressed GCL below the wrinkle} \quad [2c]$$

$\theta$ is the interface transmissivity between GMB and underlying GCL ($\text{m}^2/\text{s}$); $h_d$ is the head loss across the composite liner (m) ($h_d = h_w + H_L + H_A - h_a$); $h_w$ is the height of leachate on the GMB, $h_a$ is the potentiometric level above the top of the aquifer, and $H_L + H_A$ is the thickness over which the head loss occurs (m).

It can be shown analytically that the flow $Q$ will occur over a wetted distance $(2a)$ for each wrinkle (or cracked seam) as given by Rowe (1998):

$$a = b - \ln \left[\frac{(C)}{(h_w + C)}\right] / \alpha \quad \text{(m)} \quad [3a]$$

where

$$\alpha = \left[k_s / (H_L + H_A) / \theta\right]^{0.5} \quad \text{(m}^{-1}) \quad [3b]$$

$$k_a = \left[\frac{(H_L + H_A)}{(H_L / k_L + H_A / k_{AL})}\right] \quad \text{(m/s)} \quad [3c]$$

$$C = H_L + H_A - h_a \quad \text{(m)} \quad [3d]$$
It follows from Eqs. 2 and 3 that the average Darcy flux (Darcy velocity), $v_a$, below the wrinkle or crack is given by:

$$v_a = \frac{Q}{2a \cdot L_w} \quad \text{(m/s)} \quad [4]$$

2.4 Modified $1\frac{1}{2}$D finite layer contaminant transport modelling

POLLUTEv7 (Rowe and Booker 2005) was developed to model only vertical 1D advective-dispersive-diffusive transport through laterally continuous horizontal layers and horizontal advective transport in an aquifer layer (Rowe and Booker 1995; Rowe et al. 2004). POLLUTEv7 followed the continuity equations where advective-dispersive-diffusive flux only migrates vertically downward and can only exit horizontally by advection through aquifer layer, as such it is more than 1D but not a full 2D model and so it is referred to herein as a $1\frac{1}{2}$D modeled since this captures the 1D vertical transport and the 2D component of lateral flow in the aquifer boundary. This is appropriate for cases without a GMB or cases with a GMB where transport is dominated by diffusion of volatile organic compounds like benzene and dichloromethane through an intact GMB. However, when modelling a contaminant that does not readily diffuse through the GMB (e.g., chloride) but only leaks through a hole in a wrinkle of the GMB or a failed seam, then novel measures are needed to use POLLUTE to provide reasonable results for these cases. This modified $1\frac{1}{2}$D finite layer approach is based on the fact that for wrinkles or failed seams, leakage only extends over the wetted width $2a$ (Figure 1). This leakage at each, periodically spaced, holed wrinkle or failed seam is coupled with diffusive-dispersive contaminant transport through the underlying layers, and it is essential to model such migration to assess its impact of on the underlying aquifer using the actual flow beneath the wrinkles and not a value smeared over the entire landfill length. The approach is outlined below.

For with $N$ wrinkles with a hole or crack perpendicular to a landfill of length, $L_f$, in the direction of groundwater flow, leakage and advective-diffusive-dispersive transport is essentially only occurring
over the sum of the $N$ wetted distances since the intact geomembrane prevents any leakage or diffusion of inorganic ions (e.g., chloride) outside the wetted distance. Thus, the landfill length over which transport occurs, called the adjusted length, $L_{f_{\text{adjusted}}}$, in the direction of groundwater flow is given by;

$$L_{f_{\text{adjusted}}} = 2a \cdot N \text{ (m)}$$  \[5\]

This adjustment in length requires that a number of other parameters used in the modelling be adjusted to maintain the conservation of mass of contaminant, infiltration volume, volume of leachate collected, and to allow for the correct dilution in the pore water of the aquifer, and travel distance, diffusion into the attenuation layer away from the wetted area, and dispersion in the aquifer as outlined in Appendix A and described in more detail with examples in AbdelRazek (2018).

### 2.5 Simplified 1½D semi-analytical (SA) analysis

The simplified 1½D semi-analytical analysis combines the use of the Rowe (1998) Eqs. 2 and 3 to calculate the Darcy flux $v_a$ and the adjusted landfill length (Eq. 5) as key input parameters for the 1½D semi-analytical analysis (Rowe and Booker 1995; Rowe et al. 2004) as implemented in POLLUTE v7 (Rowe and Booker 2005) to calculate the impact and in particular the peak impact and time to peak impact in the underlying aquifer at the edge of a landfill for the problem defined in Section 2. This use of an analytic equation to calculate the leakage $Q$ and Darcy flux $v_a$ combined with the 1½D finite layer contaminant transport modelling is not prone to the numerical difficulties and limitation associated with 2D FE modelling, but it does involve some simplifications and approximations to translate a 2D analysis into a 1½D analysis. Thus, the results from the simplified analysis (denoted SA herein) were compared to the full 2D finite element model to examine the variation between the two approaches and assess how well the approaches can model leakage and contaminant migration through composite liners where diffusion through the GMB is negligible and is restricted to the wetted area at holed-wrinkles or cracked/failed seams.
2.6 Allowable impact in the aquifer

Based on Ontario Regulation 232 (MoE 1998), the allowable chloride concentration in the aquifer at the compliance point in the aquifer groundwater is given by:

\[ c_m = c_b + X (c_r - c_b) \]  \hspace{1cm} (mg/L) \hspace{1cm} [6]

where, \( c_m \) is the maximum allowable concentration for the contaminant (mg/L), \( c_b \) is the background concentration of the contaminant in the groundwater of the receptor aquifer (\( c_b = 0 \) assumed here), \( c_r \) is the aesthetic drinking water objective for the contaminant (\( c_r = 250 \) mg/L for chloride, MoE 1998), and \( X \) is 0.50 for a contaminant like chloride limited by aesthetic drinking water requirements. This gives a maximum allowable concentration \( c_m = 125 \) mg/l at the compliance point in the receptor aquifer for the problem examined in this paper.

3. Results and discussion

3.1 Comparison of flow analysis results

One objective of the study was to compare the estimated leakage (\( Q \)) obtained using the analytical equation [2] (Rowe 1998) with those computed from a 2D FE analysis (SEEP/W) for different hydraulic conductivity (\( k \)) and interface transmissivity (\( \theta \)) values. The initial calculations were performed with the same FE mesh that was used for the transport modelling of full cell (Section 2.2). Twelve cases were examined using both Eq. 2 and the FE analysis (Table 1). Both approaches provided similar leakage values for the failed seam cases (\( 2b=0.001 \)m), with the FE analysis giving a 1%~9.5% higher leakage values compared to Eq. 2 suggesting that both approaches (Eq. 2 and SEEP/W with the mesh pattern described in Section 2.2) provided comparable estimates of leakage. Both analyses showed that the interface transmissivity was a key parameter affecting the leakage. There was also excellent agreement between leakage calculated from Eq. 2 and the FE analysis beneath a holed-wrinkle (\( 2b=0.1 \) m) for the multicomponent GCL with a hydraulic conductivity below the wrinkle of \( 6 \times 10^{-12} \) m/s for a 2-order of
magnitude variation in interface transmissivity (Cases 10, 11 and 12). A larger difference was noted for the cases where the hydraulic conductivity below the wrinkle was $3 \times 10^{-11}$ m/s (15%, Case 7) and $4 \times 10^{-10}$ m/s (between 21% for Case 5 and 33% for Case 6) suggesting that mesh refinement of FE may be required for such cases to better capture the leakage ($Q$) beneath the wrinkle and converge with the analytical Eq. 2. Considering the case with worst agreement (Case 6), five different meshes were examined by adopting a global element size 0.15 m and adjusting the refinement ratio for GCL away from the wrinkle to 0.075 (element size of 0.01125 m) and that of the interface layer to 0.015 (element size of 0.00225 m). This mesh refinement reduced the FE calculated leakage from 53 lphd to 47 lphd, essentially halving the discrepancy with Eq. 2 from 33% to 17%. It was not practical to further refine the mesh for a problem of even this limited size, but the results suggest that the primary reason for the discrepancy between Eq. 2 and the FE leakage was due to mesh refinement of the FE analysis. It was not practical to use this more refined mesh in the contaminant transport analysis (due to limitations of computer memory) but these results show that to the extent that the FE analysis errs it is to overestimate the leakage and hence it errs on the conservative side with respect to leakage for the problems examined.

Since the FE flows exceeded those for Eq. 2, conservatively, the higher FE leakage value ($Q$) was adopted in both the FE (CTRAN/W) and $1\frac{1}{2}$D semi-analytical (SA) model (POLLUTEv7) contaminant analysis reported in the next section to allow a comparison of the contaminant transport modelling for the same leakage.

### 3.2 Coupled FE analysis and comparison with SA analysis

Variable hydraulic conductivity ($k$) and interface transmissivity ($\theta$) values were used to simulate different composite liner conditions, the cases also varied in waste loading ($m^3/ha$). An aquifer longitudinal dispersivity ($\alpha_L$) of 3.0 m was adopted as a base case and the concentrations at the downgradient edge of the facility were calculated (Table 2).
Case I represented a landfill liner with hydraulic conductivity $k_b = k_a = 3 \times 10^{-11}$ m/s and interface transmissivity $\theta = 1 \times 10^{-11}$ m$^2$/s (Table 2) and a small landfill with a mass loading of 100 000 tonnes/ha. This represents an optimistic case for a GCL in a landfill application since it neglects the effect of “stress-free” zone below the wrinkle on $k_b$ (which would generally lead to higher hydraulic conductivity for a conventional GCL). The interface transmissivity value corresponds to that obtained at 100 kPa after permeation with a leachate (with 0.14 M NaCl) but with no surfactant (Rowe and Abdelatty 2013). In this case the leakage calculated using FE (SEEP/W) was within 15% of that from Eq. 2 and the peak impact of 3.2 mg/L at 150 years (CTRAN/W) was very small. Using the higher flow from the FE model in the $1\frac{1}{2}$D semi-analytical (SA) model (POLLUTEv7), the SA model gave a small but (conservatively) higher impact of 5.8 mg/L at 170 years (Table 2).

