Technical Paper by R.S. Thiel

DESIGN METHODOLOGY FOR A GAS PRESSURE RELIEF LAYER BELOW A GEOMEMBRANE COVER TO IMPROVE SLOPE STABILITY

ABSTRACT: Pore pressures generated by landfill gas underneath a geomembrane final cover can significantly reduce the effective normal stress on the lower geomembrane interface to the point of creating a cover veneer instability. To the author's knowledge, no design methodology has previously been published to address this issue. Recently, however, large-scale slope failures have been attributed to landfill gas pore pressures. Therefore, a need for a design methodology exists. An estimation of gas flux from a landfill surface can allow a gas-relief layer to be designed using Darcy's law for gas flow through a porous medium. The methodology incorporates knowledge of the gas transmissivity of a chosen medium to design a spacing for highly-permeable strip drains. The strip drains in turn would discharge the gas either to vents or an active gas collection system. The gas-relief layer typically consists of sand or a geonet-composite. Limited testing of nonwoven-needlepunched (NWNP) geotextiles indicates that these materials may also be acceptable for gas relief in some designs. However, more testing is recommended before using NWNP geotextiles alone in this application. The grossest assumption in the proposed methodology concerns the estimation of gas flux. More work is needed in this regard. However, the basic concept of providing a gas-relief layer with intermittent highly-permeable strip drains is recommended as a prudent engineering measure for landfill final covers incorporating geomembrane barriers.

KEYWORDS: Geomembrane, Landfill cover, Slope stability, Landfill gas

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1. INTRODUCTION

Landfill gas is continuously generated in a landfill as the waste decomposes. The concern for final landfill cover designs incorporating geomembranes is that an uplift pressure can be caused by the gas. From a slope-stability point of view, gas pressure is an excess pore pressure that serves to reduce the effective normal stress. Classic soil mechanics methods provide all the tools necessary to address the slope stability of landfill covers, including the effects of gas pressure. However, what has been lacking to date are a) an explicit recognition of gas pressures as a design issue for landfill covers, b) a methodology to incorporate the fluid-mechanics calculations of gas (versus water) in the slope stability analyses, and c) an understanding of how to estimate gas relief requirements. This paper will attempt to address points a) and b) above, and provides some guidelines and experience relative to point c). However, estimation of landfill gas generation rates, and the resulting flux from the upper surface, is beyond the expertise of the author and, in the author's experience, is more of an art than a science even for the gas "experts."

OUTLINE OF GAS RELIEF CALCULATION PROCEDURE

The three primary steps for the gas relief calculation are described below to provide the reader with an overview of the logic, and to avoid becoming lost in the details later.

a) Estimate the maximum flux of gas that may need to be removed below the landfill final cover. The units of flux are volume per surface area per time, such as cubic meters of gas per hour per square meter of landfill surface ($m^3/hr/m^2$).

b) Perform slope stability analyses to estimate the maximum allowable gas pressure that results in an acceptable overall static factor of safety. In this case the designer can use any cover slope stability model deemed appropriate for the project, but must be able to incorporate gas pore pressures from below, as described in detail later.

   Several papers describing landfill cover veneer slope stability have been presented in the literature (for example Koerner and Soong, 1998; Kavazanjian, 1998; Thiel and Stewart, 1993). In these papers different considerations for cover slope stability are presented and developed, including infinite slope approaches, seepage forces, seismic forces, toe buttressing forces, tapered slopes, and slope reinforcement. It is left to the individual practitioner to select the model most appropriate for a given situation to develop the design. In the interests of brevity, the slope stability equations used in this paper for the development of gas pressure considerations will be limited to non-reinforced, static, infinite-slope conditions. However, the principles developed herein to include gas pressures in a stability analysis could easily be combined with other models as well.

c) Design a passive vent system below the cover that will evacuate the gas at a flow rate that matches the design flux calculated in a) above, and under a maximum allowable driving pressure determined in b) above.

Each of the three steps summarized above is described in detail below.

2. ESTIMATION OF GAS FLUX

The mass flux of gas from the surface of a landfill (Φ_g) will be site specific, and will vary spatially and temporally at a given landfill. The amount of gas will depend on the waste type, age, temperature, moisture, other avenues of gas extraction or venting, barometric pressure, etc. The literature reports landfill gas generation rates up to 0.037 standard cubic meters per wet kilogram of waste per year (0.037 m^3/kg/yr) (Pacey, 1997). However, this value is exceptionally high and is reported for controlled landfills in an enhanced decomposition mode. For closures at municipal solid waste landfills in the northwestern United States, where cell closure occurs at the end of a cell's life, the author frequently uses a gas flux of 6.24×10^{-3} m^3/kg/yr for purposes of cover design. When using gas modeling computer programs, the author recommends that the upper part of the gas estimation curve be utilized for purposes of cover slope stability. Estimation of the flux rate is very site specific, and is beyond the scope of this paper.

Example 1. Gas Flux Calculation

Given an average waste depth under the final cover area of 30 m, a waste density of 800 kg/m^3, and a landfill gas generation rate of 6.24×10^{-3} m^3/kg/yr. What is the estimated gas flux from the landfill surface?

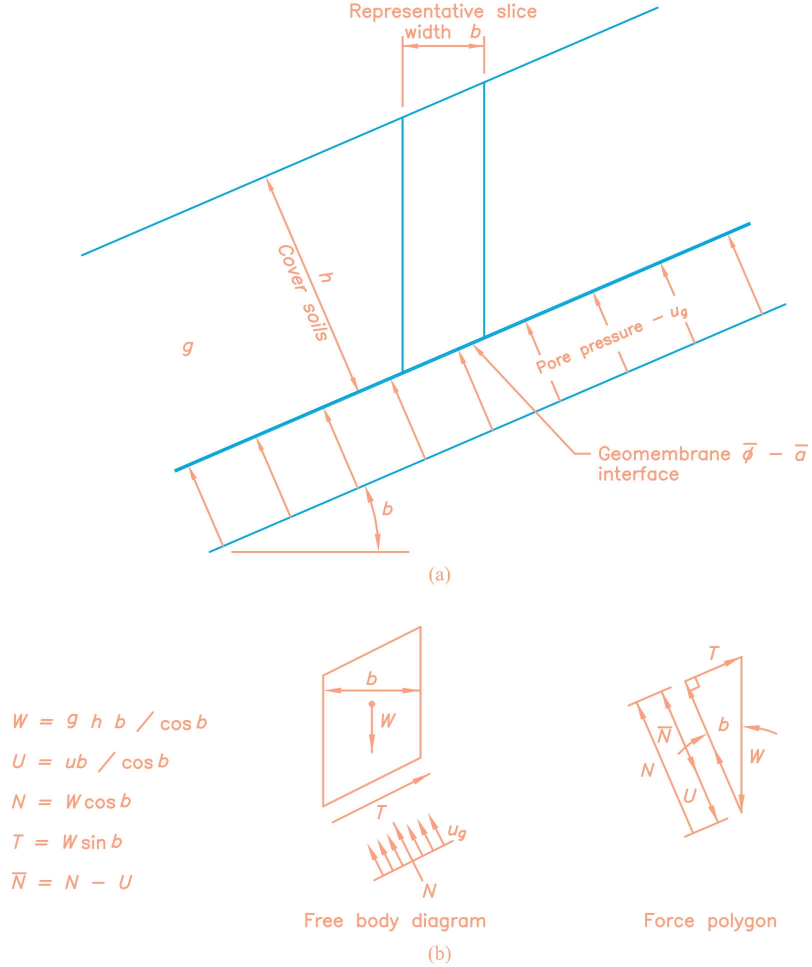
The gas flux, Φ_g, can be estimated as follows:

$$\Phi_g = \frac{0.00624\ \text{m}^3}{\text{kg} \cdot \text{yr}} \times \frac{1\ \text{yr}}{8{,}760\ \text{hr}} \times \frac{30\ \text{m}^3}{\text{m}^2} \times \frac{800\ \text{kg}}{\text{m}^3} = 0.017\ \frac{\text{m}^3}{\text{hr} \cdot \text{m}^2}$$

END OF EXAMPLE 1

3. SLOPE STABILITY CALCULATIONS INCORPORATING GAS PRESSURES USING AN INFINITE-SLOPE ANALYSIS

The general cross section of an infinite landfill slope with a final cover is shown in Figure 1(a). The free-body diagram and force polygon for a vertical slice of the cover section to the interface just below the geomembrane are shown in Figure 1(b). Because of the hydraulic break provided by the barrier geosynthetic (assumed to be a geomembrane), seepage forces that may occur in the cover soils above the geomembrane have no influence on the stability of the interface below the geomembrane. Therefore, separate slope stability analyses are required for the geomembrane's upper and lower interfaces. The stability analysis presented herein would only be for the lower interface, where the gas pressures would potentially occur.



Geometric parameters: b = slope angle; h = cover soil thickness

Material parameters: g = unit weight of cover soil; $\bar{\phi}$ = effective friction parameter for lower geomembrane interface; $\bar{a}$ = effective adhesion parameter for lower geomembrane interface.

Pressures and forces: u_g = pore pressure acting on bottom of geomembrane; U = uplift force on slice; W = total weight on slice; N = total force normal to slope exerted by W; T = tangential force to slope exerted by W; $\bar{N}$ = effective normal force.

Figure 1. Infinite slope stability with pore pressures below geomembrane: (a) Infinite slope geometry and material parameters; (b) Forces.

