Use of bituminous geomembrane to reduce environmental impact of road in aquifer

J. COPPINGER, Kildare County Council, Kildare, Ireland
E.R. FARRELL, Trinity College, Dublin, Ireland
H. CHERRILL, Ove Arup, Cardiff, U.K.
G. STENSON, Colas Ireland, Dublin, Ireland
B.BREUL, Colas, Magny les Hameaux, France
F. CAQUEL, LRPC, Nancy, France
J.P. GIROUD, JP Giroud, Inc., Ocean Ridge, Florida, USA

ABSTRACT: A motorway by-pass around the town of Kildare, Ireland, which is currently being constructed, partly penetrates an aquifer in a cut section. To ensure that an environmentally sensitive fen, which is 5 km from the motorway, is not affected by drawdown of the water level, a ‘tanked system’ was devised that maintains the water level along the cut section at 1.75 m above the finished road level. This tanked system is achieved by a combination of a geomembrane and a low-permeability soil. The total surface area of geomembrane used is 203,000 m^2. A bituminous geomembrane was selected on the basis of field trials evaluating the resistance to construction damage of three geomembranes (PVC, LLDPE and bituminous) and theoretical analyses. The theoretical analyses consisted of evaluating the rate of ground-water migration through the geomembrane due to diffusion and flow through geomembrane defects. This paper describes the project, the field trials, the theoretical analyses, the construction, and the quality assurance program.

1 INTRODUCTION

1.1 *Presentation of the project*

With a surface area of geomembrane of 203,000 m^2, the project described in this paper represents, to the best of the authors' knowledge, the largest use of a bituminous geomembrane in a single project of any kind. It may also be the road project with the largest use of geomembrane of any kind.

A motorway by-pass was needed around the town of Kildare, close to Dublin, in Ireland. Kildare County Council decided that the motorway by-pass should be depressed in a cutting to minimize environmental impact, in particular to the internationally famous local equine industry. A 3.5 km section of this cutting penetrates a major aquifer. To ensure that the hydrogeological conditions at an environmentally sensitive fen, which is about 5 km from the by-pass, are not affected, ground water drawdown had to be limited both in the long term and during construction.

To limit ground water drawdown in the long term, the road is constructed within a ‘tanked system’. A drainage system located under the tank is designed to control the ground water level along the motorway to a height of 1.75 m above the finished road level. This corresponds to the seasonally low water level. However, the water level can vary by up to 1.5 m from this level. The weight of the road structure is designed to counterbalance the uplift water pressures due to the water level difference.

Ground water drawdown (achieved by deep wells and sumps) was necessary during construction to a level 5.5 m below the normal ground water level. In order to minimize the effect of the drawdown on the hydrogeological conditions at the fen, the construction is curtailed to 500 m long sections at any one time, and construction is interrupted during the winter months to allow for ground water recovery. After construction of a 500 m long section, the ground water level in the vicinity of that section is permitted to rise to 1.75 m above the finished road level, at which level it is controlled and maintained by the drainage system located under the tanked section.

1.2 *Initial design and alternative solution*

The project was initially designed with a polymeric geomembrane, but an alternative solution with a bituminous geomembrane was proposed by the general contractor, Pat Mulcair.

The cross section considered in the initial design was as follows, from top to bottom:

- a layer of compacted clay, 1.0 m thick;
- a geosynthetic clay liner (GCL);
- a polymeric geomembrane (PVC, 1.5 mm thick, or LLDPE, 2 mm thick);
- a nonwoven geotextile, with a mass per unit area of 1000 g/m²; and
- a layer of drainage material with a blinding layer.

The alternative solution was as follows, from top to bottom:

- a layer of compacted clay, 1.0 m thick;
- Coletanche NTP 3 bituminous geomembrane, 4.8 mm thick (hereafter designated as “the bituminous geomembrane”); and
- a layer of drainage material with a blinding layer.

The bituminous geomembrane comprises from top to bottom: a bitumen coating with a fine sand surface; a polyester continuous-filament needle-punched nonwoven geotextile with a mass per unit area of 300 g/m^2 and a fiber glass fleece with a mass per unit area of 50 g/m^2, both impregnated with oxidized fillerised bitumen; and a 13 μm thick polyester film bonded to the bitumen. Each component has a specific function: the sand surface is designed to provide high friction; the nonwoven geotextile supports the bitumen, and its large porosity (90%) makes it possible for the bitumen that fills the voids of the geotextile to form a continuous barrier; the fiber glass fleece is designed to provide dimensional stability; and the polyester film is designed to prevent root penetration.