Figure 3 shows the FE predicted variation of chloride concentration across the aquifer with distance from the upgradient to the downgradient edge of the landfill cell examined at the time of peak concentration for Cases I to VI (Table 2). The aquifer up-gradient boundary is located at ($x$=-50) and it extended 100 m to the downgradient edge of the landfill ($x$=50) and a further 100 m in the buffer zone (Figure 2). The wrinkles were centered at $x$=-25 and $x$=25. The leakage at and near a wrinkle location gave rise to a contaminant plume extending down to the aquifer giving rise to an increase in the contaminant concentration as the flow migrated from the upgradient ($x$=-50) to downgradient edge of the landfill ($x$=50). The contaminant plume from the first wrinkle caused an increased contaminant concentration starting at about at $x$=-30 (i.e., up to 5 m up-gradient of the wrinkle location). There was a small decrease in concentration in the aquifer as the contaminant migrated downgradient of a wrinkle, in part due to dispersion in the aquifer and in part due to back diffusion into the overlying attenuation layer away from the plume arising from the first wrinkle. There was a second increase in concentration as the flow in the aquifer approached the second wrinkle and another small decrease in contamination as the
contaminant migrated downgradient of a wrinkle. This pattern was reflected in all cases shown in Figure 3 although in case with significantly higher leakage the migration upgradient of the wrinkle was notably larger.

Case Ia represents the same landfill as Case I (with mass loading of 100 000 t/ha and where the leachate was assumed to have no significant surfactant) except that in Case Ia the interface transmissivity was increased to $4 \times 10^{-11}$ m$^2$/s to model the effect of the presence of surfactant in the MSW leachates for a reasonable stress ($\geq 100$ kPa). Exploring this case enabled an assessment of the effect of interface transmissivity on leakage ($Q$) values and peak contaminant concentration ($c_p$). Here, a 4-fold increase in interface transmissivity led to a 1.6-fold increase in leakage ($Q$) (Table 2) from 15 lphd (FE) to 24 lphd. Similarly, using Eq. 2, the calculated leakage increased 1.6-fold from 13 lphd to 21 lphd. The 1.6-fold increase in flow was associated with a 1.15-fold increase in contaminant concentration (i.e., from 3.2 mg/l to 3.7 mg/l (FE) and 5.8 mg/l to 6.7 mg/l ($1/2$D-SA)).

Case Ib simulated a condition where the hydraulic conductivity of GCL beneath the wrinkle was increased to $k_b = 4\times 10^{-10}$ m/s in recognition of the “stress-free” zone beneath the wrinkle (i.e., allowing for cation exchange under negligible confining stress) while keeping the same $k_a = 3 \times 10^{-11}$ m/s for the area in contact with GMB as Case I (Table 2). The interface transmissivity was maintained at $\theta = 1 \times 10^{-11}$ m$^2$/s to simulate a leachate with no (or very low) surfactant concentration. Compared to Case I, a 13.3-fold increase in hydraulic conductivity beneath the wrinkle ($k_b$) led to a 3.5-fold increase in leakage ($Q$) for FE analysis (i.e., from 15 lphd to 53 lphd FE) and 3.1-fold increase using Eq. 2 (i.e., from 13 lphd to 40 lphd). The associated contaminant concentration, however, increased 11-fold in the FE analysis from 3.2 mg/l (Case I) to 36 mg/l (Case Ib) and 8.6-fold in SA-analysis from 5.8 mg/l to 50 mg/l (Table 2).

Cases Ia and Ib highlight the effect of leakage increase either due to higher interface transmissivity (Case Ia) or increased hydraulic conductivity (Case Ib) on the contaminant concentration in the
underlying aquifer. A 4-fold increase in interface transmissivity resulted in 1.6-fold increase in leakage ($Q$) but only a 1.15-fold increase in contaminant concentration ($c_p$) for FE and SA analyses; since the extra flow was spread out over a larger area and, in the FE analysis, flowed at different rates through the GCL and AL at different locations relative to the center of the wrinkle allowing for better attenuation of the impact on the aquifer. Whereas a 13.3-fold increase in the hydraulic conductivity directly beneath the wrinkle ($k_b$) led to 3.5 (FE) ~3.1 (SA) -fold increase in leakage ($Q$) and an even greater 11(FE) ~8.6 (SA)-fold increase in peak contaminant concentration in the aquifer. The magnification of the effect of increased leakage directly below the wrinkle for Case Ib (with both methods of analysis) was due to a more focused mass of contaminant flowing through the liner directly beneath the wrinkle. Thus, the magnitude of the leakage, in and of itself, is not enough to assess the effect of containment facility on the underlying aquifer; much depends on how the leakage is distributed. A contaminant impact assessment is needed to truly assess the effect of magnitude, location and intensity of the increase in leakage on the underlying aquifer.

To further explore the effect of GCL hydraulic conductivity beneath the wrinkle ($k_b$) and the interface transmissivity ($\theta$), Case II was modelled where the effect of both the “stress-free” zone beneath the wrinkle, as in Case Ib, and the effect of surfactants in the MSW leachate with a reasonable stress level ($\geq 100$ kPa), as in Case Ia, were considered. If there was a direct superposition of effects (i.e., no symbiotic or antagonistic interactions), compared to Case I, Case II would be expected to give a 4.5~5-fold increase in leakage and 9.7~12-fold increase in contaminant concentration. However, relative to Case I, Case II exhibited only an about 4-fold increase in leakage to 62 lphd (FE) and 47 lphd (SA). However, the peak FE impact of 50 mg/L at 71 years was 15.5-fold larger than in Case I (but still well below the allowable 125 mg/L). The $1\frac{1}{2}$D semi-analytical (SA) model gave a (conservatively) higher impact of 56 mg/L, which represented a 10-fold increase relative to Case I, at 82 years (also still well
below the allowable 125 mg/L; Table 2). The leakage calculated using FE was 31% higher than that from Eq. 2 but the impact calculated using the higher flow in the FE analysis was 11% less than that calculated in the SA analysis – with the latter being conservative. The reason for the difference in flow is largely due to the challenges of getting a sufficiently refined FE mesh in the very thin but critical transmissive zone and GCL as discussed in Section 3.1.

Figure 4 compares the variation of chloride concentration with time as calculated at the down-gradient edge of the landfill for 1½D semi-analytical (SA) and 2D finite element (FE) analyses. SA analysis gave a higher peak concentration after longer time than the FE for all the cases (Table 2). To the extent that the SA-analysis erred it appears to have been on the conservative side. Conversely, to the extent that the FE erred due to mesh refinement it did so on the unconservative side for contaminant impact. However, there was reasonable agreement between both approaches in all cases.

Keeping the same hydraulic parameters as Case II but increasing the mass of waste per unit area to 120 000 tonnes/ha (Case IV) and 140 000 t/ha (Case V) resulted in an increase in the peak impact from 50-56 mg/L (FE-SA) for 100 000 t/ha to 55- 62 mg/L (FE-SA) for 120 000 tonnes/ha to 58- 65 mg/L (FE-SA) for 140 000 t/ha (Table 2). In each case the 1½D semi-analytical (SA) gave a little higher impact at a slightly later time than the FE analysis. Given that the flows used in the 1½D semi-analytical analysis FE and SA transport analyses were the same, it may be inferred that the FE analysis underestimated impact and/or the 1½D SA analysis adopted somewhat overestimated the impact. However, the differences are considered small enough that they would not affect engineering decisions in any of these cases (all cases meet MoE 1998 requirements for the cases examined).

The variation of contaminant concentration ($c_p$) over the entire aquifer length was plotted for Cases II, IV and V (Figure 3). The three cases exhibited the same pattern of distribution along the aquifer (i.e., distance $x$). Cases IV (waste mass of 120 000 t/ha) had a chloride concentration of 18.5 mg/L at $x=$-
25m), the concentration was higher towards the downgradient edge of the landfill with $c_p=55.5$ mg/l ($x=30$ m; 20 m upgradient of the downgradient landfill edge) and $c_p=54.4$ mg/L at the downgradient landfill edge ($x=50$) for $t_p=73$ years. Similarly, Case V gave $c_p=59.3$ mg/l (at $x=30$) and $c_p=58$ mg/L at $t_p=76$ years for the downgradient landfill edge ($x=50$).

Based on the calculated variation in chloride concentration with time at the downgradient landfill edge ($x=50$; Figure 5), the SA analysis was conservative relative to the FE analysis for Cases II, IV and V (Table 2) with a longer time to reach the peak concentration. The chloride concentration based on SA analysis was 56 mg/L (Case II), 62 mg/L (Case IV) and 65 mg/L (Case V) and time of peak impact occurrence was 82 years, 85 years and 90 years for Cases II, IV and V respectively (Table 2; Figure 5).

Case III represented the flow parameters discussed in the previous section with respect to effect of mesh refinement. The leakage used in the FE transport analysis are 33% higher than obtained from Eq. 2 and used in the 1½D SA analysis. With mesh refinement it was possible to reduce the leakage from 53 to 47 lphd and the discrepancy with Eq. 2 from 33% to 17%. Unfortunately, it was not possible to run the FE transport analyses with similarly refined mesh – but some impact on the calculated peak concentration can be expected. For the mesh that was analyzed, the peak impact at the edge of the landfill cell was 43 mg/L (FE) and 58 mg/L (SA) for 120 000 t/ha. Again, the 1½D semi-analytical (SA) analysis gave a little higher impact at a slightly later time than the FE analysis. Compared to Case IV, the 4-fold lower transmissivity of Case III gave a 17% reduction in leakage and the peak impact reduced from 55- 62 mg/L (FE-SA Case IV) to 43-58 mg/L (FE-SA Case III). The peak impact decreased by 21% as the flow decreased by 17% for Case III compared to Case IV. This indicated that advection governed the contaminant transport such that when the leakage was decreased, due to lower interface transmissivity, the total mass of contaminant transported was reduced accordingly and hence the lower impact at the aquifer. The fact that advection governs for a leakage of 53-62 lphd is important. Had this leakage been
uniformly distributed it would correspond to an average Darcy flux $v_a = 6-7 \times 10^{-11}$ m/s or 0.002 m/a which is in the domain where one would expect diffusion to govern (Rowe et al. 2004) and the calculated impact assuming that flow was uniformly down at 0.002 m/a across the length $L_f$ (and neglecting the GMB as a diffusion barrier) would be about 5 mg/l at 100 years. However rather than being uniform, the flow is localized to the relative narrow wetted zone near the wrinkle and the impact is advection controlled and increased by an order of magnitude. Thus, it is important to model the localized nature of leakage through wrinkles and not smear the Darcy Flux across the full landfill width.