The effective stress normal to the base of the slice, $\overline{\sigma}$, is

$$\overline{\sigma} = \frac{\overline{N}}{b / \cos\beta} = h\gamma \cos\beta - u_g \tag{1}$$

The shear stress exerted tangential to the slice, τ, is

$$\tau = \frac{T}{b / \cos\beta} = h\gamma \sin\beta \tag{2}$$

The resisting shear strength at the base of the slice, *R*, is

$$R = \overline{a} + \overline{\sigma}\, tan\,\overline{\phi} \tag{3}$$

The factor of safety, defined as the ratio of the resisting shear strength divided by the driving shear stress, is

$$FS = \frac{R}{\tau} = \frac{\overline{a} + [\,h\gamma\, cos\,\beta - u_g\,]\, tan\,\overline{\phi}}{h\gamma\, sin\,\beta} \tag{4}$$

Presuming that the material properties and geometry are fixed for a specific design, the designer must then select a minimum allowable factor of safety, FS_{allow} ,and calculate a maximum allowable gas pressure, $u_{(g-allow)}$. This can either be done iteratively using Equation (4), or solved for explicitly as

$$u_{(g-allow)} = h\gamma\, cos\,\beta - \frac{\left[FS_{allow} \cdot h\gamma\, sin\,\beta - \overline{a}\right]}{tan\,\overline{\phi}} \tag{5}$$

Example 2. Cover Slope Stability Analysis to Determine Maximum Allowable Pressure

Given a final cover system on a 1(V):3(H) slope (18.4°) that consists of the following elements, from top to bottom:

- final cover soils and drainage layer thickness (*h*) = 0.9 m; with an average unit weight (γ) = 15.7 N/m^3
- geomembrane over sand with an interface shear strength between the geomembrane and underlying sand of 27° friction ($\overline{\phi}$) and zero adhesion ($\overline{a}$).

What is the variation in factor of safety (*FS*) for gas pressures ranging from zero to 4 kPa?

Solution: Using Equation (4), the *FS* is plotted against the variation in assumed gas pressure as shown in Figure 2. The results show that even for a relatively strong interface between the geomembrane and underlying sand (27° friction), very little excess gas pressure (only about 0.3

kPa) can be tolerated before the factor of safety drops below 1.5. In selecting a value for $u_{(g-allow)}$, the designer needs to use judgment regarding an acceptable *FS* for this condition, noting that in most cases the gas flux will diminish over time. Also, the gas pressure is not uniform under the cover, as discussed in the next section.

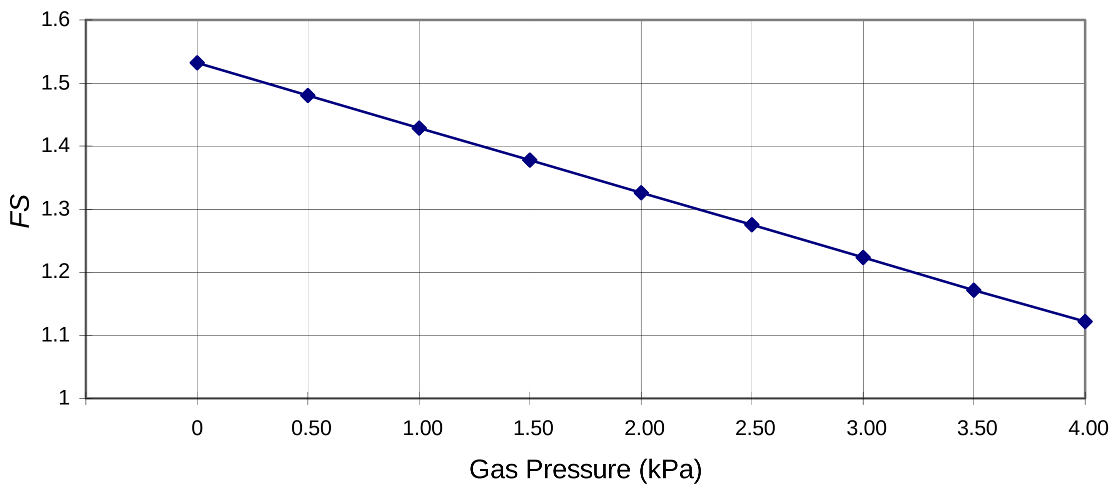


Figure 2. Solution for Example 2.

END OF EXAMPLE 2

4. DESIGN OF GAS PRESSURE RELIEF SYSTEM

4.1 Proposed Gas Pressure Relief System Geometry

The vent system immediately below a landfill cover barrier layer will conceptually consist of a transmissive gas-relief blanket layer below the cover system with occasional outlets that penetrate the cover system. The blanket gas-relief layer might just consist of the uppermost layer of waste itself, or it could be a permeable sand or geosynthetic layer. The gas-relief layer could be enhanced by the inclusion of intermittent gravel-filled trenches or strip drains, of greater permeability than the gas-relief layer, to aid in the transmission of gas to the outlets. The outlets might either be vents going directly to the atmosphere, or hooked up to an active (vacuum) gas collection system. These surface gas relief features would be in addition to, and are a separate consideration from, any other gas collection elements such as vertical or horizontal gas wells that penetrate deeper into the waste.

For purposes of the model proposed in this paper, the surface gas relief layer is assumed to be composed of the following three primary elements:

- a blanket gas-relief layer

- a series of parallel trenches or strip drains (the term ‘strip drains’ is used in the remainder of the paper), at a regular spacing (D), that collect gas from the gas-relief layer, and are more permeable than the gas-relief layer to allow the gas to be conveyed to the outlets

- outlet points for the strip-drains

4.2 Derivation of Design Equations

Figure 3(a) shows a typical landfill slope cross section, with an emphasis on the gas collection layer below the barrier layer. In the cross section two benches are shown (which could just as well be the crest and toe of slope for short landfills). Strip drains, which could be perforated pipes, gravel filled trenches, or geosynthetic highway edge drains, are shown running longitudinally along the benches. The distance *D* is defined as the slope distance between the strip drains. Figure 3(b) shows a schematic plan view of the strip-drain layout for this situation, and also indicates that outlet points (in this case vents to the atmosphere) would be intermittently located along the strip drains to relieve the collected gas.

In the event that the strip-drain spacing between benches is found to be inadequate, additional strip drains could be connected in the slope direction between benches. This is illustrated in Figure 3(c) where the spacing *D* is now defined as the distance between the drains running up and down the slope. In this case the strip drains along the benches would serve as headers.

One of the principles illustrated in the preceding discussion is that the orientation of the strip drains does not matter. That is, since the unit weight of landfill gas is so close to that of air, the gas will travel upslope as easily as downslope (or sideslope), and the elevation head does not influence the calculations.

The derivation of the relationship between the strip drain spacing (*D*), incoming gas flux rate, gas transmissivity of the gas-relief layer, and pressure in the gas relief layer is similar to the design of the drainage layer and drainage layer outlets in the cover above the geomembrane as presented by Thiel and Stewart (1993). The derivation is based on Darcy's law, which applies to fluid flow in porous media where the flow is laminar. (Proof of the applicability of laminar flow in this situation is discussed later.) The derivation steps are as follows:

1. Consider a unit-width surface area between strip drains, as shown in Figure 4(a). Figure 4(b) illustrates a cross-section between two strip drains, showing the gas flux coming uniformly into the gas-relief layer from the waste below. Ideally, the gas flow is symmetric about the centerline between the strip drains, and we need only consider the half-distance, *L*, where $L = D/2$. The figure identifies the variable distance 'x' beginning at one of the strip drains, and increasing towards the centerline.

2. Figure 5 illustrates how the volume of gas being carried in the gas-relief layer would vary linearly from zero at $x=L$, to a maximum value at $x=0$. The volume of gas per unit width can be written in terms of the gas flux as

$$Q_x = \Phi_g\left(L - x\right) \tag{6}$$

where Q_x is the gas discharge flow rate per unit width at any point x in the gas-relief layer.