The drainage material consists of crushed stone with maximum size of 40 mm. A granular material with a d_{85} of 10 mm is used as a blinding layer on top of the crushed stone to limit concentrated stresses on the geomembrane.

The clay is a local boulder clay that, in its natural state, includes stones up to 300 mm size. This material is screened to remove particles greater than 37.5 mm for the first lift of 200 mm and particles greater than 100 mm for the next two lifts of 200 mm each. This material is not screened for the last 400 mm. The water content limits of the clay and the level of compaction were selected to: (i) ensure intimacy of contact between the geomembrane and the clay; and (ii) achieve a unit weight in excess of 21 kN/m^3. The design was based on a hydraulic conductivity of 10^{-8} m/s, although this was controlled indirectly in the specification and on site through grading limits and soil condition.

1.3 *Organization of the paper*

The selection process of the geomembrane included field trials and theoretical analyses. The remainder of this paper describes the field trials and the theoretical analyses, and briefly presents the construction and the quality assurance program.

2 FIELD TRIALS

2.1 *Overview*

Pre-tender field trials were conducted on the polymeric geomembranes (LLDPE and PVC). Post-tender field trials were conducted on the bituminous geomembrane after the alternative solution had been proposed. The purpose of the field trials was to confirm that the boulder clay could be compacted over the geomembrane without damage to the geomembrane while being sufficiently compacted to achieve: (i) a unit weight in excess of 21 kN/m^3; (ii) a hydraulic conductivity lower than 1×10^{-8} m/s; and (iii) intimate contact with the geomembrane. These three parameters were measured as follows: (i) the unit weight of the compacted boulder clay was measured at the field trial site using a nuclear density gauge in direct transmission mode and was checked in the laboratory on undisturbed samples; (ii) the hydraulic conductivity of the compacted boulder clay was measured in laboratory on samples taken from the field trials site and on samples of the material recompacted within the specified range of water contents; and (iii) contact intimacy between compacted clay and geomembrane was checked visually, after carefully removing blocks of compacted clay. The last two parameters are essential because they govern the efficiency of a composite liner, i.e. the ability of the composite liner to control the rate of leakage of water through defects in the geomembrane. Indeed, as shown by Giroud and Bonaparte (1989b), the efficiency of a composite liner that consists of a geomembrane and a mineral component, such as a layer of compacted soil, is determined by the hydraulic conductivity of the soil in contact with the geomembrane and the intimacy of contact between this soil and the geomembrane.

Different field trials were performed with different values of the thickness of the lower lift of boulder clay and different values of the maximum boulder clay particle size.

2.2 *Field trials of the polymeric geomembranes*

The field trials with polymeric geomembranes were carried out without GCL because the use of a GCL was decided only after, and as a result of, the field trials.

The field trials showed that, to prevent damage to the LLDPE or PVC geomembranes, the following measures were necessary:

- A protection geotextile with a mass per unit area of 1000 g/m² was required between the drainage stone and the geomembrane.
- The maximum particle size of the boulder clay had to be limited to 37.5 mm.
- The first lift of boulder clay was required to be 350 mm thick and compacted with a non-vibratory roller having a maximum mass of 4000 kg per meter width.

This last measure, which consisted of limiting the compaction effort, did not prevent the boulder clay unit weight from meeting the specified value intended to prevent uplift of the road structure. However, this limitation of the compaction effort was of concern because it affected the ability to achieve the following two characteristics: (i) a low hydraulic conductivity of the boulder clay in the vicinity of the geomembrane; and (ii) intimate contact between the boulder clay and the geomembrane. Deficiencies in these two characteristics would impair the performance of the composite liner for the reasons given in Section 2.1.

In order to address concerns over the effectiveness of the compacted boulder clay as the mineral component of the composite liner, a GCL was introduced into the design over the geomembrane. The GCL was then to act as the mineral component of the composite liner as well as providing protection to the geomembrane. This design was considered to be effective in terms of its ability to act as a low permeability barrier and robust in terms of resistance to installation damage. This was the design that was tendered in April 2000.

In summary, the risk of mechanical damage of the LLDPE and PVC geomembranes was minimized by limiting compaction, which made it impossible for the compacted clay to have a hydraulic conductivity sufficiently low to form an effective composite liner with the geomembrane, hence the need for the GCL.

2.3 *Field trials of the bituminous geomembranes*

In the field trials with the bituminous geomembrane, a layer of about 200 mm of clay was compacted on top of the geomembrane with at least 8 passes of a vibratory compactor Bomag BW 213 D-3. This compactor has a mass of approximately 3215 kg per meter width of roller and can be used with or without vibrations.