Given that the flows obtained from Eq. 2 are below those from the FE analysis (Tables 1 and 2) but were generally used in both the FE and SA transport analysis, the effect of flow on the $1\frac{1}{2}$D SA analysis was assessed for this Case IV by repeating it using the results from Eqs. 2 and 3 to define the flow and wetted distance used in the analysis. It was found that the peak impact calculated from the SA analysis reduced from 62 mg/L using the FE flow and wetted distance to 45 mg/L using the Eqs. 2 and 3 for flow and wetted distance. The reduction in peak impact was nearly 38% for a 32% drop in leakage (i.e., from 62 lphd in FE to 47 lphd for Eq. 2). This confirmed the earlier finding of the effect of lower interface transmissivity of Case III to Case IV (i.e. lower leakage) indicating that advection was controlling the contaminant transport and that lower leakage essentially led to lower mass of contaminant reaching the aquifer, hence lower peak impact. In addition knowing that with mesh refinement it was possible to reduce the FE leakage from 53 to 47 lphd and the discrepancy with Eq. 2 from 33% to 17%, it is suspected that if the refined mesh with the lower flow could have been used in the FE transport analysis then the results for the FE would have been very close to those from the $1\frac{1}{2}$D SA using the Eqs. 2 and 3 for flow and wetted distance.

Cases III and IV show the effect of a 4-fold increase in interface transmissivity (Table 2 and Figure 3) with Case III having a lower chloride concentration than Case IV over the entire aquifer length and a
peak chloride concentration ($c_p$) in the aquifer at the downgradient landfill edge ($x=50$) of 43 mg/L at 79 years for Case III compared to 55 mg/L after 73 years for Case IV.

The parameters used in Cases II-V are considered reasonable for a traditional needle punched GCL with a natural sodium bentonite. Case VI represented a liner system comprised a multicomponent GCL (Table 2) and GMB. Multicomponent GCL, are comprised of a conventional GCL with a geofilm applied to its carrier geotextile. The geofilm is either applied in molten state and allowed to solidify on the carrier geotextile (coated GCL) or glued to the carrier surface (laminated GCL). Measuring the hydraulic conductivity of multicomponent GCLs can often be very challenging, in fact a multicomponent GCL is essentially impervious if the geofilm is in perfect condition (i.e., no holes or perforations), and can be below the sensitivity of the measuring apparatus or inaccurate due to the sidewall leakage. However, for very thin films (especially when applied in the molten state), the film is imperfect and the hydraulic conductivity of the multicomponent GCL can be measured but is then governed by geofilm imperfections. Rowe and Hosney (2013) measured the hydraulic conductivity for coated GCLs under low stresses and relatively high gradient for pond applications and verified that the flow occurred through the bentonite surrounding the needle bundles stitching the geotextile carrier and cover layers together. They also verified, using blue dye, that the flow was not an artifact of side wall leakage. Hence, hydraulic conductivity values (typically $k_a=k_b \leq 6 \times 10^{-12}$ m/s off the roll) were considered to model a multicomponent GCLs with imperfect geofilm. The interface transmissivity between the geofilm and the rest of a multicomponent GCL has been reported to range from 1–4 $\times 10^{-11}$ m$^2$/s for a normal stress of 50 kPa (Bannour et al. 2013,2015), while the interface transmissivity between GMB and the geofilm of multicomponent GCL has been reported to be $1 \times 10^{-11}$ m$^2$/s at 150 kPa (AbdelRazek and Rowe 2016; AbdelRazek and Rowe 2018). There is, however, a paucity of published data regarding the interface transmissivity between GMB and geofilm when multicomponent GCL is
permeated with leachate having both a high ionic strength and containing a surfactant until recent work
(AbdelRazek and Rowe 2018). Thus, Case VI was modelled such that $k_b = k_a = 6 \times 10^{-12}$ m/s together
with an interface transmissivity of $7 \times 10^{-10}$ m$^2$/s estimated for a coated GCL in contact with a smooth
GMB and permeated at about 140 kPa based on interpolation (AbdelRazek 2018). For this case the
leakage is 29 lphd (based on both FE and Eq. 2) and a minimal peak impact in the aquifer of 6-8 mg/L at
130 years (Table 2).

The leakage for Case V with a conventional GCL in the same application as the multicomponent
GCL in Case VI, was 62-47 lphd (FE-SA) and the peak impact in the aquifer was 58-65 mg/l (FE-SA).
Thus, the use of the multicomponent GCL reduced the leakage 1.6 to 2-fold and the peak impact 8-fold
(Table 2). This case nicely contrasts the role of the transmissivity and hydraulic conductivity (especially
$k_b$), this is because the leakage directly beneath the wrinkle was reduced 67-fold due to the much lower
$k_b$ in the multicomponent GCL ($6 \times 10^{-12}$ vs $4 \times 10^{-10}$ m/s) but the 18-fold higher transmissivity $\theta$
($7 \times 10^{-10}$ vs $4 \times 10^{-11}$ m$^2$/s) spread the leachate out over a much larger area ($2a$ of 12 m vs 6.7 m) of the GCL. The
combination of a larger wetted area but 20-fold lower $k_a$ ($6 \times 10^{-12}$ vs $3 \times 10^{-11}$ m/s) for the multicomponent
GCL resulted in a 1.6 to 2-fold reduction in leakage and a 3 to 4-fold reduction in average Darcy flux.
Most importantly Darcy flux for the multicomponent GCL was uniformly low while for the
conventional GCL there was a substantial proportion of the leakage with high Darcy flux directly below
the wrinkle due to the (relatively) high head and high hydraulic conductivity in this region; this likely
more than anything else contributed to the 8-fold greater impact of the conventional GCL. This
comparison highlights the important interaction between the hydraulic conductivity of the GCL and the
interface transmissivity on not only the total leakage but also the distribution of that leakage and hence
the contaminant impact on an underlying aquifer that is not evident from a simple comparison of
calculated leakages.
Case VI is where the multicomponent GCL was placed with the geofilm-up against the GMB and so the interface transmissivity was for GMB-Geofilm. The same GCL could equally well be placed with the geofilm-down in contact with the attenuation layer and the nonwoven cover geotextile in contact with the GMB Case VII; Table 2). Since the hydraulic conductivity is controlled by the geofilm it would be the same in both cases VI and Case VII, however the transmissivity of the GMB-GCL interface could be reduced to as little as $4 \times 10^{-11} \text{m}^2/\text{s}$ thereby reducing leakage 3.6-fold from 29 lphd to 8 lphd and the peak impact about 2.7-fold from 5-8-7.9 mg/l to 2.1-2.9 mg/l. This how one uses a multicomponent GCL can make a difference to leakage and contaminant impact (although the impact is small in both cases).

3.3 Sensitivity analysis using FE analysis

Dispersivity, $\alpha$, tends to be a scale dependent and is not a true material property (Rowe et al. 2004). Transverse dispersivity, $\alpha_T$, affects the uniformity of contaminant distribution across the aquifer thickness whereas longitudinal dispersivity, $\alpha_L$, affects the mixing of contaminant in the direction of groundwater flow. The coefficient of hydrodynamic dispersion ($D_H$) is given by:

$$D_H = D_e + \alpha \cdot \frac{v_a}{n} \quad (\text{m}^2/\text{s})$$  \[7\]

where $D_e$ is the molecular diffusion coefficient, $v_a$ is Darcy flux, $n$ is the porosity and $\alpha$ is the dispersivity (m). The diffusion coefficient ($D_e$) of chloride for aquifer ($n=0.3$) was taken to be $6.3 \times 10^{-10}$ m$^2$/s (Rowe at al. 2004). Given the wide range of longitudinal dispersivity ($\alpha_L$) and transverse to longitudinal dispersivity ratios ($\alpha_T/\alpha_L$) available in the literature (e.g., Rowe et al. 2004), a sensitivity analysis was conducted to assess the effect of the coefficient of hydrodynamic dispersion ($D_H$) and ratio of transverse to longitudinal dispersivity ratio ($\alpha_L/\alpha_T$) on the chloride concentration in the aquifer.

Cases I and III (Table 2) were investigated for four different longitudinal dispersivity values ($\alpha_L$; Table 3) based on values given in Rowe et al. (2004). Case I-1 represents a low longitudinal dispersivity ($\alpha_L = 1 \text{ m}; D_H = 3.4 \text{ m}^2/\text{a}$), and in this case the peak chloride concentration was 3.6 mg/l at $x = 50 \text{ m}$.
(downgradient edge), 3.2 mg/l at 30 m \((x = 80\text{m})\) and 2.5 mg/l at 100 m \((x = 150\text{m})\) away of the facility edge. Case I-2 (\(\alpha_L = 3\text{ m}; D_H = 10.2 \text{ m}^2/\text{a}\)) had a peak concentration of 3.2 mg/l at the downgradient edge \((x = 50\text{m})\), 3.2 mg/l at \(x = 80\text{m}\) and 2.5 mg/l at \(x = 150\text{m}\). Case I-3 (\(\alpha_L = 100\text{ m}; D_H = 340 \text{ m}^2/\text{a}\)) represented a higher \(D_H\) that resulted in a decrease in calculated peak chloride concentration at the facility edge to 2.8 mg/l at 161 years (12% compared to Case I-1); similarly, the peak concentration also decreased to 2.6 mg/l at \(x = 80\text{m}\) and 2.4 mg/l at \(x = 150\text{m}\). Case I-4 had a coefficient of hydrodynamic dispersion \(D_H = 1000 \text{ m}^2/\text{a}\), yet the effect on contaminant concentration essentially the same as Case I-3 (Table 3; Figure 6). In all cases these values were below those calculated using the \(1^{1/2}D\) semi-analytical (Table 2) which can be regarded as conservative. It is also table that over a range of \(\alpha_L = \text{from 1 to 266m}\), \(c_p\) at 100m only decreased from 2.5 to 2.3 mg/l.