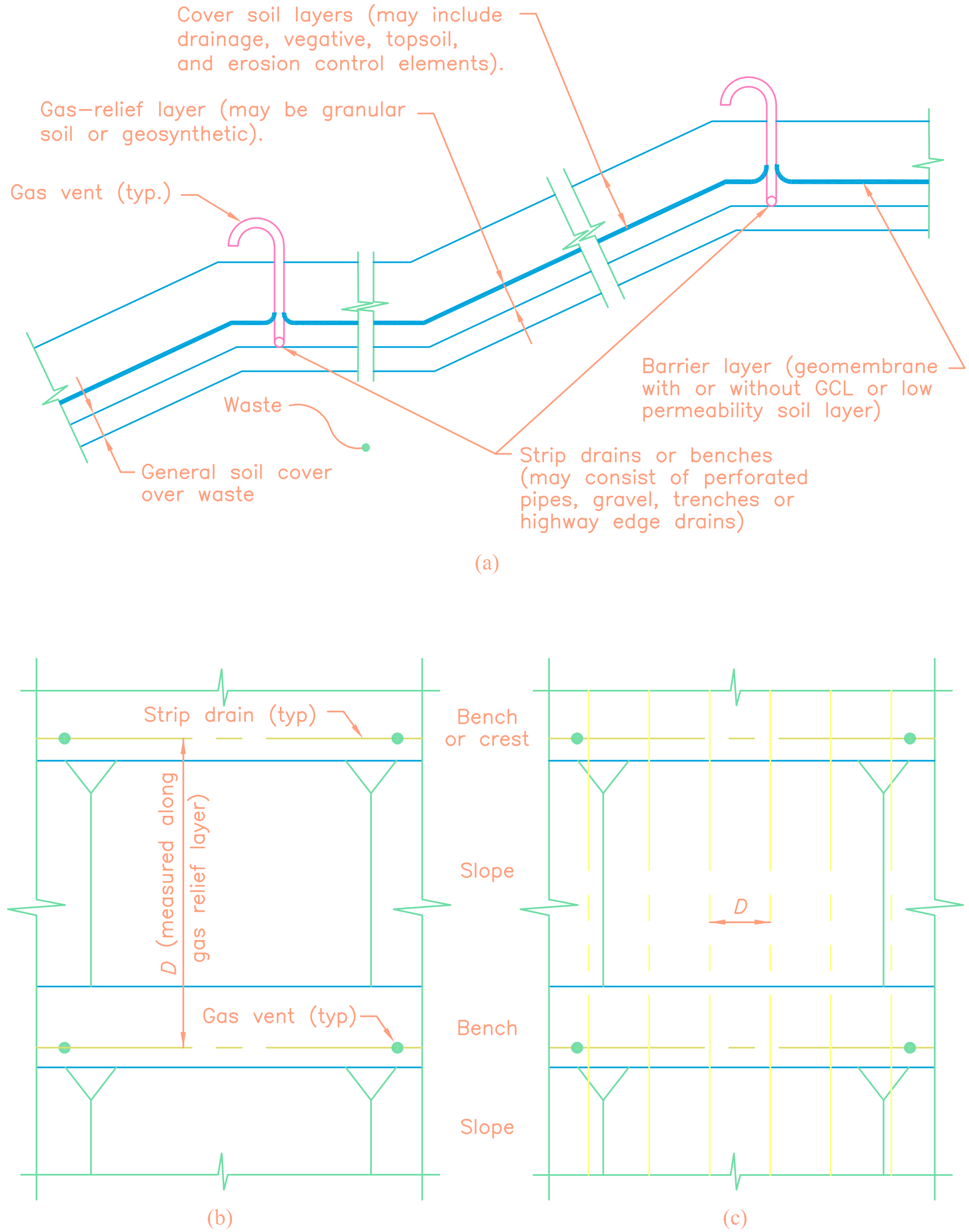


Figure 3. Schematic showing design elements of gas-relief layer: (a) Side view of final cover with gas-relief layer and strip drains; (b) Plan view schematic of strip drain layout on benches only; (c) Plan view of strip drain layout on slopes and benches.

3. The flow of gas in the gas-relief layer can be assumed to follow Darcy's law, which can be written in terms of the pressure gradient as follows:

$$Q_x = \left(\frac{k_g}{\gamma_g}\right) \cdot A \cdot \left(\frac{\mathrm{d}\,u_g}{\mathrm{d}x}\right) = \left(\frac{k_g}{\gamma_g}\right) \cdot (t \times 1) \cdot \left(\frac{\mathrm{d}u_g}{\mathrm{d}x}\right) = \left(\frac{k_g \cdot t}{\gamma_g}\right)\left(\frac{\mathrm{d}u_g}{\mathrm{d}x}\right) \tag{7}$$

where k_g=gas permeability of the gas-relief layer; γ_g=the gas unit weight; A=cross-sectional flow area which is the thickness of the layer (t) times a unit-width; and du/dx is the pressure gradient.

4. Since we can define the transmissivity (Ψ_g) of the gas-relief layer as the permeability times the thickness:

$$\Psi_g = k_g \cdot t \text{ (gas transmissivity of the gas relief layer)} \tag{8}$$

we can combine equations (6), (7), and (8) as:

$$\Phi_g (L - x) = \frac{\Psi_g}{\gamma_g}\frac{\mathrm{d}u_g}{\mathrm{d}x} \tag{9}$$

5. Equation (9) can be rearranged to solve for 'u' by integrating in terms of 'x' as:

$$u_x = \frac{\Phi_g \gamma_g}{\Psi_g} \int_0^x (L - x)\mathrm{d}x = \frac{\Phi_g \gamma_g}{\Psi_g}\left(Lx - \frac{x^2}{2}\right) \tag{10}$$

where 'u_x' is the gas pressure at any distance 'x' from a strip drain.

The normalized gas pressure is plotted in Figure 5 as a function of distance from the strip drain. From Figure 5, and Equations (9) and (10), we can observe the following:

- The pressure gradient, du_g/dx, varies linearly with distance x. It is a maximum at x=0 (where the gas volume is greatest), and is zero at x=L (where there is essentially no gas flow).

- The pressure varies as a polynomial function of distance. It is zero at x=0 (that is, it is at the backpressure value in the strip drain). The maximum pressure at x=L is:

$$u_{max} = \frac{\Phi_g \gamma_g}{\Psi_g}\left(\frac{L^2}{2}\right) \tag{11a}$$

or in terms of the strip-drain spacing, D:

$$u_{max} = \frac{\Phi_g \gamma_g}{\Psi_g}\left(\frac{D^2}{8}\right) \tag{11b}$$

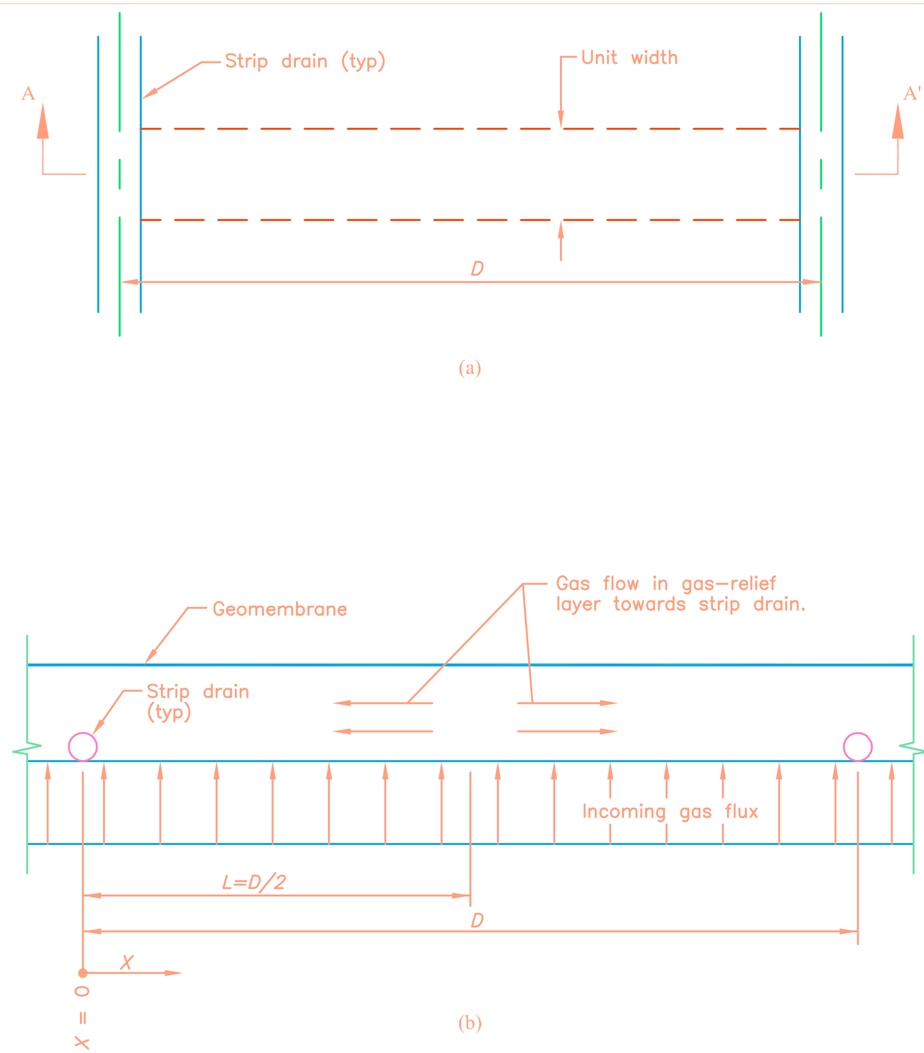


Figure 4. Model of gas flow to strip drains: (a) Plan; (b) Section A-A'

Using Equation (11b), the distance D can be written in terms of the maximum pressure as:

$$D = \sqrt{\frac{8u_{max}\Psi_g}{\Phi_g \gamma_g}} \tag{12}$$

Although Equation (12) could conservatively be used to solve for the strip-drain spacing, it is reasonable that engineering judgment may be applied to select a gas pressure less than the maximum pressure for use in the stability analysis. Since slope stability involves an area, rather than a point location, it is reasonable that the pressure at $x = L/2$ would be appropriate for the design gas pressure ($u_{g\text{-}allow}$). From Equation (10) (or Figure 4c) it can be determined that $u_{(L/2)}$=0.75(u_{max}).