After compaction, the clay covering the geomembrane was carefully removed. Then the geomembrane was removed for visual inspection and laboratory testing (200 mm diameter burst test). Based on the behavior of the geomembrane in the field trials and the laboratory tests, the following conclusions were reached:

- If a drainage aggregate with particles not greater than 40 mm was used, together with the blinding layer, the protection geotextile beneath the geomembrane could be omitted.
- Boulder clay with a maximum particle size of 100 mm could be compacted in a 200 mm layer over the bituminous geomembrane with vibratory roller having a mass less than 3215 kg per meter width without damage to the geomembrane. However, as a further precaution, the maximum stone size permitted within the initial 200 mm lift is 37.5 mm.
- Based on laboratory tests that gave a hydraulic conductivity of between 5×10^{-10} m/s and 1.5×10^{-9} m/s (for core and recompacted samples of boulder clay, compacted in the laboratory within the working range of water contents) and bulk unit weights in excess of 22 kN/m^3, it was concluded that a hydraulic conductivity of 5×10^{-9} m/s or less can be achieved for the boulder clay directly overlying the bituminous geomembrane using the compaction methodology outlined above without damage to the geomembrane.
- Compaction in 200 mm thick layers using the vibratory roller gave a very good intimacy of contact between the boulder clay and the geomembrane. The underside of the compacted boulder clay contained very few voids and the fine grained material showed the imprint of the rough surface of the geomembrane, thus indicating intimate contact.

In summary, the bituminous geomembrane survived the field trials without protection, even though sufficient compaction was used to ensure: (i) high unit weight and low permeability of the compacted clay overlying the geomembrane; and (ii) intimate contact between the geomembrane and the overlying clay.

3 THEORETICAL ANALYSES

3.1 *Purpose of the theoretical analyses*

The theoretical analyses consisted of evaluating the rate of ground water migration through the geomembrane. Two water migration mechanisms were considered.

3.2 *Water migration mechanisms*

The two water migration mechanisms considered were: water migration through intact geomembrane and water migration through geomembrane defects.

3.2.1 *Water migration through intact geomembrane*

Two approaches can be considered to evaluate water migration through intact geomembrane: advective flow and diffusion.

Advective flow is the water migration mechanism that is most familiar to civil engineers because it is the mechanism of water flow through porous media such as soils. Advective flow is governed by Darcy's equation and the effectiveness of a liner (such as a clay liner) in controlling advective flow is evaluated by the hydraulic conductivity, *k*, also called coefficient of permeability. The smaller the hydraulic conductivity, the greater the ability of the liner to control advective flow. Polymeric and bituminous geomembranes are not porous media and water migration through geomembranes is not due to advective flow.

As indicated by Rowe (1998), “the only significant mechanism for water migration through the geomembrane is diffusion”. Accordingly, in Section 3.3.1, diffusion calculations will be performed. However, in the early days of geomembrane use (1970s, and even 1980s), since civil engineers were more familiar with advective flow than diffusion, geomembranes were often characterized using an equivalent hydraulic conductivity. This approach will be used in Section 3.3.2, but, only for comparative purposes. It is important to note that the two mechanisms of diffusion and advective flow are different; advective flow is driven by pressure gradient whereas diffusion is driven by concentration gradient.

3.2.2 *Water migration through geomembrane defects*

In the field, geomembranes may have defects and, as a result, water can flow through the defects. If the geomembrane were both overlain and underlain by very permeable media, the migration of water through a defect in the geomembrane would be described and quantified by the theory of free flow through an orifice. However, in general, there is a low-permeability medium on one side of the geomembrane, generally the downstream side with respect to the flow direction. The system formed by the geomembrane and the adjacent low-permeability medium is called a “composite liner”. As described by Giroud and Bonaparte (1989b), water migration through composite liners includes: (i) flow through the geomembrane defect; (ii) interface flow, i.e. flow in the space between the geomembrane and the low-permeability medium; and (iii) advective flow in the low-permeability medium. As a result, the parameters that govern water migration through defects in a geomembrane associated with a low-permeability medium are: (i) the number and size of defects; (ii) the quality of contact between the geomembrane and the low-permeability medium; and (iii) the hydraulic conductivity and thickness of the low-permeability medium. Therefore, to minimize water migration through defects in geomembranes, the following should be done:

- the number and size of defects should be minimized;
- the contact between the geomembrane and the low-permeability medium should be as intimate as possible; and
- the hydraulic conductivity of the low-permeability medium should be low and its thickness should be as large as practically possible.