Case I above had only a negligible increase in chloride in the aquifer at the edge of the landfill (maximum \(c_p = 3.6\text{ mg/l}\)). Case III had a more significant effect with a maximum \(c_p = 43.5\text{ mg/l}\). However again there was no significant variation of the effect of the horizontal coefficient of hydrodynamic dispersion \((D_H)\) between Cases III-1a \((D_H = 3.75 \text{ m}^2/\text{a})\) and III-2 \((D_H = 11.3 \text{ m}^2/\text{a})\) on the contaminant concentration at the landfill edge. Likewise, there was no significant effect of this range of \(D_H\) at the monitoring points 30 and 100 m down-gradient of the landfill (Table 2; Figure 7). However, especially at the landfill edge \((x = 50\text{m})\), increasing \(D_H\) to 112.6 \text{ m}^2/\text{a}\) (Case III-3), 225 \text{ m}^2/\text{a}\) (Case III-4) and 375 \text{ m}^2/\text{a}\) (Cases III-5) reduced \(c_p\) by 27% from 43.5 mg/l to 31.6 mg/l with only a minor further decrease to 29.2 mg/l for \(D_H = 1000 \text{ m}^2/\text{a}\) (Cases III-6; Table 3).

To assess the effect of transverse dispersivity, Case III-1b had a transverse dispersivity of 1.0 m \((\alpha_T/\alpha_L = 1.0)\) whereas Case III-3b had a transverse dispersivity of 30 m \((\alpha_T/\alpha_L = 0.3)\). The effect of transverse dispersivity on the peak chloride concentration was negligible for both cases (Table 3). Figure 8 shows the variation of peak concentration for Cases I-1 to I-4 and Cases III-1a to III-6 with time to
peak (Table 3). The plots highlight the time required for the plume to develop at the landfill edge and the
time needed for the maximum concentration to reach the two monitoring points. For all the cases, the
time ($t_p$) at which the peak contaminant concentration was reached increased as the distance ($x$) from the
landfill edge increased (Table 3; Figure 8). The time to peak increased a little and the magnitude of the
peak concentration decreased with increasing $D_H$ at all three locations ($x = 50, 80$ and $150$ m).

In summary, horizontal dispersivity in the aquifer ($3.0 \text{ m}^2/\text{a} \leq D_H \leq 12 \text{ m}^2/\text{a}$) had little to modest
effect on contaminant impact at the monitoring points considered (Figure 6 and 7). Higher horizontal
dispersion ($12 \text{ m}^2/\text{a} \leq D_H \leq 300 \text{ m}^2/\text{a}$) showed a greater effect, since it reduced the contaminant impact in
the aquifer and the landfill edge by 26-32% (Table 3; Figures 6 and 7). Further increases in $D_H \geq 300$
$m^2/a$ had only a marginal (6~7%) effect (Figures 6 and 7). The three most notable observations were
that: (i) for any reasonable transverse dispersivity the aquifer concentration at the landfill edge was not
much affected in any practical way by uncertainty in that parameter; (ii) the impact at the end of the
landfill edge ($x=50$ m), $c_p$, gave a conservative estimate of impact away from the landfill and hence if $c_p < c_m$
(from Eq. 6) then the design will have an acceptable impact at the site boundary and it is not
necessary to analyze at the site boundary; and (iii) the impact at landfill edge $c_p$ began to approach that
calculated by the FE analysis at $x=150$ m for high $D_H = 1000$ m but was still slightly conservative
relative to the value calculated at 100 m from the edge (Table 3).

3.4 Leakage ($Q$) and $1/2$D SA contaminant migration analysis for holed wrinkles and failed seams

Based on the foregoing, it is concluded that the use of Eqs. 2 and 3 and the $1/2$D semi-analytic (SA)
analysis will give reasonable but conservative estimate of peak impact in an aquifer below the
downgradient edge of the landfill for the type of problem examined. It also became apparent, that with a
highly configured (in terms of CPU and RAM) modern PC it was not practical to get accurate results
from the FE analysis for a landfill cell larger than about 100m when dealing with a composite liner such
as those examined (e.g., Figs. 1 and 2). Thus, this simplified, SA, approach was used to evaluate the expected leakage ($Q$) through a composite liner and assess the chloride contaminant concentration ($c_p$) in the underlying aquifer for a landfill with a length of 1000 m in direction parallel to the groundwater (unless otherwise noted). Consideration will be given to different values of interface transmissivity ($\theta$), GCL hydraulic conductivity ($k_a$ and $k_b$), wrinkle lengths ($L_w$), waste loading, and failed seam lengths. Leakage ($Q$) was calculated using both Rowe (1998) Eq. 2 and SEEP/W (for a single wrinkle) for the cases discussed herein. Generally, the higher leakage value of the two approaches was used in SA contaminant analysis (Rowe and Booker 2005) although in some cases SA contaminant analyses were performed for leakage calculated by both methods and the results compared.

3.4.1 Effect of interface transmissivity and GCL hydraulic conductivity for GMB with holed-wrinkles

Consider a base case with waste loading of 140,000 m$^3$/ha (largest permitted by MoE 1998 for a generic design with a single composite liner) and a corresponding initial chloride concentration of 1500 mg/l and mass of 1800 mg chloride per kg of waste (as per Table 4 of MoE 1998) (Table 4).

Case 1A represented a landfill liner with hydraulic conductivity $k_b = k_a = 3 \times 10^{-11}$ m/s and interface transmissivity $\theta = 1 \times 10^{-11}$ m$^2$/s. This represents an optimistic case for a GCL in a landfill application since it neglects the effect of “stress-free” zone below the wrinkle which would lead to higher hydraulic conductivity. When used in a contaminant transport model, the calculated leakage of 15 lphd resulted in a peak chloride concentration of 19 mg/l in the aquifer after about 380 years (compared to 5.9 mg/L at 170 years when only one 100 m cell was examined in Table 2).

Case 1B represented a more realistic scenario compared to Case 1A as it accounts for the stress-free zone beneath the wrinkle. This resulted in a 4-fold higher leakage (53 lphd based on FE basic mesh; Case 1B-1) and an almost 6-fold higher chloride concentration (123 mg/l) which was very close to the allowable chloride concentration (125 mg/l) about 180 years after start of landfilling. However, this value was obtained using the 33% higher FE analysis flow than that obtained with Eq. 2 and as shown in
Section 3.2, this resulted in a 35% higher impact for a 100m landfill than obtained using the FE analysis. Thus, it could be hypothesized that although close to the allowable limit this landfill’s impact would be acceptable; this hypothesis will now be checked.

To assess the effect of the method of calculating leakage on the peak contaminant impact, Case 1B was analyzed using the flow obtained with Eq. 2 (40 lphd; Case 1B*) instead of the FE analysis flow (53 lphd, Case 1B-1). The calculated impact \(c_p\) was reduced from 123 mg/l (Case 1B-1) to only 80 mg/l (Case 1B*), suggesting that a 33% decrease in leakage resulted in 35% decrease in the contaminant concentration at the aquifer level. The time of peak occurrence increased from 180 years to 190 years.

Case 1B was also analyzed for the refined mesh leakage (\(Q\)) of 47 lphd (Case 1B-2, FE-analysis), the impact indicated a 15% decrease in peak concentration from 123 mg/L (53 lphd, Case 1B-1) to 105 mg/L (47 lphd; Case 1B-2, flow from more refined mesh-FE model), the time to peak remained at \(t_p = 180\) years. This highlights the significance of leakage, for the examined cases, in the impact assessment of the facility. It also confirmed that advection was the dominant transport mechanism.

Cases 1A and 1B represent a reasonable lower bound value of interface transmissivity \(\theta = 1\times10^{-11}\) m²/s at stresses of 100 kPa or more for leachate with no significant surfactant of (Rowe and Abdelatty 2013). However, \(\theta = 4\times10^{-11}\) m²/s, used in Case 1C, represents an upper end of the typical reported range (i.e., Barroso et al. 2008,2010; Mendes et al. 2010; Bannour and Touze-Foltz 2010; and AbdelRazek et al. 2016). Case 1C resulted in a leakage of 62 lphd and a peak chloride concentration of 128 mg/l in the aquifer (Table 4). This impact was obtained with a flow that is likely 33% higher thus it is considered that although slightly above the allowable limit, this landfill impact likely would be acceptable. To examine the effect of leakage, Case 1C was analyzed using leakage from Eq. 2 and a wetted distance from Eq. 3 and this gave a chloride concentration (86 mg/L) well below the allowable limit. These results do, however, show that the MoE (1998) limit set 20 years ago of 140 000 t/ha for a single lined
landfill was very appropriate and any significantly greater mass likely would lead to an unacceptable impact. These results also suggested that the wrinkles must be kept to less than 200 m/ha even for a relatively small landfill (140,000 m$^3$/ha) landfill to meet Ontario regulation 232 (MoE 1998) requirements.