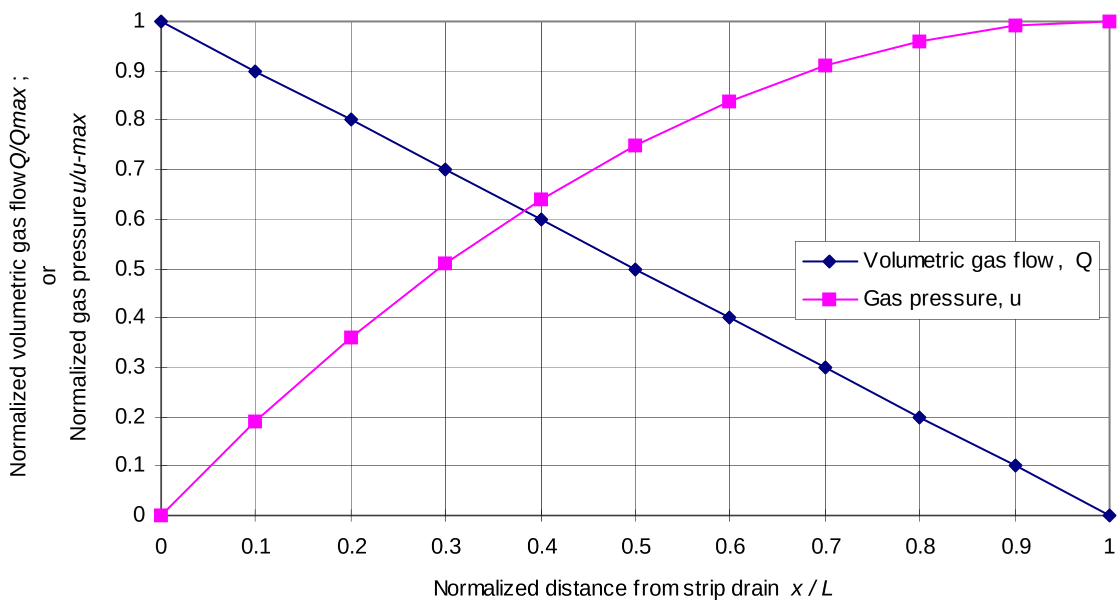


Figure 5. Normalized gas pressure and volume vs. distance from strip drain.

4.3 Backpressure Considerations

The calculation of the gas pressure gradient described above is relative to the gas pressure in the strip drains. Since a certain amount of pressure will be needed in the strip drains to cause the gas to flow from the strip drains to the vents at atmospheric pressure (unless the system is under vacuum), the pressure in the strip drains relative to the gas-relief layer is referred to as a back-pressure.

The back-pressure required in the strip drains is a function of the strip drain cross section, flow rate, and length of strip drain. For purposes of establishing a relative example, Figure 6 is provided to show the required back-pressure in a 76 mm diameter smooth pipe as a function of flow rate and length of pipe. The figure shows, for example, that 187 Pa are required to cause 0.0274 m^3/s to flow through 30 m of the selected pipe. Assuming a relatively permeable strip drain is provided, at least equivalent to 76 mm diameter pipe, it seems reasonable that a back-pressure of 250 Pa would be conservative in most cases. If the strip-drain system is hooked up to an active vacuum system, the backpressure could be a negative value (which would be beneficial to slope stability).

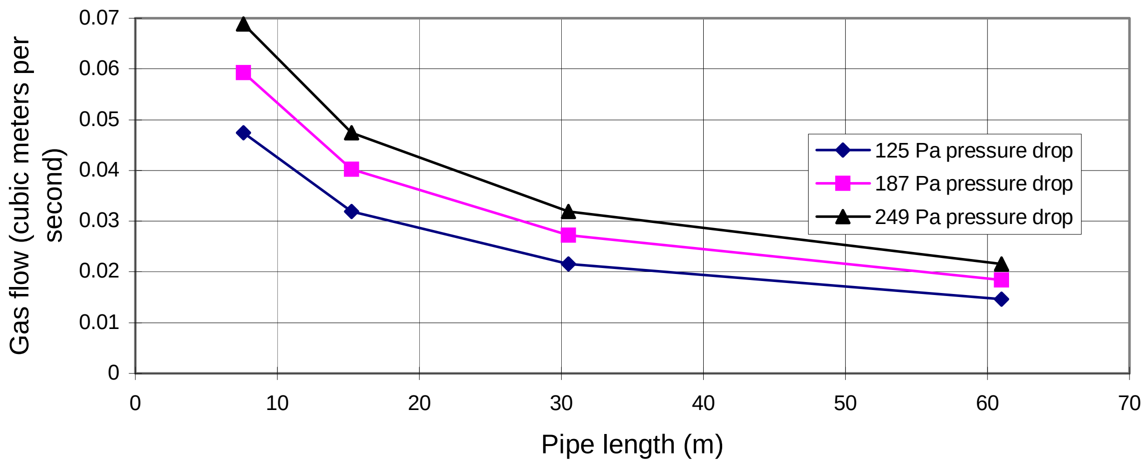


Figure 6. Flow capacity for 76 mm pipe vs. distance for various pressure drops.

5. DISCUSSION OF GAS TRANSMISSIVITY COMPARED TO WATER TRANSMISSIVITY FOR SOILS AND GEOSYNTHETICS

5.1 Intrinsic Permeability

Use of Equations (11) or (12) requires to designer to select, or back-calculate, the value of transmissivity, Ψ_g, of the gas-relief layer. However, little if any testing or manufacturer data are available regarding the gas transmissivity of soils or geosynthetics. Therefore, the design will usually have to resort to assuming or specifying an equivalent hydraulic (water) transmissivity. In theory, the gas transmissivity can easily be calculated from the water transmissivity using the concept of intrinsic permeability.

Standard civil engineering practice that deals with the seepage of water through soils utilizes a constant, herein called k_w, to represent the proportionality between the flow rate of water, Q_w, and the area (A) times the unitless head gradient, i_w, between the two ends of the flow. Darcy's law for water is written as

$$Q_w = k_w \, i_w A \tag{13}$$

This result was deduced empirically through experimentation by H. Darcy in 1856 when he studied water flow through sand filters for the city of Dijon, France. The units for k_w are length per time (L/T), and the value of k_w is characteristic to the soil tested, and is valid for water at a specified temperature. In essence, the value of k is dependent on both the medium in question (e.g. a certain soil or geosynthetic) and the properties of the fluid.

Later investigators sought to evaluate the range of validity of this formulation, and determine the physical basis for its validity. One of the results of these investigations was the ability to define an intrinsic permeability characteristic of the medium in question, and entirely independent of the nature of the fluid. (See, for example, Lambe and Whitman, 1969, pp 287-289; McWhorter and Sunada, 1977, pp 65-71; or for an excellent analytical and historical discussion Muskat, 1937). The resulting reformulation of Darcy's law is:

$$Q_f = K \cdot \frac{\gamma_f}{\mu_f} \cdot i_f \cdot A \tag{14}$$

where Q_f = the flow rate of the fluid; γ_f = unit weight of the fluid; μ_f = dynamic viscosity of the fluid; i_f = the fluid gradient, and K is the intrinsic permeability of the medium (independent of the fluid) with units of L^2. Comparing Equations (13) and (14) allows the relationship between the standard civil engineering coefficient of permeability and the intrinsic permeability to be developed as follows:

$$k_f\, i_f\, A = K \frac{\gamma_f}{\mu_f} i_f\, A \tag{15a}$$

whereby the coefficient of permeability of a medium to any given fluid, k_f , can be solved knowing the intrinsic permeability of the medium, K, and the properties of the fluid as follows:

$$k_f = K \cdot \frac{\gamma_f}{\mu_f} \tag{15b}$$

Since K is a constant independent of the fluid, the ratio between the coefficients of permeability for two different fluids (denoted by subscripts 1 and 2) can be determined as

$$\frac{k_1}{k_2} = \frac{\mu_2}{\mu_1} \cdot \frac{\gamma_1}{\gamma_2} \tag{16}$$

Using Equation (16), the design of a gas-relief layer can now be accomplished by converting the required gas transmissivity to a required hydraulic (water) permeability. All that is required are the physical properties of density and viscosity for the fluids of concern. These are easily obtained from published literature. (The important physical properties for water, air, carbon dioxide, methane, and landfill gas are presented in Appendix A.) The validity of this relationship has been experimentally proven for sand mediums having coefficients of permeability to water (k_w) ranging from 0.001 to 0.1 cm/s by Muskat (1937, Table 7 pg 93). The experiments by Muskat showed that the intrinsic permeability of this range of sands to air was identical to that measured for both water and carbon tetrachloride.

Example 3. Back-Calculation of Equivalent Water Permeability from Required Gas Permeability

Assume that through iteration of Equation (12) a designer determines that the minimum required landfill gas transmissivity for a 0.3 m thick layer of sand is 0.0003 m^2/s. What is the equivalent water coefficient of permeability, k_w?

Rearranging Equation (8) we find:

$$k_g = \frac{\Psi_g}{t} = \frac{0.0003}{0.3} = 0.001\ \frac{\text{m}}{\text{s}}$$

From Equation (16) and Appendix A:

$$k_w = k_g \cdot \frac{\mu_g}{\mu_w} \cdot \frac{\gamma_w}{\gamma_g} = 0.001\left(\frac{1.32(10)^{-5}}{1.01(10)^{-3}}\right)\left(\frac{9797}{12.8}\right) = 0.01\ \frac{\text{m}}{\text{s}}$$

END OF EXAMPLE 3

From the example we can note that the water coefficient of permeability for a soil is conveniently almost exactly 10 times the landfill-gas coefficient of permeability. (Note that the calculations presented in this paper assume that the gas is at standard temperature. Technically, the gas coefficient of permeability should be adjusted for temperature due to viscosity changes. At landfills the gas temperature is typically higher than 20^{o} C. However, the change in viscosity with temperature for gases is much less than for water. It is also interesting to note that gases behave differently from water in that the viscosity increases as the temperature increases.)

The principle of intrinsic permeability is considered valid for granular soils, and probably most geosynthetic drainage layers, but would not hold for silts and clays where the polartiy of the fluid and electro-osmotic potentials begin to have a significant influence on the measured flow rates. In the case of finer-grained soils, the coefficients of permeability must be measured on a fluid-specific basis. However, finer-grained soils would generally not be appropriate for a gas-relief layer in any case.