3.3 *Evaluation of water migration through intact geomembrane*

3.3.1 *Diffusion*

As indicated by Rowe (1998), the mass flux of water through a geomembrane is given by the following equation:

$$f = S\,D\,\frac{\Delta C}{t} \qquad (1)$$

where: f = mass flux due to water diffusion through intact geomembrane; S = solubility factor; D = diffusion coefficient; ΔC = difference in water concentration between the two sides of the geomembrane; and t = geomembrane thickness. Basic SI units are: f (kg m^2 s^{-1}), D (m^2 s^{-1}), ΔC (kg m^{-3}), and t (m); S is dimensionless.

The volumetric flux can be derived from Equation 1 as follows:

$$q_{diff} = \frac{f}{\rho} = S\,D\,\frac{\Delta C}{\rho t} \qquad (2)$$

where: q_{diff} = volumetric flux due to water diffusion through intact geomembrane; and ρ = water density. Basic SI units are: q_{diff} (m s^{-1}), and ρ (kg m^{-3}).

Practical information on diffusion of water through geomembranes, based on work performed by Eloy-Giorni et al. (1996), is provided by Rowe (1998). In particular, as indicated by Rowe (1998), the following values can be considered for the diffusion coefficient for water migration:

$D \approx 2.9 \times 10^{-13}$ m²/s for HDPE geomembranes
$D \approx 4.4 \times 10^{-13}$ m²/s for PVC geomembranes
$D \approx 8.0 \times 10^{-13}$ m²/s for bituminous geomembranes

(It should be noted that data for HDPE geomembranes are used here since no data are readily available for LLDPE geomembranes. The diffusion coefficient may be expected to be larger for LLDPE geomembranes than for HDPE geomembranes.)

Also, as indicated by Rowe (1998), the following values can be considered for the solubility coefficient for water migration:

$S \approx 8 \times 10^{-4}$ for HDPE geomembranes
$S \approx 7 \times 10^{-2}$ for PVC geomembranes
$S \approx 9 \times 10^{-3}$ for bituminous geomembranes

Based on these values of the diffusion coefficient and the solubility coefficient, the following rates of water diffusion were calculated using Equation 2:

- $q_{diff} = 1.16 \times 10^{-13}$ m/s = 0.004 mm/year for a 2 mm thick HDPE geomembrane
- $q_{diff} = 2.05 \times 10^{-11}$ m/s = 0.65 mm/year for a 1.5 mm thick PVC geomembrane
- $q_{diff} = 1.50 \times 10^{-12}$ m/s = 0.05 mm/year for a 4.8 mm thick bituminous geomembrane

(Note: 1 mm/year = 1 liter/m² per year)

The above values are for the case where the upper surface of the geomembrane is absolutely dry. In reality, the upper surface of the geomembrane cannot be absolutely dry. In the actual field situation, it is expected that, in general, there will be water standing on the upper surface of the geomembrane. In this case, the concentration gradient between the lower surface and the upper surface of the geomembrane is zero. As a result, the water diffusion rate would be zero.

For the sake of conservativeness, it is assumed herein that water might not be standing permanently on the upper surface of the geomembrane. It is assumed that, as a result of water standing at some periods of time and the presence of humidity the rest of time, the above values would be decreased by a factor of 10 or more. Therefore, the following maximum values can be considered for the rate of water diffusion through the geomembranes:

- $q_{diff} = 1.16 \times 10^{-14}$ m/s = 0.0004 mm/year for a 2 mm thick HDPE geomembrane
- $q_{diff} = 2.05 \times 10^{-12}$ m/s = 0.065 mm/year for a 1.5 mm thick PVC geomembrane
- $q_{diff} = 1.50 \times 10^{-13}$ m/s = 0.005 mm/year for a 4.8 mm thick bituminous geomembrane

The above values indicate a lesser rate of water diffusion for HDPE geomembranes than for bituminous geomembranes, which is an advantage for HDPE geomembranes, but may not be an advantage for LLDPE geomembranes because the rate of water diffusion is likely to be greater for LLDPE geomembranes than for HDPE geomembranes. Also, the rate of water diffusion through intact geomembranes is negligible compared to the rate of water migration through geomembrane defects, as indicated in Sections 3.4.3 and 3.4.5. Therefore, even if a geomembrane had an advantage compared to another geomembrane regarding water diffusion, this advantage would have no practical impact on the comparison between the two geomembranes when water migration through geomembrane defects is considered.