Considering a composite liner with a multicomponent GCL having a thin smooth coating against the GMB, and using the hydraulic conductivity $k_b = k_a = 6 \times 10^{-12}$ m/s obtained by Rowe and Hosney (2013) at low stress, the calculated leakage varied from as high as 29 lphd (Case 1D, $\theta = 7 \times 10^{-10}$ m$^2$/s previously considered as Case IV in Section 3.2 for a 100 m long landfill) to as little as 10 lphd (Case 1E, $\theta = 6 \times 10^{-11}$ m$^2$/s) and 5 lphd (Case 1F; $\theta = 8 \times 10^{-12}$ m$^2$/s) (Table 4) and the peak chloride concentration ranged between 47 mg/l (for $\theta = 7 \times 10^{-10}$ m$^2$/s), 22 mg/l ($\theta = 6 \times 10^{-11}$ m$^2$/s), and 11 mg/l ($\theta = 8 \times 10^{-12}$ m$^2$/s). These impact calculations are valid for as long as the geofilm component of the multicomponent GCL remains effective as a hydraulic barrier. The long-term performance of the geofilm/laminate component of multicomponent GCLs requires investigations but it can be hypothesized that in the event of its ultimate failure, the results would be similar to it not being present at all (e.g., Case 1A-1C). Thus, even though one can calculate that longer wrinkles would be acceptable for a geofilm placed facing up against the GMB (e.g., Rowe et al. 2016), it would be prudent not to allow more wrinkles than what would give an acceptable impact if the geofilm were not present (i.e., $L \leq 200$ m/ha in this case).

### 3.4.2 Effect of different landfill sizes, wrinkle lengths, and waste loadings for holed-wrinkles

The leakage and peak contaminant concentration in the aquifer for landfills of different sizes (both length in direction of groundwater flow and mass loading), wrinkle length, and interface transmissivity $\theta$ were examined assuming a hydraulic conductivity of $k_b = 4 \times 10^{-10}$ m/s beneath the wrinkle and $k_a = 3 \times 10^{-11}$ m/s in contact with GMB (Table 5). Case 1B, previously presented in Table 4, is a reference case for comparison with Cases 2A to 2B. Case 1B represented a wrinkle length of 200 m and waste loading of
140,000 m$^3$/ha, the calculated leakage was 53 lphd associated with contaminant concentration of 123 mg/l (as discussed above). For the lowest waste loading (100,000 m$^3$/ha), the peak contaminant concentration estimated in the aquifer was 95 mg/l (Case 2B). Case 2A (120,000 m$^3$/ha), previously considered as Case III in Section 3.2 for a 100 m long landfill cell, represented an intermediate condition between 2B and 1B and the peak chloride concentration was 110 mg/l. Thus, other things being equal, a 40% increase in the average thickness of waste from 10 to 14 m increased the peak impact on the aquifer by 30% from 95 to 123 mg/L (for the same initial concentration of $c_o$ = 1500 mg/L).

Case 3A represented an extreme scenario where the waste loading exceeds the maximum waste loading allowed by Ontario Regulation 232/98 (MoE 1998) for the single liner design (the situation would be worse if a CCL were used instead of the GCL). A waste loading of 250,000 m$^3$/ha accompanied by initial chloride concentration of 2500 mg/l (Table 4 of Ontario Regulation 232/98) resulted in an excessive calculated peak contaminant concentration in the aquifer ($c_p$ = 330 mg/l), for a landfill of this length in the direction of groundwater flow and 200m/ha of holed-wrinkle; this peak concentration is almost 3 times the allowable concentration level of chloride ($c_m$ = 125 mg/l).

Case 3B is like Case 3A except that the holed-wrinkles were restricted to 100 m/ha thereby reducing the leakage by a factor of two to 27 lphd and this reduced the peak chloride concentration to 185 mg/L, 55% of the previous value (330 mg/L; Case 3A). Thus, for a landfill of this size and waste loading, even 100 m/ha of holed-wrinkles would be excessive with a single composite liner. Case 3C considers a smaller landfill 500 m long in the direction of groundwater flow and 200m/ha of holed-wrinkle. Since leakage ($Q$) is related to wrinkle length ($L_w$) (Rowe 1998; Eq. 2), the leakage of Case 3C was similar to Case 3A ($Q = 53$ lphd) but the contaminant concentration was lower (185 mg/L) because there was only half as much area affecting the aquifer and hence the total leakage to the aquifer was halved. Case 3D represents the same smaller landfill 500 m long where the holed-wrinkles were restricted to 100 m/ha.
thereby halving both the area and number of wrinkles compared to Case 3A reducing the peak impact to an still unacceptable 155 mg/L. These examples highlight the appropriateness of Ontario Regulation 232/98 limiting the allowable mass loading for a single liner design.

Cases 4A to 4D represented a liner with higher interface transmissivity $\theta=4\times10^{-11}$ $m^2/s$. Case 4A represented a waste loading of 250,000 m$^3$/ha long where the holed-wrinkles were restricted to 100 m/ha giving a leakage of 31 lphd and the peak chloride impact in the aquifer of 188 mg/L. Case 4B had the same waste loading but a holed-wrinkle length (50 m/ha) half that of Case 4A (100 m/ha), giving a leakage of 16 lphd and contaminant impact of 102 mg/L. This implies that for this mass loading the wrinkles would need to be kept to less than about 60 m of interconnected wrinkle/ha.

Cases 4C and 4D consider a waste loading of 140,000 m$^3$/ha and corresponding lower initial chloride concentration of 1500 mg/l. Case 4C (holed-wrinkle length of 200 m/ha), previously presented as Case V for a 100m long landfill in Section 3.2, had a leakage of 62 lphd and contaminant concentration of 128 mg/L, whereas Case 4D (holed-wrinkle length of 100 m/ha) had half the leakage at 31 lphd was associated with contaminant concentration of 68 mg/L. This highlights the importance of controlling wrinkle lengths at the time cover soil is placed over the GMB; an issue that does not currently receive sufficient attention in many projects.

### 3.4.3 Effect of different interface transmissivity and GCL hydraulic conductivity for failed seam

For a typical roll width, there will be over 1500 m of dual wedge weld per hectare plus many extrusion welds. Assuming that at the end of the welds service-life there is a linear failure in the heat affected zone along the edge of the dual wedge welds (Rowe and Shoaib 2017,2018), and that a 0.001 m-wide crack develops, the leakage can be calculated. Assuming no other defects in the GMB and no wrinkles coinciding with a failed seam, analyses were performed for a range of GCL hydraulic conductivities, transmissivities, and failed seam lengths (Table 6). The peak chloride impact was calculated assuming that the seam failure occurred at three times after the landfill was filled, viz: (a) $t=0$, assuming
(unrealistic but as a worst case) a seam failure immediately after the landfill was filled, (b) t=75 years (i.e., more realistically assuming that the seams do not fail until half the nominal service life of the GMB sheet (assuming a nominal service-life of 150 years as per MoE 1998 generic design), and (c) t=140 years (i.e., a decade before the GMB sheets nominal service life was reached). These values are then compared with the allowable chloride concentration of 125 mg/l.

Firstly, consider an essentially total seam failure (1500 m/ha) assuming a typical transmissivity range of 1x10^{-11} to 3x10^{-11} m²/s (Rowe 2012a) and a relatively low but reasonable hydraulic conductivity (k = 3x10^{-11} m/s; Petrov and Rowe 1997) under 150 kPa vertical stress for a conventional GCLs (Table 6). For these Cases S-A and S-B, the leakage was 60 lphd (θ = 1x10^{-11}) and 100 lphd (θ = 3x10^{-11}) respectively, assuming an immediate failure (i.e., at t=0) the peak chloride impact in the aquifer was calculated to be 86 mg/l (S-A) to 122 mg/l (S-B; Table 6). Thus, for allowable chloride concentration of 125 mg/l, in this case an ultimate immediate complete failure of the seams could just be tolerated provided that there were no wrinkles intersecting a failed seam. As soon as there were wrinkles, the impact can be expected to exceed allowable limit since the failed seam would invariable cross a wrinkle and allow transport of contaminant over a much larger area. More realistically assuming that the seams do not fail until at least 75 years, the peak chloride impact would be 28 mg/L (S-A) and 40 (S-B) whereas if the seams did not fail until 140 years, the impact would only be 9 mg/L (S-A) and 13 (S-B).

To illustrate the effect on interface transmissivity, Cases S-C and S-D simulated a liner with a high interface transmissivity of 1x10^{-10} m²/s and hydraulic conductivities of a reasonable 3x10^{-11} m/s (S-C) and very high 1x10^{-10} m/s (S-D) respectively. This resulted in leakage of 180 lphd (Case S-C) and 320 lphd (Case S-D) and the corresponding peak contaminant impact was an unacceptable 210 mg/l and 325 mg/l for an immediate (t=0) failure but these reduced to 68 mg/L (S-C) and 105 mg/L (S-D) for a failure at 75 years and 22 mg/L (S-C) and 34 mg/L (S-D) for a failure at 140 years (Table 6).
Cases S-E explore the effect of a shorter seam failure of length 200 m/ha for the same hydraulic parameters as case S-A, the reduction in failed seam length from 1500 to 200 m/ha reduce leakage from 60 lphd to about 8 lphd and the peak impact for immediate (t=0) failure from 86 mg/L (S-A) to 11 mg/L (S-E). Thus, provided it was not combined with wrinkles intersecting the seam or other defects in wrinkles then a limited seam failure would have essentially negligible impact for this case even if it occurred immediately for this small landfill. Cases S-F and S-G explore the effect of a limited (200 m and 100 m/ha) seam failure for a higher but still reasonable transmissivity and again, with the same caveats as above, a limited seam failure would have essentially negligible impact for this case even if it occurred immediately.

Cases S-H to S-J examine the performance of multicomponent GCLs with coating placed in contact with the GMB assuming a complete seam failure (Table 6). Despite to the low hydraulic conductivity provided by the coating, the peak concentration of chloride varied from 307 mg/l for a very high transmissivity $\theta = 7 \times 10^{-10} \text{ m}^2/\text{s}$ (S-H) to 38 mg/l for a very low transmissivity $\theta = 8 \times 10^{-12} \text{ m}^2/\text{s}$ (S-H) assuming immediate (t=0) failure. These values reduced to 100 mg/l for $\theta = 7 \times 10^{-10} \text{ m}^2/\text{s}$ (S-H) to 12 mg/l for $\theta = 8 \times 10^{-12} \text{ m}^2/\text{s}$ (S-H) for a failure at 75 years and to 32 mg/l for $\theta = 7 \times 10^{-10} \text{ m}^2/\text{s}$ (S-H) to 4 mg/l for $\theta = 8 \times 10^{-12} \text{ m}^2/\text{s}$ (S-H) for a failure at 140 years.