5.2 Gas Permeability in Partially Saturated Soils

If the gas-relief layer is a granular soil, it is reasonable to assume that the soil will be holding a certain amount of capillary water either due to rain during construction, or from condensate underneath the geomembrane. Note that condensate water will be prevalent under landfill covers due to landfill gas, which is generally saturated. Since the bottom of the gas-relief layer is not a water table (hopefully!), a sand in this application would probably be at (or possibly slightly above) its field capacity. Guidance on the field capacity for typical sands can be found in the reference documents for the HELP computer program (Schroeder et al, 1994).

The reduction in gas permeability due to partial saturation of the sand layer can be estimated using the Brooks and Corey (1964, as reported by Fredlund and Rahardjo, 1993) relationship:

$$k_g = k_d\left(1 - S_e\right)^2\left(1 - S_e^{(2+\lambda)/\lambda}\right) \tag{17}$$

where: k_g= gas coeff of permeability under given moist conditions

k_d = coeff of permeability to air for a dry soil (S=0)

λ = pore size distribution index (typical values range from 2 for porous rocks, to infinity for uniform sands)

S_e = effective degree of saturation = $\dfrac{S - S_r}{1 - S_r}$ (18)

S_r = residual degree of saturation at which point an increase in matric suction does not produce an appreciable change in the degree of saturation (S). Typical values for residual saturation are presented by Schroeder et al (1994, pg13, Figure 2).

Example 4. Air-Permeability Calculation for Moist Sand

Given the following parameters:

- sand with saturated hydraulic conductivity k_w = $6(10)^{-3}$ cm/s (actual laboratory value)
- field moisture content of sand w = 16.9% (actual laboratory value)
- dry density γ_d = 13.61 kN/m^3 (actual laboratory value)
- sand porosity n = 0.46 (value from Peck, Hanson, and Thornburn, 1974, for loose, uniform sand that approximately matches dry density used for this example)
- S_r = 0.05
- pore-size distribution index λ = 4 (value for natural sand deposits reported by by Fredlund and Rahardjo, 1993)

What is the air coefficient of permeability of the moist sand?

Calculations:

Soil degree of saturation $S = \dfrac{w}{n} \cdot \dfrac{\gamma_d}{\gamma_w} = \dfrac{0.169}{0.46} \cdot \left(\dfrac{86.7}{62.4}\right) = 0.51$ (19)

where: γ_d = dry density of soil; γ_w = density of water; w = soil moisture content; n = soil porosity

From Equation (18):

$$S_e = (0.51\text{-}0.05)/(1\text{-}0.05) = 0.484$$

Using Equation (16), and the appropriate values for density and viscosity for water and air (see Appendix A), the value of the dry-air permeability of the sand can be calculated as

$$k_d = 0.006\left(\frac{2.12(10)^{-5}}{3.78(10)^{-7}}\right)\left(\frac{.0753}{62.4}\right) = 4.06(10)^{-4} \text{ cm / s}$$

Calculate the air (gas) permeability under moist conditions from Equation (17):

$$k_g = 4.06(10)^{-4}(1-0.484)^2\left(1-0.484^{1.5}\right) = 7.2(10)^{-5} \text{ cm / s} \quad \text{(Answer)}$$

——— END OF EXAMPLE 4 ———

In the example above, which was based on laboratory data from a field-exhumed sample, the average measured gas permeability of the moist sand was $8(10)^{-5}$ cm/s, which is in good agreement with the theoretically-derived value. Note that the ratio of k_g/k_d is approximately 0.18. That is, the gas permeability of the sand was reduced by over 80% due to the presence of field moisture!

Using typical values of moisture field capacity for sands presented by Schroeder et al (1994), and going through the same calculations above, indicates that the gas permeability of a typical sand would be reduced by 25-50%. The example above, derived from actual field data, showed a considerably greater reduction because the field moisture content was greater than the static-drained field capacity. This was probably due to rains during construction, and the constant presence of moisture due to saturated landfill gas. Coarser sands will be less saturated and retain better gas permeability. Based on the range of numbers described herein, and limited field experience, the following preliminary recommendations are put forward until more data is available:

1. For fine sands containing less than 10-15 percent fines, the field-gas permeability can be taken as the dry-gas permeability reduced by a factor of 5 to10 (one-half order of magnitude to one order of magnitude) to account for the presence of field moisture.

2. For clean medium and coarse sands, the field-gas permeability can be taken as the dry-gas permeability reduced by a factor of 2 (one-half the dry value) to account for the presence of field moisture.

5.3 Validity of Darcy's Law For Typical Gas Gradients Expected Under Landfill Covers

As stated above, Darcy's law is only valid when the flow is laminar. It can be shown that the character of a flow depends on the relationship between the fluid velocity, density, viscosity, and characteristic diameter of the flow path. The relationship is expressed as the Reynolds number, *Re*, as follows:

$$Re = \frac{\rho v d}{\mu_f} \tag{20}$$

where ρ = fluid density, μ_f = fluid dynamic viscosity , v = fluid velocity, and d = characteristic flow dimension. The units must all be consistent to produce a unitless value for *Re*.

In classical fluid mechanics considering the flow of fluids in pipes, the flow is considered laminar for *Re* values less than 2000, and the characteristic dimension, *d*, is the pipe diameter (for example, see Mott, 1979). For *Re* values greater than 4000, the flow in pipes is generally considered turbulent. For *Re* values between 2000 and 4000 it is very difficult to predict which type of flow exists.

In porous media such as sands, investigators have generally taken the characteristic diameter, *d*, as a representative grain size of the soil. Physically it would seem more appropriate to have the term *d* represent an average pore size rather than grain diameter. However, direct measurement of pore sizes is very difficult, and most historic correlations of Reynolds numbers for flow through soils have referred to the averages of actual grain diameters obtained from sieve analyses. (For a discussion, see Muskat, 1937.) The results of experiments involving flow rates of various fluids (liquids and gases) through soils indicates that the critical Reynolds number below which the flow is laminar is somewhere between 1 and 10, with most references conservatively identifying a value of 1 as a safe limit for the applicability of Darcy's law. In this case, the velocity term, *v*, is the macroscopic velocity obtained by dividing the flow rate by the entire cross sectional area of the soil medium. (Note that this is not the actual velocity of the liquid in the pores.)

Given these parameters, we can now verify that the flow of landfill gas in a porous medium below a landfill cover would usually be expected to be laminar. Three examples below confirm this assumption for sand, geonet, and nonwoven-needlepunched (NWNP) gas relief layers by showing the Reynolds numbers for the assumed situations are less than the critical Reynolds numbers for laminar flow.

Example 5a. Reynolds Number for Gas Flow Through a Fine-to-Medium Sand

Given the following parameters:

- Gas flux from Example 1
- Spacing between strip drains of 30.5 m
- 0.3 m thick sand gas relief layer

•Average sand particle size of 0.5 mm

What is the Reynolds number, *Re*, and is the flow laminar?

Using the flux rate from Example 1, we calculate the maximum gas flow rate, *Q*, from the half-distance between strip drains per unit width as:

$$Q = \Phi_g \cdot \frac{D}{2} \cdot (\text{unit width}) = 0.017 \frac{\text{m}^3}{\text{hr} \cdot \text{m}^2} \cdot \frac{30.5\ \text{m}}{2} \cdot 1\ \text{m} \cdot \frac{\text{hr}}{3600\ \text{s}} = 7.2(10)^{-5}\ \frac{\text{m}^3}{\text{s}} \qquad (21)$$

The velocity is calculated as:

$$v = \frac{Q}{A} = \frac{7.63(10)^{-5}\ \text{m}^3/\text{s}}{0.3\ \text{m} \times 1\ \ \text{m}} = 2.542(10)^{-4}\ \ \frac{\text{m}}{\text{s}} \qquad (22)$$

From Eqn (20) the Reynolds number is calculated as:

$$Re = \frac{\rho v d}{\mu_f} = \frac{1.31\ \ {}^{\text{kg}}\!/\!{}_{\text{m}^3} \times 2.542(10)^{-4}\ \ {}^{\text{m}}\!/\!{}_{\text{s}} \times 0.0005\ \ \text{m}}{1.32(10)^{-5}\ \ {}^{\text{kg}}\!/\!{}_{\text{s}-\text{m}}} = 0.0126$$

Since *Re*<<1, flow is laminar.

———————————— END OF EXAMPLE 5a ————————————

Example 5b. Reynolds Number for Gas Flow Through a Geonet Composite.

Given the same parameters as in Example 5a, except that a geonet composite replaces the sand layer. Assume the effective depth of the flow path through the geonet (accounting for encroachment from geotextiles) is 1.5 mm.

What is the Reynolds number, *Re*, and is the flow laminar?

As in Example 5a, the velocity is calculated as:

$$v = \frac{Q}{A} = \frac{7.63(10)^{-5}\ \text{m}^3/\text{s}}{0.0015\ \text{m} \times 1\ \text{m}} = 0.05\ \frac{\text{m}}{\text{s}}$$

From Eqn (20) the Reynolds number is calculated as:

$$Re = \frac{\rho v d}{\mu_f} = \frac{1.31\ \ {}^{\text{kg}}\!/\!{}_{\text{m}^3} \times 0.05\ \ {}^{\text{m}}\!/\!{}_{\text{s}} \times 0.0015\ \text{m}}{1.32(10)^{-5}\ \ {}^{\text{kg}}\!/\!{}_{\text{s}-\text{m}}} = 7.4$$

In this case, since the characteristic dimension of flow was the actual height of the flow path, and the calculated velocity is probably close to the true velocity of the fluid flow, the critical Reynolds number for laminar flow is probably close to that used for pipes, which is 2000. This conclusion was also inferred by Williams et al (1984). Since 7.4 << 2000, the flow is expected to be laminar.