3.3.2 *Conventional calculation considering advective flow*

As noted in Section 3.2.1, the rate of water migration through intact geomembranes is often calculated using an equivalent hydraulic conductivity with Darcy’s equation:

$$q_{adv} = k\,\frac{h+t}{t} \qquad (3)$$

where: q_{adv} = volumetric flux due to assumed advective flow through intact geomembrane; k = equivalent hydraulic conductivity; h = water head; and t = geomembrane thickness. Basic SI units are: q_{adv} (m s^{-1}), k (m s^{-1}), h (m), and t (m).

The following values of the equivalent hydraulic conductivity are sometimes considered:

$k \approx 10^{-15}$ m/s for LLDPE geomembranes
$k \approx 10^{-13}$ m/s for PVC geomembranes
$k \approx 10^{-14}$ m/s for bituminous geomembranes

Using Darcy's equation (i.e. Equation 3) with the above values of hydraulic conductivity and a water head of 3.5 m gives the following values for the equivalent advective flow rate:

- $q_{adv} \approx 1.75 \times 10^{-12}$ m/s = 0.06 mm/year for a 2 mm thick LLDPE geomembrane
- $q_{adv} \approx 2.33 \times 10^{-10}$ m/s = 7.36 mm/year for a 1.5 mm thick PVC geomembrane
- $q_{adv} \approx 7.29 \times 10^{-12}$ m/s = 0.23 mm/year for a 4.8 mm thick bituminous geomembrane

Comparing the above values to the values calculated for diffusion in Section 3.3.1 shows that using advective flow as an "equivalent representation" of diffusion tends to overestimate the rate of water migration when the above values of water head and equivalent hydraulic conductivity are used. However, the relative water migration rates for the three considered geomembranes are similar.

It is important to note that calculations based on advective flow should only be regarded as "equivalent calculations" meant to evaluate in a conventional way what is, in reality, water diffusion. Therefore, the above water migration rates calculated for advective flow should not be added to water migration rates obtained for diffusion, because calculations performed for advective flow do not represent an additional mechanism of water migration.

3.4 *Evaluation of water migration through geomembrane defects*

3.4.1 *Method and calculations*

The method used to calculate the rate of water migration through geomembrane defects is the method initially developed by Giroud and Bonaparte (1989b) and improved by Giroud and coworkers (Giroud et al. 1989, 1992, 1994), and Giroud (1997). This method has been used for the design of numerous landfills worldwide. The following equation from Giroud (1997) was used:

$$Q_{def} = 0.21\left[1 + 0.1\left(h/t\right)^{0.95}\right] a^{0.1}\, h^{0.9}\, k^{0.74} \tag{4}$$

where: Q_{def} = rate of flow through geomembrane defect; and a = defect area. Equation 4 can only be used with the following units: Q_{def} (m^3 s^{-1}), a (m^2), h (m), and k (m s^{-1}). All calculations were performed for a water head of 3.5 m.

The following values were obtained using Equation 4 for one hole with a diameter of 2 mm (as typically considered in landfill design, a defect size adopted for this project):

- $Q_{def} \approx 1.45 \times 10^{-7}$ m³/s, in the case of the initial design where an LLDPE or a PVC geomembrane is overlain by a GCL. The GCL was assumed to have a typical thickness of 8 mm in the hydrated state and a typical hydraulic conductivity of 5×10^{-11} m/s (the maximum specified value). The hydraulic conductivity of the compacted clay overlying the GCL was not considered because this clay has a negligible impact on water migration rate since its hydraulic conductivity is significantly greater than the hydraulic conductivity of the GCL.
- $Q_{def} \approx 2.02 \times 10^{-7}$ m³/s, in the case of the alternative solution where a bituminous geomembrane is overlain by a 0.6 m thick layer of clay with a hydraulic conductivity of 5×10^{-9} m/s. This hydraulic conductivity can be achieved at the site, as demonstrated by the results of permeability tests conducted on samples from the field trials (Section 2.3). The clay thickness considered in the calculations is 600 mm, not 1 m, because it is assumed that the specified value of the hydraulic conductivity of 5×10^{-9} m/s is only met by the first three lifts of compacted clay, i.e. the lifts that consist of screened material, as indicated in Section 1.2.

For the case of one defect with a surface area of 1 cm² (a value typically used for an upper boundary of leakage evaluation in landfill design), the following values were calculated using Equation 4:

- $Q_{def} \approx 2.05 \times 10^{-7}$ m³/s, in the case of the initial design where an LLDPE or a PVC geomembrane is overlain by a GCL.
- $Q_{def} \approx 2.85 \times 10^{-7}$ m³/s, in the case of the alternative solution where a bituminous geomembrane is overlain by a 0.6 m thick layer of clay with a hydraulic conductivity of 5×10^{-9} m/s.