4. Summary

A method of using a closed form solution (Rowe 1998) to calculate leakage together with new approach for using an existing l½D semi-analytical contaminant transport model for modelling leakage at holed-wrinkle or failed seam was proposed and evaluated relative to a 2D finite element flow and transport model. The numerical/semi-analytical study examined the leakage and the peak chloride concentration in an aquifer for a single composite liner facility comprised of a 0.0015 m-thick GMB and a 0.007 m-thick GCL overlying a 3.74 m-thick attenuation layer over a 3.0 m-thick sandy aquifer. The study
investigated the effect of hydraulic conductivity of GCL \((k)\) and interface transmissivity \((\theta)\) on the estimated leakage and contaminant transport from a hypothetical containment facility for two types of defect: (i) a hole in a 0.1 m-wide (after compression by the weight of the waste) GMB wrinkle with a hole of length \(L_w \leq 200\, \text{m/ha}\), and (ii) a failed seam of lengths 1500, 200, 100 and 50 m/ha.

The 2D finite element modelling was also used in a sensitivity analysis to examine the effect of different longitudinal and transverse dispersivity values on the peak concentration in the aquifer downgradient landfill edge and two monitoring points placed at 30 m and 100 m downgradient facility. The findings of the analyses and the practical implications have been assessed in the context of the regulatory framework of Ontario Regulation 232 (MoE 1998).

The findings form this study may be summarized as follows:

1. Leakage rate \((Q)\) calculation for failed seam \((2b=0.001\, \text{m})\) using finite element (SEEP/w) and analytical solution (Rowe 1998) showed a good agreement, FE gave a 1% to 10% higher leakage than the analytical solutions.

2. For holed wrinkle \((2b=0.1\, \text{m})\), FE flow analysis and the Rowe (1998) equation showed good agreement when the ratio between the hydraulic conductivity beneath the wrinkle \((k_b)\) to the interface transmissivity \((\theta)\), \(k_b/\theta \leq 3.0\, \text{m}^{-1}\) for the mesh pattern adopted.

3. For cases of \(k_b/\theta \geq 3.0\, \text{m}^{-1}\), mesh refinement of FE flow model reduced the difference in leakage rate \((Q)\) values from 33% to 17%; thus, much if not most, of the discrepancy between the analytical and FE leakages appears to be due to difficulty of modeling this problems using a Finite Element analyses package commonly used by industry.

4. Finite Element (FE) analyses had difficulty coping with the large range in scale and hydraulic conductivities associate with modelling leakage from failed seams and holed-wrinkles for landfills. However, for the problems that could be analyzed with both the FE contaminant
transport analysis and the 1½D semi-analytic (SA) contaminant transport analysis, generally good agreement was obtained with the latter tending to predict slightly higher (conservative) peak impacts. The differences are considered small enough that they would not affect engineering decisions in any of the cases examined.

5. The importance of modelling the leakage focused on the wetted area only, and not averaging the leakage across the entire landfill (as is tempting to simplify the use both FE and SA methods), was illustrated by an example. In that case the analysis considering the transport only below wetted area showed that advective transport governs for a leakage of 55 lphd and that had this leakage been uniformly distributed it would correspond to an average Darcy flux (0.002 m/a) which is in the domain where one would expect diffusion to govern and that modeling using this average value would have underestimated the impact by an order of magnitude.

6. A FE examination of the effect of dispersivity in the aquifer indicated that (i) for any reasonable transverse dispersivity the aquifer concentration at the landfill edge was not affected in any practical way by uncertainty in that parameter; and (ii) the impact at the end of the landfill edge gave a conservative estimate of impact away from the landfill.

7. A crack in seam (2b=0.001 m) can cause significant leakage through composite liners if crack length exceeds about 200 m with a GCL OR if the crack intersects with a significant wrinkle.

8. These result show that the MoE (1998) limit set 20 years ago of 140 000 t/ha for a single lined landfill is very appropriate and any significantly greater mass likely would lead to an unacceptable impact. These results also suggested that the wrinkles must be kept to less than 200 m/ha even for a relatively small landfill (140,000 m³/ha) landfill in order to meet Ontario regulation 232 (MoE 1998) requirements.
5. Conclusion

A new approach to modelling contaminant impact by (i) using a closed form solution (Rowe 1998) to calculate leakage and the wetted area below holed-wrinkles or failed seams over which leakage occurs, and then (ii) modeling contaminant transport using this leakage only below the calculated wetted area using this in an existing a 1½D semi-analytic (SA) contaminant transport model was proposed. This approach was evaluated relative to a full 2D finite element (FE) flow and transport model. Based on the SA and FE analyses of the single composite liner and general configuration examined, it is concluded that:

1. The new approach to modelling contaminant impact using a closed form solution to obtain leakage and the wetted area for use in a new way of using the 1½D semi-analytic (SA) contaminant transport model have good, if sometimes slightly conservative, predictions of impact relative to the 2D finite element model.

2. Finite element analyses had difficulty coping with the large range in scale and hydraulic conductivities associate with modelling leakage from failed seams and holed-wrinkles for landfills. It was not practical to examine more than a relatively small portion of a landfill with any accuracy.

3. Holes in wrinkle network or failure of welds could be readily modelled for a full-size landfill using the proposed approach and 1½D semi-analytical contaminant transport analysis.

4. Leakage was highly dependent on the interaction between the interface transmissivity (θ) and hydraulic conductivity beneath the wrinkle (k_b). However, the magnitude of the leakage itself did not provided a good indicator of likely contaminant impact – much also depended on the distribution of the leakage at and away from the wrinkle. Thus, similar leakages arising from different combinations of transmissivity and hydraulic conductivity could have significantly different impacts on an underlying aquifer.
5. Contaminant transport modelling was needed to assess the contaminant impact for the likely range of uncertainty regarding interface transmissivity ($\theta$) and hydraulic conductivity both below the wrinkle ($k_b$) and away from the wrinkle ($k_a$).

6. Ontario’s Regulation 232/98 limit on the allowable mass loading for a single liner design is appropriate, provide there is also a limit on the amount of wrinkling, and hence the longest connected wrinkle(s) in the GMB to 200m/ha (or less), at the time the liner is covered by the leachate collection layer.

7. Holed-wrinkles play a critical role in the shorter term and wrinkles intersecting seams (as is often the case) can be expected to greatly compound the effect of an eventual seam failure.

In summary, the assessment of the impact of containment facility using 2D and $1\frac{1}{2}$D approaches were in relatively good agreement. The 2D approach is conceptually more comprehensive in the sense of capturing different complexities such as effects of dispersivity and migration outside the landfill, however it was very limited in the size of problem that could be accurately modeled given the greatly different scales that have to be considered. In contrast, the semi-analytic $1\frac{1}{2}$D approach readily allowed consideration of the highly variable scales, different hydraulic parameters, and different combinations of wrinkle lengths and waste loading for a typical containment facility and gave results at the downgradient edge sufficiently similar to the 2D, for the cases examined, to suggest that it is the more practical means of modelling these situations for typical design purposes.

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List of symbols

$2a$  Wetted distance measured from the holed wrinkle/failed seam centerline (m)

$2b$  Wrinkle / seam crack width (m)

$c_t$  Concentration at time (t) (mg/l)

$c_0$  Initial concentration of contaminant source (mg/l)

$c_b$  Background concentration of the contaminant in the groundwater of the receptor aquifer (mg/l)

$c_m$  Maximum allowable concentration of the contaminant in the groundwater of the receptor aquifer (mg/l)

$c_p$  Peak concentration of the contaminant in the groundwater of the receptor aquifer (mg/l)

$c_r$  Health related or aesthetic drinking water objective for the contaminant (mg/l)

$D$  Composite liner thickness over which head loss occurs (m)

$D_e$  Molecular diffusion (m$^2$/s)

$H_r$  Reference height of the contaminant (m)

$H_L$  GCL thickness (m)

$H_A$  Attenuation layer thickness (m)

$k_b$  Harmonic mean of the hydraulic conductivities of clay liner and attenuation layer for the area below the GMB wrinkle (m/s)

$k_a$  Harmonic mean of the hydraulic conductivities of clay liner and attenuation layer for the area of intimate contact with GMB (i.e. away of the wrinkle) (m/s)

$L_w$  Wrinkle length (m)

$L_{f\,\text{adjusted}}$  Adjusted length of the landfill (m)

$Q$  Leakage (m$^3$/s)

$q_o$  Infiltration rate (m/a)

$Q_c$  Leachate collected volume (m/a)

$v_a$  Darcy velocity (m/s)

$v_{b\,\text{out}}$  Groundwater velocity flowing out of the aquifer (m/a)

$\alpha$  Dispersivity (m)
Figure Captions

Figure 1. Schematic (not to scale) showing leakage through wrinkle of length $L_w$ (into page), width $2b$, and wetted distance, $a$, away from the center of the wrinkle over which there is downward flow

Figure 2. Boundary condition for finite element (FE) leakage (SEEP/w) and contaminant transport (CTRAN/w) model (a) plan view (b) cross-section

Figure 3. FE calculated variation in chloride concentration in the aquifer below the landfill cell from the upgradient to downgradient edge of the landfill for Cases I-VI at the time of peak impact

Figure 4. Effect of $k_h$ and $\theta$ on chloride concentration at the top of the aquifer versus time at dowgradient edge of the landfill for $1\frac{1}{2}$D and 2D analyses with 100 000 t/ha: (a) Case I with $k_h=3\times10^{-11}$ m/s and $\theta=1\times10^{-11}$ m$^2$/s (b) Case I-a with $k_h=3\times10^{-11}$ m/s and $\theta=4\times10^{-11}$ m$^2$/s, (c) Case I-b with $k_h=4\times10^{-10}$ m/s and $\theta=1\times10^{-11}$ m$^2$/s and (d) Case II with $k_h=4\times10^{-10}$ m/s and $\theta=4\times10^{-11}$ m$^2$/s