——————————— END OF EXAMPLE 5b ———————————

Example 5c. Reynolds Number for Gas Flow Through a Nonwoven Needlepunched (NWNP) Geotextile.

Given the same parameters as in Example 5a, except that a polypropylene NWNP geotextile replaces the sand layer. Assume the geotextile fibers are 45 denier (very coarse), and an average geotextile thickness of 3 mm. Noting that the specific gravity of polypropylene is 0.91, and that the definition of denier is grams per 9000 m of fiber, the average diameter of a fiber can be calculated as 8.36 $(10)^{-5}$ m. Assume that flow through the NWNP geotextile is analogous to flow through a soil when calculating the Reynolds number. That is, use the macroscopic average flow velocity (*Q/A*), and use the fiber diameter as the characteristic flow dimension.

What is the Reynolds number, *Re*, and is the flow laminar?

As in Example 5a, the velocity is calculated as:

$$v = \frac{Q}{A} = \frac{7.63(10)^{-5}\ \mathrm{m^3/s}}{0.003\ \mathrm{m} \times 1\ \ \mathrm{m}} = 2.54(10)^{-2}\ \ \frac{\mathrm{m}}{\mathrm{s}}$$

From Eqn (20) the Reynolds number is calculated as:

$$Re = \frac{\rho v d}{\mu_f} = \frac{1.31\ \ \mathrm{kg/m^3} \times 2.54(10)^{-2}\ \ \mathrm{m/s} \times 8.36(10)^{-5}\ \ \mathrm{m}}{1.32(10)^{-5}\ \ \mathrm{kg/s-m}} = 0.21$$

Since *Re*<1, flow is expected to be laminar.

——————————— END OF EXAMPLE 5c ———————————

The above examples 5a through 5c employed typical gas flow parameters to be expected under a landfill final cover, and a reasonably wide spacing between strip drains. The results illustrate that gas flow would generally be expected to be laminar for all types of flow mediums. Note that the calculations in these examples were independent of the pressure gradients required to create the assumed flow rates. As discussed in the section on Case Histories, the pressure gradients required to pass the assumed gas flow may be unacceptably high for some of the flow mediums.

6. DESIGN CASE HISTORIES

6.1 Final Cover Design and Gas Collection for Coffin Butte Landfill, Corvallis, Oregon

In 1996, 38,060 m^2 of final cover was installed over a portion of Cell 1 at the Coffin Butte landfill. Most of the cover occurred on a 1(V):3(H) slope. It is worthwhile noting that Cell 1 had been in place since 1977, filling had stopped since 1992, and there was an active gas collection system working in part of the cell using vertical wells during cover construction.

The landfill cover cross section consisted of the following elements, from bottom to top:

- Foundation soil over waste
- 15 cm sand gas relief layer
- 1.5 mm textured polyethylene geomembrane
- 30 cm gravel drainage layer
- geotextile filter
- 45 cm vegetative and topsoil planted with grasses and legumes

The strip-drain spacing (D) was an average of 45 m. The gas relief layer consisted of a medium, poorly-graded sand having an average grain size of 0.85 mm, and a hydraulic conductivity (k_w) of 0.03 cm/s. The friction angle between the sand and the textured geomembrane was estimated to be 30°. The assumed unit weight of the cover materials is 16.5 kN/m^3.

After the cover was constructed, gas flows were monitored from the strip drain header collection pipe. The gas was being extracted under a vacuum of approximately 124 Pa, and was an average temperature of 25°C. The maximum flow rate observed was 0.045 m^3/s. Dividing the flow rate by the area yields a gas flux, Φ_g, equal to $1.2(10)^{-6}$ $m^3/s/m^2$. Note that this gas flow is only about 23% of what might have been estimated using Example 1. However, this amount of gas is impressive considering the age of the waste in the cell (i.e. most of the gas would already have been generated and released), and the fact that much of the gas was being collected in an active collection system.

Given the relatively clean nature of the sand, the field gas permeability is assumed to be one-half the calculated dry gas permeability. Using the results from Example 3 and Equation (16),

$$k_g = \frac{1}{2}(k_d) = \frac{1}{2}\left(\frac{k_w}{10}\right) = \frac{1}{2}\left(\frac{0.03}{10}\right) = 0.0015\,\text{cm/s}$$

Given the thickness of the sand layer, the effective gas transmissivity, Ψ_g, can be calculated from Equation (8) as:

$$\Psi_g = k_g \cdot t = 0.0015\frac{\text{cm}}{\text{s}} \cdot 15\text{cm} \cdot \frac{\text{m}^2}{100^2\,\text{cm}^2} = 2.25(10)^{-6}\ \text{m}^2/\text{s}$$

Assume that the measured vacuum on the strip-drain header offsets the back-pressure in the strip drains, such that the back-pressure in the gas-relief layer next to the strip drains is zero gage pressure. Using Equation (11b), find u_{max}, as:

$$u_{max} = \left(\frac{D^2}{8}\right)\left(\frac{\Phi_g \gamma_g}{\Psi_g}\right) = \left(\frac{45^2\,\mathrm{m}^2}{8}\right)\left(\frac{1.2(10)^{-6}\ \mathrm{m}^3/\mathrm{s}/\mathrm{m}^2 \cdot\ 12.8\ \mathrm{N}/\mathrm{m}^3}{2.25(10)^{-6}\ \mathrm{m}^2/\mathrm{s}}\right) = 1728 \quad \mathrm{Pa}$$

Using Equation (4), estimate the factor of safety (FS) as:

$$FS = \frac{\left[0.9\mathrm{m}\cdot 16{,}500\mathrm{N}/\mathrm{m}^3\cdot\cos(18.4^\circ) - 1728\mathrm{N}/\mathrm{m}^2\right]\cdot\tan(30^\circ)}{0.9\mathrm{m}\cdot 16{,}500\mathrm{N}/\mathrm{m}^3\cdot\sin(18.4^\circ)} = 1.5$$

While the safety factor of 1.5 calculated above seems reassuring for the completed project, note that during construction of the drainage layer on top of the geomembrane, when the drainage layer is only 30 cm thick, the factor of safety drops to less than 1.1! One of the lessons here is to consider the effects of gas during construction, which is especially brought out in the following case history of a cover that slid during construction.

6.2 Final Cover Sliding Failure, Confidential Project

A sliding failure occurred during construction of a 6 ha final cover project. The slope on which the failure occurred was inclined at 1(V):4(H), and was 18 m high with no benches. The cover system design consisted of the following elements, from bottom to top:

- Foundation soil over waste
- 30 cm thick gas relief layer consisting of a fine sand with a measured hydraulic conductivity of 0.006 cm/s.
- Geosynthetic clay liner (GCL)
- PVC geomembrane
- 30 cm drainage layer sand
- 30 cm vegetative soil
- 15 cm topsoil

The design also included vertical gas vents on a 60 m spacing each way. The wells consisted of 45 cm borings 4.6 to 18.3 m deep, with 15 cm diameter slotted PVC pipe, backfilled with pea gravel around the pipe.

Failure occurred after the following elements had been constructed:

- gas vents
- sand gas-relief layer
- GCL
- geomembrane
- 3.2 ha had just been covered with 30-60 cm of drainage sand

The observed failure mode was the geomembrane stretching and then tearing at the top of the slope. The sand on top of the geomembrane, and the geomembrane, slid downslope along the geomembrane/GCL interface. The GCL did not appear to be distressed. However, a thin film of bentonite had extruded from the slit-film side of the GCL at the geomembrane interface.

As the failure progressed, and rain eroded portions of the top sand drainage layer, large gas bubbles formed in the geomembrane. Even the exposed GCL appeared to be uplifted by gas pressures. Subsequent installation of 12 gas probes monitored over a period of two months revealed an average gas pressure in the gas-relief layer of 1.7 kPa in the nine most critical locations. The probe with the highest pressure, averaged over 23 readings, was 3.3 kPa, and had a single high reading of 4 kPa.

Shear strength testing was conducted on the PVC geomembrane/hydrated GCL interface over a normal load range of 2.4-12 kPa. The measured Mohr-Coulomb shear strength parameters were 16° friction, and a y-intercept of 0.5 kPa. Using the moist sand unit weight of 17.3 kN/m^2, and a sand layer thickness of 30 cm, the factor of safety can be calculated from Equation (4) as:

$$FS = \frac{500\mathrm{N}/\mathrm{m}^2 + \left[0.3\mathrm{m} \cdot 17{,}300\mathrm{N}/\mathrm{m}^3 \cdot \cos(14^\circ) - 1{,}700\mathrm{N}/\mathrm{m}^3\right] \cdot \tan(16^\circ)}{0.3\mathrm{m} \cdot 17{,}300\mathrm{N}/\mathrm{m}^3 \cdot \sin(14^\circ)} = 1.16$$

This factor of safety is still greater than one. However, the factor of safety is extremely sensitive to the shear strength parameters, and assumed pore pressure. A discussion of these sensitivities is provided by Liu et al (1997). For example, simply ignoring the very small y-intercept value of the Mohr-Coulomb envelope (which is a very common practice, and in fact, is often recommended in the literature such as Koerner and Soong, 1998) reduces the factor of safety to 0.76! In addition, two other factors were surmised to drop the factor of safety to less than one:

- The gas pressure in many places was greater than the overall average gas pressure. The gas pressure required to cause the factor of safety to drop below 1.0 is 2.5 kPa. The average gas pressure at two of the probes was above 2.5 kPa.