It should be noted that Equation 4, used for the above calculations, is the equation for "good contact" conditions, as defined by Giroud (1997). Contact quality is further discussed in Section 3.4.6.

The volumetric flux (i.e. flow rate per unit area) can be derived from the rate of flow through a defect using the following equation:

$$q_{def} = \frac{Q_{def}}{F} \tag{5}$$

where: q_{def} = volumetric flux due to water flowing through geomembrane defects; Q_{def} = rate of flow through geomembrane defect; and F = defect frequency. Basic SI units are: q_{def} (m s^{-1}), Q_{def} (m^3 s^{-1}), and F (m^{-2}).

In the case of the Kildare by-pass, the number of defects used in the calculations was estimated based on published data and results of the field trials, as discussed in the following sections.

3.4.2 *Data on geomembrane defect frequency*

An analysis of the equations for water migration (Giroud 1997) shows that the number of defects has more influence on the water migration rate than the size of defects. Therefore, it is of utmost importance to minimize the number of geomembrane defects.

Geomembranes manufactured in accordance with modern standards (e.g. ISO 9002) have no manufacturing defects. Defects are caused in the field: (i) during geomembrane installation; (ii) during placement of materials overlying the geomembrane; and (iii) during operation of the geomembrane-lined facility. The following is a quotation from Giroud (2000, page 90):

"Based, in particular, on results of electric leak detection and location surveys [. . .], the author [. . .] has established the following approximate statistics for geomembrane defects in geomembrane liners that do not exhibit any failure mode other than localized defects:

25% of the detected leaks are due to installation problems (including 20% inadequate seams and 5% mechanical damage);

70% of the detected leaks are due to mechanical damage caused during placement of the overlying soil; and

5% of the detected leaks are due to problems that occurred during operations."

It is important to note that the above statistics were based essentially on polymeric geomembranes used in landfills (i.e. essentially HDPE geomembranes). It should be noted that the above quotation is based on landfill construction, where no compaction or only limited compaction is used for the materials overlying the geomembrane. Even more mechanical damage of the geomembranes would have been observed in the surveyed landfills if the materials overlying the geomembranes had been compacted in accordance with typical road specifications

Regarding the number of defects to be considered at the design stage, Giroud and Bonaparte (1989a) recommended that one to two defects per acre (2.5 to 5 defects per hectare) be considered for cases where a strict construction quality assurance program is implemented. This recommendation is only for defects that occur during geomembrane installation. Herein, a value of 5 defects per hectare is considered. (It should be noted that the selection of the number of defects that occur during geomembrane installation, e.g. 2.5 or 5 or any other number does not affect the comparison between solutions presented hereafter, because this comparison is based on proportions derived from the above statistics and not on absolute numbers.) Based on the number of defects that occur during geomembrane installation (i.e. 5) and the statistics quoted above, the following number of defects during placement of materials overlying the geomembrane is derived: 14, i.e. 5 × 70% / 25%. Therefore, the following numbers of defects can be considered for polymeric geomembranes used in landfills:

- 4 defects per hectare due to inadequate seams;
- 1 defect per hectare due to mechanical damage during installation; and
- 14 defects per hectare due to mechanical damage during placement of the overlying material.

(hence a total number of 19 defects per hectare, which is consistent with typical results of leak detection surveys performed after placement of material overlying geomembranes)

3.4.3 *Discussion of potential geomembrane defect frequency at Kildare by-pass*

In the case of the initial design solution, the number of defects that occur during geomembrane installation could have been expected to be the same as indicated above, since the geomembrane is polymeric, i.e. similar to the geomembranes used in the statistics presented above. In other words, the number of defects that occur during geomembrane installation could have been expected to be 5 (i.e. 4 defects per hectare due to inadequate seams, and 1 defect per hectare due to mechanical damage during installation). In contrast, the number of defects due to mechanical damage during placement of the overlying material would have been expected to be much less than the value of 14 mentioned above, because, in the initial design solution, the polymeric geomembrane is better protected than the polymeric geomembranes used in landfills. Considering that the field trials have demonstrated that, in the case of the initial design, the protection of the polymeric geomembrane by the underlying geotextile is effective, and considering that additional protection is provided by the GCL, a small value, such as 2 defects per hectare due to placement of overlying materials, was considered in the evaluation of the solution. It should be noted that a frequency of 2 defects per hectare corresponds to 0.04 defect in a 200 m^2 field trial and is, therefore, not in contradiction with the fact that no defect was observed in the field trials. Clearly, field trials provide valuable information on construction damage, but they do not provide statistical data. It should also be noted that the landfill construction statistics on which the evaluation of the number of defects is based do not include potential damage caused to the geomembrane by the crew placing a GCL on top of the geomembrane. However, it was deemed that this activity is not likely to cause significant damage to the geomembrane.