Figure 5. Effect of waste mass on chloride concentration versus time at downgradient edge of landfill for $1\frac{1}{2}$D and 2D analyses with same hydraulic parameters: (a) Case II with 100 000 t/ha (b) Case IV with 120 000 t/ha, and (c) Case V with 140 000 t/ha

Figure 6. Peak chloride concentration ($c_p$) versus coefficient of hydrodynamic dispersion ($D_H$) for Case I

Figure 7. Peak Concentration ($c_p$) versus coefficient of hydrodynamic dispersion ($D_H$) for Case III

Figure 8. Variation of peak concentration ($c_p$) with time ($t_p$) for landfill edge ($x=50$ m) and the two monitoring points ($x=80$ m) and ($x=150$ m) for Cases I and III
Appendix A

The semi-analytic finite layer analysis (Rowe and Booker; Rowe et al. 2004) was developed for layered media assuming 1D flow from a landfill through layers of different materials (e.g., GMBs, GCL, attention and other layers) to an underlying aquifer and horizontal movement of contaminant out from below a landfill. It can also be used for modelling lateral removal of contaminants from an aquifer between two aquitards. It has been used for many such applications but as typically used it is not suitable for modelling leakage through holes in wrinkles or failed seams between GMB panels and an modification of approach is needed to allow this common practical case to be modelled. The concept of the modified approach is outlined below; additional details and an example are given in AbdelRazek (2018). In the following it is assumed that the reader is familiar with the semi-analytic finite layer analysis as described by Rowe et al. (2004) and Rowe and Booker (2005). Consider $N$ similar periodically distributed wrinkles at a spacing $L_{sw} = L_f/N$.

Landfill parameters

Since the waste and leachate covers the entire landfill length ($L_f$) but can only leak through the adjusted length ($L_{f,\text{adjusted}}$) influence by the wetted distance from the holed-wrinkle or failed seam (Eqs. 3 and 5) , it is necessary to scale the reference height of chloride ($H_r$), which represent the mass available for transport,

$$H_{r,\text{adjusted}} = m_{TC}/(c_o \cdot L_{f,\text{adjusted}} \cdot W) \quad \text{(m)}$$  \[A1\]

where $W$ is the width of the landfill (m) perpendicular to groundwater flow being modelled ($W = L_w$ in this paper).

To maintain the same infiltration volume for the adjusted landfill length ($L_f$), adjusted infiltration rate ($q_{0,\text{adjusted}}$) can be calculated as:
where $q_0$ (m$^3$/m$^2$/yr) is the infiltration into the landfill per unit area.

The collected volume of leachate per unit area ($q_{collected}$) is the difference between the infiltration rate through the cover ($q_0$) and Darcy flux through the liner ($v_a$).

$$q_{collected} = q_0 - v_a \text{ (m}^3/\text{m}^2/\text{s})$$ \[A3a\]

and hence the adjusted values are given by

$$q_{collected\_adjusted} = q_0\_adjusted - v_a \text{ (m}^3/\text{m}^2/\text{s})$$ \[A3b\]

**GCL and attenuation layer parameters**

The Darcy flux through the GCL and attenuation layer are the same (continuity of flow) and given by Eq. 4. The coefficient of hydrodynamics dispersion comprises the molecular diffusion coefficient ($D_e$) and mechanical dispersion ($D_m$), the equation to calculate the dispersion through the layer beneath the adjusted length ($L_f$) is:

$$D = D_e + \alpha \cdot v_a / n, \text{ (m}^2/\text{s})$$ \[A4\]

where $\alpha$ is the dispersivity (m) and $n$ is the porosity (in this paper $n=0.3$ for the AL layer beneath the adjusted length and $n=0.7$ for GCL).

**Aquifer parameters**

Consider an aquifer of thickness ($h_b$), porosity ($n_b$) and with an inflow Darcy flux at the upgradient end of the landfill of $v_{bin}$ underlying the landfill of length ($L_f$). For an aquifer denoted by $Aq_1$ (Figure A1) between the wetted zone of adjacent wrinkles there is advective-dispersive-diffusive transport along the aquifer over a distance $L_{aq1} = L_f - 0.5L_f/N - L_f\_adjusted$ (i.e., neglecting the aquifer length $0.5L_f/N$ upgradient of the wetted zone of the first wrinkle). Also, over this distance between the wetted zone of adjacent wrinkles there is possible diffusive transport back into the overlying attenuation layer. Finally there is dilution of contaminant by the initial clean
pore water in the aquifer. All three aspects are considered. Considering firstly the flow in the aquifer it is necessary to allow enough time for contaminants in the aquifer to reach the downgradient edge of the landfill over a distance $L_{aq1}$. Figure A1 shows schematic to model the landfill involving only 1D migration from the attenuation layer through the aquifer to the point where it exits into the portion of the aquifer $Aq_0$ below the wetted areas (of length $L_{f\text{ adjusted}}$) that is conceptually adopted in this simplified description. The Darcy flux in the aquifer flowing down from the attenuation layer in Figure A1 where there this leakage, $v_{b\text{ out}}$ in this simplified description is given by:

$$v_{b\text{ out}} = v_{b\text{ in}} + \left(v_a \cdot \frac{L_{adj}}{h_b}\right)$$

[1] AbdelRazek (2018) indicated how the aquifer can be split into sub-aquifers and the increase in Darcy flux moving from one wrinkle to the next as adopted in the modelling reported herein, but this detail is omitted from this simplified conceptual version.
Figure A1. Schematic of the conceptual model for a landfill of length ($L_f$) as conceptually modelled using the semi-analytic finite layer model (Not to scale)

The contaminant is removed by modelling an aquifer $Aq_o$ as a SINK layer at the rate defined by $v_{b\ out}$ (Eq. A5). The porosity of the aquifer $Aq_o$ is adjusted to maintain the same volume of water in the aquifer pore space for dilution.

$$n_{b\ adjusted} = L_f \cdot (1 - 0.5/N) \cdot n_b / L_{adj} - L_{aq1} \cdot n_b / h_b$$  \hspace{1cm} [A5]

*Attenuation layer beneath the aquifer*

This layer accounts for the potential back diffusion over the length of the attenuation layer where there is no downward flow into the aquifer, $L_f^* = L_f - 0.5L_f/N - L_{f\ adjusted}$. The thickness of this layer was taken equal to the thickness of attenuation layer. The bottom of this layer was modelled in POLLUTE as a “No-Flux boundary” since it only simulates the back diffusion from the aquifer. Since the distance available for back diffusion below aquifer $Aq_o$, $L_{f\ adjusted}$, is less than $L_f^*$, the porosity of the previously neglected attenuation layer $n_{AL2}$ is adjusted to allow the same flux (attentively the diffusion coefficient could have been changed)

$$L_f^* = L_f - 0.5L_f/N - L_{f\ adjusted}$$  \hspace{1cm} [A6]

For equality of diffusive flux:

$$F = L_f \cdot n_{AL} \cdot D_{AL} \cdot \Delta C/\Delta t = L_{f\ adjusted} \cdot n_{AL2\ adjusted} \cdot D_{AL} \cdot \Delta C/\Delta t$$  \hspace{1cm} [A7]

$$n_{AL2\ adjusted} = n_{AL} \cdot L_f^* / L_{f\ adjusted}$$  \hspace{1cm} [A8]
Table 1. Leakage rate \((Q)\) calculated using FE analysis and Rowe (1998) Eq. 2

<table>
<thead>
<tr>
<th>Case ID</th>
<th>Leakage width (2b) (m)</th>
<th>(k_a) (GCL)</th>
<th>(k_b) (GCL)</th>
<th>(\theta) (m²/s)</th>
<th>Leakage length (L_w^a) (m/ha)</th>
<th>(k_w/\theta) (m⁻¹)</th>
<th>(Q_{FE}) (lphd)</th>
<th>(Q_{Eq. 2}) (lphd)</th>
<th>(Q_{FE}/Q_{Rowe}) (-)</th>
</tr>
</thead>
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<td>3x10⁻¹¹</td>
<td>1x10⁻¹¹</td>
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<td>53</td>
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<td>1x10⁻¹⁰</td>
<td>1500</td>
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<td>207</td>
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</tr>
<tr>
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<td>22.3</td>
<td>1.01</td>
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</table>

\(a\) Leakage length was assumed as 1500 m/ha for failed seam and 200 m/ha for holed wrinkle, unless otherwise noted

\(b\) Same parameters as Case 8 but for twice the wrinkle length
Table 2. Leakage ($Q$) and peak chloride concentration ($c_p$) in the aquifer calculated with 2D finite element (FE) and 1.5D semi-analytical (SA) analysis for conventional GCL (Cases I-V) and multicomponent (Case VI) GCL. Landfill length=100 m, 2 wrinkles length $L_w = 100$ m and initial chloride concentration ($c_0$) =1500 mg/l. Results rounded to 2 significant figures.