- The rapid increase in the unit weight of the cover sand during rain events would cause an instantaneous pore water pressure increase at the interface between the geomembrane and the hydrated GCL. While the rain might saturate the sand in a matter of hours, it may take days for the excess water pressure to dissipate through the GCL. In fact, the slide movement was noted to greatly accelerate during rains, and stabilize some time after the rains stopped. The increase in unit weight of the sand going from moist to saturated is on the order of 2.4 kPa. Added to the average gas pressure, this would result in a factor of safety of approximately 1.0.

In this case history, no strip drains were provided in the gas-relief layer. We can use this opportunity, in hind sight, to calculate the improvement in factor of safety by installing strip drains. In this case we will assume that the effects of a rapid increase in unit weight of the cover from rain will increase the pore pressure by 2.4 kPa at the critical interface.

Assume the gas flux as estimated in Example 1, that is Φ_g = $5(10)^{-6}$ m^3/s/m^2. The air-permeability of the moist gas-relief layer was measured in the laboratory as $8(10)^{-5}$ cm/s. The equivalent landfill gas permeability and transmissivity can be calculated from Equations (16) and (8) as:

$$k_g = k_{air}\frac{\mu_{air}}{\mu_{gas}}\cdot\frac{\gamma_{gas}}{\gamma_{air}} = 8(10)^{-5}\frac{\text{cm}}{\text{s}}\cdot\frac{1.79(10)^{-5}\text{N}-\text{s}/\text{m}^2}{1.32(10)^{-5}\text{N}-\text{s}/\text{m}^2}\cdot\frac{12.8\text{N}/\text{m}^3}{11.8\text{N}/\text{m}^3} = 1.2(10)^{-4}\frac{\text{cm}}{\text{s}}$$

$$\Psi_g = k_g\cdot t = 1.2(10)^{-4}\frac{\text{cm}}{\text{s}}\cdot 30\text{cm} = 3.6(10)^{-3}\frac{\text{cm}^2}{\text{s}}$$

Using Equations (4) and (11b), the variation in factor of safety with strip drain spacing (*D*) is graphically presented in Figure 7. The figure indicates a variation in *FS* from 1.4 with back-to-back strip drains (i.e. no gas pressure buildup), to *FS* = 1.0 with a strip drain spacing of 8.8 m. The close strip-drain spacing required by this case history is caused by the poor transmissivity of the sand. One of the lessons learned in this case is that fine sands that may demonstrate relatively good hydraulic conductivity lose a lot of their gas permeability due to the presence of field moisture (again, see Example 4, which was taken from this case history).

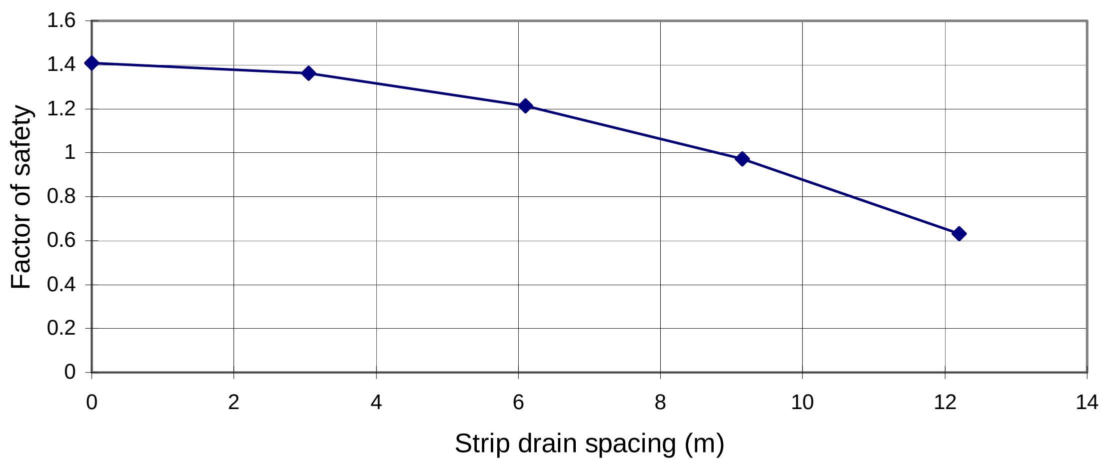


Figure 7. Solution for Case History.

6.3 Laboratory Study of Gas Transmissivity of NWNP Geotextiles

In specifying a gas-relief layer, it is tempting to consider use of a NWNP geotextile for three reasons. First, the thickness of the layer is small which would conceivably leave more room for waste (compared to, say, a 30 cm thick layer of granular material). Second, the cost to supply and install a single geotextile may be less than either a geonet-composite or a granular layer.

Third, the construction time and efficiency is most desirable for a single geotextile compared to the alternatives.

There is very limited test data available regarding the in-plane air transmissivity (or permeability) of geotextiles. Koerner et al (1984) presented their interpretation of water and air transmissivity testing using a radial flow device. However, the data interpretation appears to have been flawed in that the authors did not take into account the gradient for the air testing, and the relationship between permeability and transmissivity for the water testing does not seem to result in a realistic material thickness (the raw data was not provided in the paper). Therefore this reference is not able to be used without further evaluation of the raw data.

Weggel and Gontar (1993) used the same radial flow device as Koerner et al (1984) to study in-plane air flow through eight NWNP geotextiles. They provided a substantial amount of raw experimental data, and a relatively thorough derivation of the flow analysis. Their testing appears to have been outside of the laminar flow region as indicated by the trends in the test data (i.e. permeability decreased with increased gradient), and by calculation of the Reynolds no. (>1 per calculation method used in Example 5c). Their empirically derived relationship results in, for example, a transmissivity of a dry geotextile with a thickness of 0.37 cm (presumably a 540 g/m^2 material) of approximately $1.5(10)^{-7}$ m^2/s (this is about one-half the gas-transmissivity of the fine sand described in the failure case history).

Perhaps the most interesting data provided to the author was from a manufacturer who had testing of their products performed at an independent laboratory (personal communication, Geocomp, 1998). Again, a radial-flow device was used. Testing was performed on a suite of three NWNP polyester geotextiles, under both dry and wet conditions, all at a normal load of 47.8 kPa. The relevant test data, and author's interpretation of the data, are summarized in Table 1. Since the author was not involved with the testing, and since the test method is non-standard, no statement can be made concerning the accuracy of the results. However, assuming the relative precision of the data is high, it is interesting to note the following:

- The wet specimens lost 25-33% of their transmissivity, compared to the dry specimens.
- Going from a material with a linear density and mass per area of 6 denier[1] (dn) and 540 g/m^2, to a 45 dn/1,080 g/m^2 material, resulted in an order-of-magnitude increase in transmissivity. This great of an increase cannot be attributed simply to the doubling of the mass per unit area. Apparently the larger fiber size (45 dn vs 6 dn) has a significant influence on the in-plane flow rate. The relative intensity of needle punching for each specimen may also have an influence, but a description of these characteristics were not available to the author. These observations point to the need for a more complete decription of NWNP geotextiles when evaluating drainage characteristics than is often reported in the literature or by testing laboratories.

[1] Denier is a unit of measurement of the linear density of fibers, and is defined as the weight in grams per 9,000 m of fiber. Knowing the specific gravity of the geotextile polymer, an approximate average fiber diameter could be calculated. For example, assuming that the specific gravity of polyester is 1.3, the average fiber diameters of 6 denier, 15 denier, and 45 denier fibers would be estimated at $2.56(10)^{-5}$, $4.04(10)^{-5}$, and $7.00(10)^{-5}$ m, respectively.

- If the results are assumed to be approximately accurate, then it is interesting to note that the estimated in-plane water transmissivity of the 45 dn material has the equivalent water transmissivity of a 30 cm thick layer of sand having a permeability of $4.7(10)^{-2}$ cm/s. This is approximately equivalent to the sand layer described for the Coffin Butte case history above. Since the effects of wetting the 45 dn material appear to be less than the assumed wetting effects on sand in terms of affecting gas transmissivity, the 45 dn material may have been an appropriate design substitute for the sand as the gas-relief layer at Coffin Butte.

It is interesting to compare the air transmissivity data described above to water transmissivity test results made available by another manufacturer of a nominal 540 g/m^2 polyester material (Hoechst Celanese Corp., 1991, test results for Trevira ® 1155). The interpolated water transmissivity of this material under a 47.8 kPa normal load is $1.55(10)^{-5}$ m^2/s. This compares very well with the water transmissivity value of $1.44(10)^{-5}$ m^2/s estimated for the equivalent Geocomp 6 dn/540 g/m^2 polyester material described above. This comparison either points to the validity of the concept of intrinsic permeability and relatively good accuracy of the testing performed, or is a coincidence of errors. Only additional well documented testing will be able to answer which is true.

Table 1 - Radial Air Flow Test Results (Geocomp, 1998).