In the case of the alternative solution (bituminous geomembrane), the number of defects that occur during geomembrane installation can be expected to be significantly less than in the case of the initial design solution, for the following reasons: (i) seaming bituminous geomembranes is more reliable than seaming polymeric geomembranes because, in particular, it is less sensitive to seaming temperature control and to weather conditions (and, also, because it is less prone to mistakes due to the large width of the seams, i.e. 0.20 m); and (ii) a sturdy bituminous geomembrane, such as the one used at Kildare by-pass, is less likely than PVC or LLDPE geomembranes to suffer from mechanical damage during installation. It was, therefore, considered that the following numbers of defects are appropriate for this design: 1 per hectare for seaming and zero for mechanical damage during installation. Regarding the number of defects due to mechanical damage during placement of the overlying material, a number smaller than the number recommended above for the polymeric geomembranes used in the initial design solution was considered appropriate, based on the sturdiness of the bituminous geomembrane. However, since the site-specific field trials passed with success by the initial design solution (with polymeric geomembrane) are similar to the site-specific field trials passed with success by the alternative solution (with bituminous geomembrane), the same value is proposed for the numbers of defects due to mechanical damage during placement of the overlying material: 2 per hectare.

In summary, the following numbers of defects were considered appropriate: 7 per hectare for the initial design solution (polymeric geomembranes) and 3 per hectare for the alternative solution (bituminous geomembrane). It is important to note that these numbers are based on an analysis of available data and are not an arbitrary recommendation based solely on experience. Furthermore, it is important to note that these numbers are conservative, i.e. they do not tend to favor the alternative solution, which ensures that the selection of the alternative solution is fully justified.

3.4.4 *Water migration evaluation as a function of the number of geomembrane defects*

Based on the water migration rates per defect given in Section 3.4.1 and the numbers of defects indicated in Section 3.4.3, the following water migration rates per unit area are derived for water migration through geomembrane defects:

- $q_{def} \approx 1.02 \times 10^{-10}$ to 1.44×10^{-10} m/s = 3.2 to 4.5 mm/year for the initial design (with a polymeric geomembrane, i.e. LLDPE or PVC geomembrane) with 7 defects per hectare
- $q_{def} \approx 6.06 \times 10^{-11}$ to 8.55×10^{-11} m/s = 1.9 to 2.7 mm/year for the alternative solution (with a bituminous geomembrane) with 3 defects per hectare

3.4.5 *Comparison between water diffusion through intact geomembrane and water migration through geomembrane defects*

Comparing the above values to the water diffusion rates presented in Section 3.3.1 (and even to the conventional calculations based upon equivalent advective flow presented in Section 3.3.2), shows that the rate of water diffusion through intact geomembrane is negligible compared with the rate of water migration through geomembrane defects. Therefore, the following rates of water migration can be considered:

- 4.5 mm/year for the initial design solution (LLDPE or PVC geomembrane); and
- 2.7 mm/year for the alternative solution.

From a physical standpoint, it is important to note the following: (i) to control water diffusion, the geomembrane works by itself; whereas (ii) to control water migration through geomembrane defects, the geomembrane works in association with the adjacent low-permeability medium and the intimacy of contact between the geomembrane and the low-permeability medium is very important in this respect.

3.4.6 *Effect of the quality of contact between the geomembrane and the overlying material*

As indicated in Section 3.2.2, the contact between the geomembrane and the adjacent low-permeability medium should be as intimate as possible. In the case of landfills, the low-permeability medium is located beneath the geomembrane. In contrast, in the case of the Kildare by-pass, the low-permeability medium is located on top of the geomembrane because, in this case, the composite liner is a barrier against upward migration of water.

In the case of the alternative solution where the bituminous geomembrane is overlain by low-permeability boulder clay, intimate contact is ensured by the fact that the boulder clay can be well compacted due to the sturdiness of the bituminous geomembrane. The feasibility of boulder clay compaction and the resulting intimate contact between the bituminous geomembrane and the clay were demonstrated by the field trials conducted specifically for this project (Section 2.3). It may be concluded that the contact quality in the case of the alternative solution with bituminous geomembrane is better than the “good contact” conditions considered in the calculations performed in Section 3.4. Therefore, the calculations performed in Section 3.4 for the bituminous geomembrane can be considered conservative.