<table>
<thead>
<tr>
<th>Case</th>
<th>Waste loading (m$^3$/ha)</th>
<th>$k_a$ (GCL)</th>
<th>$k_b$ (GCL)</th>
<th>$\theta$ (m$^2$/s)</th>
<th>$Q_{FE}$ (lphd)</th>
<th>$Q_{Eq. 2}$ (lphd)</th>
<th>$c_p$ (mg/l)</th>
<th>$t_p$ (yrs.)</th>
<th>$c_p$ (mg/l)</th>
<th>$t_p$ (yrs.)</th>
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</thead>
<tbody>
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<td>3x10$^{-11}$</td>
<td>1x10$^{-11}$</td>
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<tr>
<td>Ia</td>
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<td>95</td>
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<td>4x10$^{-11}$</td>
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<td>71</td>
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<td>82</td>
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Table 3. Effect of horizontal coefficient of hydrodynamic dispersion ($D_{H}$) in aquifer on the peak chloride concentration ($c_{p}$) at different points: landfill edge (x=50 m), 30 m away from landfill edge (x=80 m), and 100 m away from landfill edge (x=100m)

<table>
<thead>
<tr>
<th>Case ID</th>
<th>$\alpha_{L}$ (m)</th>
<th>$\alpha_{T}/\alpha_{L}$</th>
<th>$v_{h\ out}$ (m/a)</th>
<th>$D_{H}$ (m$^{2}$/a)</th>
<th>Edge x=50 m</th>
<th>Monitoring point locations x=80 m</th>
<th>Monitoring point locations x=150 m</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>$c_{p}$ (mg/l)</td>
<td>$t_{p}$ (years)</td>
<td>$c_{p}$ (mg/l)</td>
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<td>3.6</td>
<td>127</td>
<td>3.2</td>
<td>190</td>
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<td>$3 \times 10^{3}$</td>
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<td>10.2</td>
<td>3.2</td>
<td>152</td>
<td>3.2</td>
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<tr>
<td>I-3</td>
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<td>100</td>
<td>340</td>
<td>2.8</td>
<td>161</td>
<td>2.6</td>
<td>200</td>
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<tr>
<td>I-4</td>
<td>266</td>
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<td>1000</td>
<td>2.7</td>
<td>171</td>
<td>2.6</td>
<td>194</td>
</tr>
<tr>
<td>III-1a</td>
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<td>$10^{4}$</td>
<td>3.75</td>
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<td>79</td>
<td>35.5</td>
<td>120</td>
</tr>
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<td>1</td>
<td>3.75</td>
<td>43.4</td>
<td>76</td>
<td>35.8</td>
<td>102</td>
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<td>III-2</td>
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<td>$3 \times 10^{3}$</td>
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<td>43.0</td>
<td>79</td>
<td>35.6</td>
<td>118</td>
</tr>
<tr>
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<td>30</td>
<td>$3 \times 10^{2}$</td>
<td>1.13</td>
<td>112.5</td>
<td>36.7</td>
<td>95</td>
<td>29.35</td>
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<td>III-4</td>
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<td>225</td>
<td>33.6</td>
<td>100</td>
<td>28</td>
<td>120</td>
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<tr>
<td>III-5a</td>
<td>100</td>
<td>100</td>
<td>375</td>
<td>31.6</td>
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<td>27.7</td>
<td>120</td>
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<tr>
<td>III-5b</td>
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<td>0.3</td>
<td>375</td>
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<td>100</td>
<td>27.7</td>
<td>119</td>
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<tr>
<td>III-6</td>
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<td>37.5</td>
<td>1000</td>
<td>29.2</td>
<td>110</td>
<td>27.8</td>
<td>119</td>
</tr>
</tbody>
</table>

$v_{h\ out}$ = Horizontal Darcy flux in aquifer at the exit boundary
Table 4. Effect of different flow parameters \((k\) and \(\theta\)) on calculated leakage \((Q)\) and peak chloride concentration \((c_p)\) in the aquifer for a conventional GCL (Cases 1A-1C) and multicomponent GCL (Cases 1D-1F); waste loading = 140,000 m\(^3\)/ha, landfill length \(L_f=1000\) m, wrinkle length \(L_w=100\) m, 2-holed wrinkles/ha (200m/ha). Time to peak rounded to nearest decade.

<table>
<thead>
<tr>
<th>Case</th>
<th>(k_a) (GCL) (\times 10^{-11})</th>
<th>(k_b) (GCL) (\times 10^{-10})</th>
<th>(\theta) ((m^2/s))</th>
<th>(Q) (FE) (lphd)</th>
<th>(Q) (Eq. 2) (lphd)</th>
<th>(c_p) in aquifer ((mg/l))</th>
<th>Approx. time to peak ((years))</th>
</tr>
</thead>
<tbody>
<tr>
<td>1A</td>
<td>3x10^{-11}</td>
<td>3x10^{-11}</td>
<td>1x10^{-11}</td>
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<td>19</td>
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<tr>
<td>1B-1</td>
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<td>4x10^{-10}</td>
<td>1x10^{-11}</td>
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<td>-</td>
<td>123</td>
<td>180</td>
</tr>
<tr>
<td>1B-2</td>
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<td>4x10^{-10}</td>
<td>1x10^{-11}</td>
<td>47</td>
<td>-</td>
<td>105</td>
<td>180</td>
</tr>
<tr>
<td>1B*</td>
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<td>4x10^{-10}</td>
<td>1x10^{-11}</td>
<td>-</td>
<td>40</td>
<td>80</td>
<td>190</td>
</tr>
<tr>
<td>1C</td>
<td>3x10^{-11}</td>
<td>4x10^{-10}</td>
<td>4x10^{-11}</td>
<td>62</td>
<td>-</td>
<td>128</td>
<td>180</td>
</tr>
<tr>
<td>1C*</td>
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<td>4x10^{-11}</td>
<td>-</td>
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<td>47</td>
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<tr>
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<td>5</td>
<td>11</td>
<td>520</td>
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</table>

* Leakage using Eq. 2 was used for the impact assessment

Table 5. Effect of different landfill sizes, wrinkle lengths, and waste loadings on calculated leakage rate \((Q)\) and peak chloride concentration \((c_p)\) in the aquifer at the edge of the landfill for conventional GCL \((k_a=3x10^{-11} m/s; k_b=4x10^{-10} m/s)\). Time to peak rounded to nearest decade.

<table>
<thead>
<tr>
<th>Case</th>
<th>Waste loading ((m^3/ha))</th>
<th>(c_o) ((mg/l))</th>
<th>Landfill length ((m))</th>
<th>Wrinkle length ((m))</th>
<th>(\theta) ((m^2/s))</th>
<th>(Q) (FE) ((lphd))</th>
<th>(Q) (Eq. 2) ((lphd))</th>
<th>(c_p) in aquifer ((mg/l))</th>
<th>Approx. time to peak ((years))</th>
</tr>
</thead>
<tbody>
<tr>
<td>1B</td>
<td>140,000</td>
<td>1500</td>
<td>1000</td>
<td>200</td>
<td>1x10^{-11}</td>
<td>53</td>
<td>40</td>
<td>123</td>
<td>180</td>
</tr>
<tr>
<td>2A</td>
<td>120,000</td>
<td>1500</td>
<td>1000</td>
<td>200</td>
<td>1x10^{-11}</td>
<td>53</td>
<td>40</td>
<td>110</td>
<td>150</td>
</tr>
<tr>
<td>2B</td>
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<td>1500</td>
<td>1000</td>
<td>200</td>
<td>1x10^{-11}</td>
<td>53</td>
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<td>1x10^{-11}</td>
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<tr>
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<td>2500</td>
<td>1000</td>
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<tr>
<td>3C</td>
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<td>1x10^{-11}</td>
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<td>185</td>
<td>280</td>
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<tr>
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<td>2500</td>
<td>500</td>
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<td>1x10^{-11}</td>
<td>27</td>
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<td>250</td>
</tr>
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<td>1000</td>
<td>200</td>
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<td>47</td>
<td>128</td>
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<tr>
<td>4D</td>
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<td>1500</td>
<td>1000</td>
<td>100</td>
<td>4x10^{-11}</td>
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</tbody>
</table>

* \(c_o\) varied from 1500 mg/L to 2500 mg/L for mass loading of 150 000 t/ha to 250 000 t/ha assuming waste density of 1.0 t/m\(^3\)
Table 6. Calculated leakage rate ($Q$) and peak chloride concentration ($c_p$) in the aquifer for a failed seam in a 1000 m long landfill with waste loading 140,000 m$^3$/ha, waste density 1.0 t/m$^3$, $p = 1800$ mg/kg, infiltration $q_o = 0.15$ m/a, and initial chloride concentration $c_o = 1500$ mg/L.

<table>
<thead>
<tr>
<th>Case ID</th>
<th>$k$ (GCL) (m/s)</th>
<th>$\theta$ (m$^2$/s)</th>
<th>Failed seam length (m/ha)</th>
<th>$Q_{FE}$ (lphd)</th>
<th>Time after landfilling at which seam failure occurred (years)</th>
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Figure 1. Schematic (not to scale) showing leakage through wrinkle of length $L_w$ (into page), width $2b$, and wetted distance, $a$, away from the center of the wrinkle over which there is downward flow.
Figure 2. Boundary condition for finite element (FE) leakage (SEEP/w) and contaminant transport (CTRN/w) model (a) plan view (b) cross-section
Figure 3. FE calculated variation in chloride concentration in the aquifer below the landfill cell from the upgradient to downgradient edge of the landfill for Cases I-VI at the time of peak impact.
Figure 4. Effect of $k_b$ and $\theta$ on chloride concentration at the top of the aquifer versus time at downgradient edge of the landfill for 1/2D and 2D analyses with 100 000 t/ha: (a) Case I with $k_b=3\times10^{-11}$ m/s and $\theta=1\times10^{-11}$ m²/s (b) Case I-a with $k_b=3\times10^{-11}$ m/s and $\theta=4\times10^{-11}$ m²/s, (c) Case I-b with $k_b=4\times10^{-10}$ m/s and $\theta=1\times10^{-11}$ m²/s and (d) Case II with $k_b=4\times10^{-10}$ m/s and $\theta=4\times10^{-11}$ m²/s

Figure 5. Effect of waste mass on chloride concentration versus time at down-gradient edge of landfill for 1/2D and 2D analyses with same hydraulic parameters: (a) Case II with 100 000 t/ha (b) Case IV with 120 000 t/ha, and (c) Case V with 140 000 t/ha
Figure 6. Peak chloride concentration ($c_p$) versus coefficient of hydrodynamic dispersion ($D_H$) for Case I

Figure 7. Peak Concentration ($c_p$) versus coefficient of hydrodynamic dispersion ($D_H$) for Case III
Figure 8. Variation of peak concentration ($c_p$) with time ($t_p$) for landfill edge ($x=50$ m) and the two monitoring points ($x=80$ m) and ($x=150$ m) for Cases I and III.