DATA FROM LAB REPORT					VALUES CALCULATED BY AUTHOR[(1)]			
Specimen	pressure drop (kPa)	Speci-men inner radius (m)	Speci-men outer radius (m)	Air flow rate (m3/s)	Assumed unit wt of air (N/m3)	Calculated transmissivity for air (m2/s)	Calculated transmissivity for water[(2)] (m2/s)	.
6 dn/540g/m2 dry	0.69	0.0635	0.1397	4.68E-04	11.8	1.00E-06	1.49E-05	
6 dn/540g/m2 dry	1.38	0.0635	0.1397	9.16E-04	11.8	9.83E-07	1.46E-05	
6 dn/540g/m2 dry	2.07	0.0635	0.1397	1.31E-03	11.8	9.36E-07	1.39E-05	
					Average	**9.74E-07**	**1.44E-05**	
6 dn/540g/m2 wet	0.69	0.0635	0.1397	3.21E-04	11.8	6.89E-07	1.02E-05	
6 dn/540g/m2 wet	1.38	0.0635	0.1397	6.00E-04	11.8	6.44E-07	9.53E-06	
6 dn/540g/m2 wet	2.07	0.0635	0.1397	8.64E-04	11.8	6.18E-07	9.15E-06	
					Average	**6.50E-07**	**9.63E-06**	ratio wet/dry=70%
15 dn/680g/m2 dry	0.69	0.0635	0.1397	1.39E-03	11.8	2.98E-06	4.41E-05	
15 dn/680g/m2 dry	1.38	0.0635	0.1397	2.56E-03	11.8	2.75E-06	4.07E-05	
15 dn/680g/m2 dry	2.07	0.0635	0.1397	3.76E-03	11.8	2.69E-06	3.98E-05	
					Average	**2.81E-06**	**4.15E-05**	

15 dn/680g/m2 wet	0.69	0.0635	0.1397	9.30E-04	11.8	2.00E-06	2.96E-05	
15 dn/680g/m2 wet	1.38	0.0635	0.1397	1.71E-03	11.8	1.84E-06	2.72E-05	
15 dn/680g/m2 wet	2.07	0.0635	0.1397	2.49E-03	11.8	1.78E-06	2.64E-05	
					Average	**1.87E-06**	**2.77E-05**	ratio wet/dry=67%
45 dn/1080g/m2 dry	0.69	0.0635	0.1397	4.52E-03	11.8	9.69E-06	1.43E-04	
45 dn/1080g/m2 wet	0.69	0.0635	0.1397	3.37E-03	11.8	7.24E-06	1.07E-04	ratio wet/dry=75%

Notes to Table 1:

(1) Data interpretation after method presented by Weggel and Gontar (1993).
(2) Water transmissivity calculated from air transmissivity per Equation (16).

CONCLUSIONS

Slope stability of landfill covers incorporating geomembrane barriers can be compromised by pore pressures caused by landfill gas. This has been demonstrated by field failures in which gas pressures appeared to play a significant role.

Standard geotechnical and fluid mechanics engineering principles can be used to design final cover systems to accommodate potential landfill gas pressures. However, as is typical with many geotechnical problems, the basic input of field parameters to the analysis (in this case an estimation of the field gas pressures and volumes) is not an exact science, and involves educated assumptions and experience.

Calculations and experimental evidence from the literature suggests that landfill gas flow rates expected in gas-relief layers are generally expected to be laminar, and Darcy’s law applies. The fluid-mechanics principle of intrinsic permeability can allow estimations of gas transmissivity and permeability to be made based on more well known, or more easily obtained, values for water.

The analytical and design approach proposed herein to account for landfill gas pressures below covers appears to have been corroborated by one successful case history, and could have potentially predicted the failure of another case history. While the case histories described were not controlled and monitored to an extent that could be said to validate the proposed approach, the case history observations and measurements that were made do not contradict it.

Limited laboratory test data suggests that coarse, heavy (e.g. 45 dn/1,080 g/m^2) NWNP geotextiles may have adequate gas transmissivity under field conditions for many typical situations. However, industry testing and design experience in this regard is sparse.

RECOMMENDATIONS AND DISCUSSION

The theoretical solution to gas flow presented in this paper is undoubtedly more developed than the profession’s ability to provide the basic input parameters to the model. To that extent, it may be found that the theoretical assumptions presented herein are incorrect when a more accurate understanding of landfill gas generation, flux, and flow mechanisms is attained. However, in lack of any other procedures available, the model presented in this paper is meant to serve as a starting point, and has been qualitatively calibrated by two field experiences - one successful cover design with gas flow measurements - and one cover failure attributed, in part, to excess gas pressures.

The key input parameter that needs more development is the assumed gas flux that might cause pressures below a landfill cover. To that end, additional gas flow measurements below installed covers, such as that described for the Coffin Butte landfill, would be useful. Gas pressure measurements, as described for the failure history, would also be very useful.

The industry is also in need of good, well documented test data for in-plane gas (typically air) transmissivity. The testing should be performed at relatively low pressure gradients representative of landfill gas collection requirements, where the flow is laminar as discussed in Example 5c. (However, higher gradient tests with non-laminar flow would be conservative in that they would result in lower transmissivity values.) The testing should be performed not only for dry geotextiles, but also on wet geotextiles at a simulated field moisture capacity obtained from soaking the geotextile and then letting it drain. When possible, it would be useful to provide side-by-side testing of air and water transmissivity in the laminar flow region to verify that the concept of intrinsic permeability can be applied to geotextiles. The geotextiles being tested should be fully described in terms of their mass per unit area, fiber size, initial thickness, and polymer type.

The model presented herein is probably conservative since many successful landfill covers have been constructed without explicit considerations for gas pressures. However, the author has witnessed several cover construction projects that, even though successful in the end product, experienced significant landfill gas problems during construction. Whether the design procedures presented in this paper are used, or some other method, the author believes that all parties involved with a landfill final cover will be well served if some degree of highly permeable strip drains and gas-relief layer are constructed below the barrier layer system.

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NOTATIONS

Basic SI units are given in parentheses.

$\bar{a}$	=	effective shear strength adhesion parameter (Pa)
A	=	cross sectional area of fluid flow (m^2)
b	=	width of representative slice in Figure 1 (m)
d	=	characteristic diameter or dimension for computing Reynolds number (m)
D	=	spacing between strip drains (m)
FS	=	factor of safety (dimensionless)
h	=	thickness of cover soils normal to slope (m)
i_f	=	fluid gradient (dimensionless)
i_w	=	water gradient (dimensionless)
k_d	=	permeability of soil or geosynthetic to dry air (m/s)
k_f / k_1 / k_2	=	permeability of soil or geosynthetic to a particular fluid / permeability of soil or geosynthetic to fluid no. 1 / permeability of soil or geosynthetic to fluid no. 2 (m/s)
k_g	=	gas permeability of soil or geosynthetic (m/s)
k_w	=	water permeability of soil or geosynthetic (m/s)
K	=	intrinsic permeability (m^2)
L	=	half-spacing between strip drains (m)
n	=	soil porosity (dimensionless)
N	=	total normal force (N)
$\overline{N}$	=	effective normal force (N)
Q / Q_x / Q_{max}	=	volumetric flow rate/volumetric flow rate at specific location ‘x’/maximum volumetric flow rate (m^3/s)
R	=	resisting shear strength (Pa)

Re	=	Reynolds number (dimensionless)
S	=	degree of saturation (dimensionless)
S_e	=	effective degree of saturation (dimensionless)
S_r	=	residual degree of saturation (dimensionless)
t	=	gas vent layer thickness (m)
T	=	tangential force (N)
u_g	=	gas pore pressure (Pa)
u_{max}	=	maximum gas pore pressure (Pa)
U	=	uplift force (N)
v	=	velocity (m/s)
w	=	soil moisture content (dimensionless)
W	=	total weight (N)
x	=	general distance parameter (m)
β	=	slope angle from horizontal
γ	=	average total unit weight of fluid or soil (N/m^3)
γ_d	=	dry unit weight of soil (N/m^3)
γ_f	=	unit weight of fluid (N/m^3)
γ_g	=	unit weight of gas (N/m^3)
γ_w	=	unit weight of water (N/m^3)
λ	=	pore size distribution index (dimensionless)
μ_f	=	dynamic (absolute) viscosity of fluid ($N\text{-}s/m^2$)
μ_g	=	dynamic (absolute) viscosity of gas ($N\text{-}s/m^2$)
μ_w	=	dynamic (absolute) viscosity of water ($N\text{-}s/m^2$)

ν	=	kinematic viscosity (m^2/s)
ρ	=	fluid density (kg/m^3)
$\bar{\sigma}$	=	effective normal stress (Pa)
$\bar{\phi}$	=	effective shear strength friction parameter (°)
Φ_g	=	gas flux from landfill surface ($m^3/s/m^2$)
Ψ_g	=	gas transmissivity of soil or geosynthetic ($m^3/s/m$)

APPENDIX A - FLUID DENSITIES AND VISCOSITIES

Fluid	Density (ρ)	Unit Weight (γ)	Dynamic Viscosity (μ)	Kinematic Viscosity ($\nu=\mu/\rho$)
	kg/m^3	N/m^3	$N\text{-}s/m^2$ or kg/(s-m)	m^2/s
Water	9.99E+02	9.80E+03	1.01E-03	1.01E-06
Air	1.20E+00	1.18E+01	1.79E-05	1.48E-05
CO2	1.83E+00	1.79E+01	1.50E-05	8.21E-06
Methane (CH_4)	6.66E-01	6.54E+00	1.10E-05	1.65E-05
LFG: 55% CO2, 45% CH_4	1.31E+00	1.28E+01	1.32E-05	1.01E-05

Notes:
(1)Values for LFG (landfill gas) were assumed to be prorated as 55% properties of carbon dioxide, and 45% properties of methane. This ratio was used to match the LFG characteristics for the Coffin Butte case history, which may be different than other landfills.
(2) Values are at standard temperature and pressure.