In the case of the initial design, the low-permeability medium in contact with the geomembrane is a GCL. Due to the migration of some bentonite particles through the fabric component of the GCL, good contact conditions may be expected to exist between a hydrated GCL and a geomembrane, at least at places where the geomembrane is not affected by wrinkling. An LLDPE geomembrane is more likely to exhibit wrinkles than a PVC or a bituminous geomembrane, for the reasons presented by Giroud and Morel (1992).

Based on the above discussion and the results of field trials, it may be concluded that, in spite of the beneficial effect of bentonite on contact quality, contact may be less intimate in the case of the initial design solution with LLDPE than in the case of the alternative solution with bituminous geomembrane. However, for the sake of conservativeness in the comparison of the two solutions (and considering that wrinkling is usually negligible with PVC geomembranes), calculations for all geomembranes were performed using the same "good contact" conditions.

4 PROJECT IMPLEMENTATION

4.1 *Geomembrane selection*

The field trials and theoretical analyses showed that all three geomembranes could provide satisfactory performance: the PVC and LLDPE geomembranes protected using geotextile and GCL, and using the GCL as the mineral component of the composite liner; and the bituminous geomembrane without protection and using the compacted boulder clay as the mineral component of the composite liner. The bituminous geomembrane alternative solution was selected for the following reasons: installing one geosynthetic (the bituminous geomembrane) is easier than installing three (geotextile, polymeric geomembrane, and GCL); a bituminous geomembrane is less sensitive to wind uplift during installation than lighter geosynthetics; installation of a GCL is sensitive to rain; the bituminous geomembrane is very sturdy and resistant to damage caused by construction activities; seaming bituminous geomembranes is easy and not overly sensitive to damp conditions, and repairs can be made easily; and bituminous geomembranes can be tightly connected to appurtenant structures because they are bonded to concrete over the entire area in contact, an important consideration for the Kildare by-pass due to the large number of bridges. Also, based on available information, it was concluded that the design life of the bituminous geomembrane (as well as the design life of the LLDPE geomembrane) could be expected to exceed the design life of the roadway.

4.2 *Important design detail*

The median strip of the road includes a large-diameter concrete conduit. Instead of being connected to the concrete on each side of this conduit, as initially designed, the bituminous geomembrane extends under the conduit. Even though bituminous geomembranes are easy to connect to concrete structures, it is always preferable to select a configuration where the geomembrane is continuous and concrete structures are entirely contained within the waterproofed area.

4.3 *Construction*

Construction started in October 2001, using bituminous geomembrane rolls 80 m by 5.15 m. Construction was interrupted in early 2002 to allow recovery of general ground water levels, after completion of a 500 m long section of the roadway. During this period, 30,000 m^2 of bituminous geomembrane were installed. It is expected that the rate of installation could be as high as 2000 m^2 per day in 2002, as a result of the experience gained in 2001.

4.4 *Construction quality assurance*

The composite liner is being constructed with strict construction quality assurance. Both the geomembrane and the welding crew are certified. Since there is no certification agency for geomembranes in Ireland, the geomembrane manufacturer/installer, Colas, selected Asqual, a French certification organization with experience and recognition at the European level.

Quality control is provided by a Colas Ireland engineer full time on site. Construction quality assurance is provided by Golder Associates reporting directly to the Kildare County Engineer.

For conformance testing of the bituminous geomembrane, two samples are taken on every lot of 12 rolls. Tests are performed by an independent laboratory (Apave in Lyon, France) to check that the characteristics of the geomembrane meet the set of characteristics approved by Asqual.

Seaming quality is checked in several ways: (i) visually; (ii) nondestructively by ultrasonic testing, or vacuum box in areas not accessible to ultrasonic testing equipment (e.g. connection of geomembrane to concrete structures); and (iii) destructively by cutting out seam samples, which are tested for shear strength by the independent laboratory.

5 CONCLUSION

Design engineers working on projects requiring the use of a geomembrane often limit their consideration of options to polymeric geomembranes, perhaps on the basis of their experience with landfill design. This paper shows that, when a rational approach based on theoretical analyses and field trials, is used to compare candidate geomembranes, there are cases where a solution incorporating a bituminous geomembrane is a viable alternative to a solution incorporating a polymeric geomembrane. This should encourage design engineers to consider a broad range of geomembranes in their designs and to use a rational approach for geomembrane selection.

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