Innovative use of geosynthetics in landfill construction at Sutton Wick, Oxfordshire

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ABSTRACT: Geosynthetic materials are now commonly used in landfills for many applications for example geomembranes used as primary liners as barriers to leachate and landfill gas escape, geotextiles used as separation layers, filter layers and geomembrane protectors and geonets and geocomposites used as leachate, landfill gas and groundwater drainage layers. This paper concerns a case history of a landfill with a single clay liner and describes the design and construction details of the geosynthetics used in uncommon applications. A geomembrane was placed beneath the single clay liner instead of the more traditional composite lining system and a separator geomembrane was used instead of a separator geotextile. In addition, a geocomposite drainage system was designed to replace a granular leachate drainage system and the benefits gained from this application are presented.

1 INTRODUCTION

Geosynthetics are used extensively in landfill sites in the UK with geomembranes typically used as the primary liner, and geotextiles used as protection to the geomembrane and as separator layers. Landfill design is governed by a site specific risk assessment with the emphasis on the impact of any leachate leakage on the groundwater. The choice of lining system is made as part of this risk assessment process. This paper concerns a case history of a landfill with a single clay liner and describes the design and construction details of the geosynthetics used.

2 SITE DESIGN

2.1 *Geological Setting*

Sutton Wick landfill is on the outskirts of Abingdon, to the west of the historic city of Oxford, and is located on the Kimmeridge Clay formation which outcrops along a broad tract trending south-west to north-east across the area. The Kimmeridge Clay is an over consolidated, fissured, locally bituminous or calcareous silty clay which may be silty or sandy with thin siltstones or sandstones and is known to be typically around 60 m thick. Unweathered Kimmeridge Clay is dark grey in colour due to the presence of carbonaceous matter and iron pyrite, and highly fossile ferous. It weathers to a lighter grey colour, leached of some of the fossil content with the development of selenite crystals.

Below the Kimmeridge Clay are the Corallian Beds which consist dominantly of silty sands and clayey silts with siltstones and sandstones. Upper parts are mainly calcareous with limestones, interbedded limestones and clays; silty sandy mudstones occur at the top of the unit.

The recent deposits above the Kimmeridge Clay are river terrace deposits of sand and gravels, which have been extensively worked in the vicinity of the site.

The aquifer present beneath the site (the Corallian Beds) is known to be artesian. A perched water table is present on the site in the river terrace deposits above the Kimmeridge Clay and this is known to be affected by dewatering in the active gravel extraction works to the north of the site.

2.2 *Design Overview*

The landfill has been designed within the framework given in Waste Management Paper 26B (DoE, 1995) which is based on a site specific risk based approach. In this risk assessment prime consideration is given to the protection of groundwater. The main aquifer beneath the site, the Corralian Beds, is not a major aquifer however the groundwater within this stratum still had to be protected.

A more important aquifer, however, is the Terrace Gravels around the perimeter of the site. Since the groundwater, both in the Corralian Beds and in the Terrace Gravels, has an inward gradient, the landfill is hydraulically contained.

2.3 *Lining System Design*

The base of the site comprises a 1m thick reworked engineered clay barrier with a maximum hydraulic conductivity of $1x10^{-9}$ m/s, overlying up to 25 m of *in situ* Kimmeridge Clay. Hydraulic conductivity testing of undisturbed samples of the *in situ* clay gave results in the region of $1x10^{-8}$ m/s. The side slopes also comprise a reworked engineered clay liner with a minimum 1m thickness. The outside face of the engineered clay has a gradient of 1 (vertical) to 3 (horizontal), however in the region of the Terrace Gravels, the inside face has gradient of 1 to 1.5. This results in a clay thickness considerably in excess of the minimum required 1m for a majority of the slope, see Figure 1.

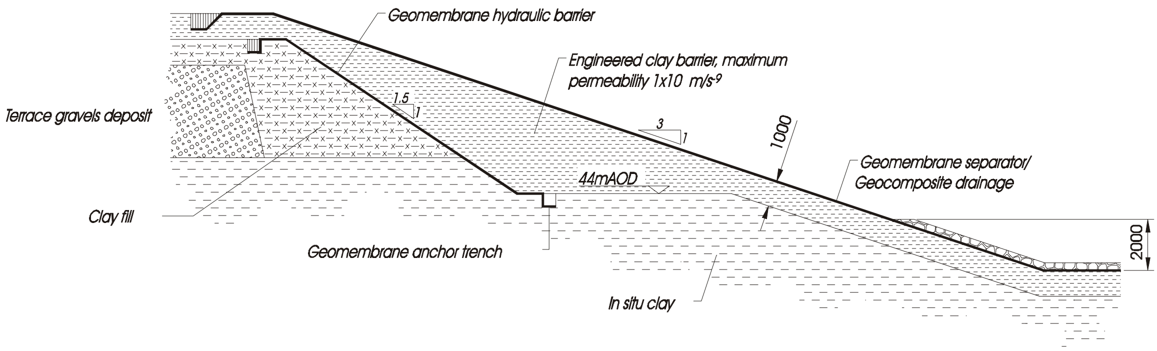

Figure 1. Typical side slope detail.

3 DESIGN ISSUES

3.1 *Overview*

Geosynthetic materials were used extensively in the construction of Sutton Wick Landfill. Instead of being used as a primary liner, a 2 mm thick textured High density polyethylene (HDPE) geomembrane was used as a hydraulic barrier beneath the single clay liner. A separator layer was required between the clay liner and gravel leachate drainage blanket and instead of using a traditional geotextile, a novel reinforced low density polyethylene (LDPE) geomembrane was used.

The use geosynthetics in Phase II developed as each cell was designed. In addition to the novel uses described above, a geocomposite leachate drainage layer was placed on the side slopes as a more cost effective solution than using a gravel drainage layer. Each aspect of the geosynthetic design will be described in detail in the following sections.

3.2 *Hydraulic Barrier*

The design of the landfill perimeter slope contains a hydraulic barrier behind the clay liner (Figure 1). Concern has been raised regarding the long-term stability of clay slopes, e.g. Jones & Dixon (1997), and although these slopes can remain stable during construction due to their undrained shear strength, their stability in the long term as pore pressures equilibriate cannot be guaranteed. Although waste will buttress the slopes in the long term, large displacements are required to mobilised the waste support (Dixon et al., 1999). The perimeter slopes at Phase II have been designed accordingly.

The main failure mechanism postulated is side slope instability caused principally by the build-up of water in the sands and gavels. Such a build-up reduces the stability of the slopes in two ways. Firstly, the water saturates the clay liner and with the build-up of positive pore water pressures in the clay the shear strength reduces. Secondly, once the clay is saturated, an active thrust can be imparted on the clay liner. These two factors lead to the reduction in overall slope stability and the possibility of a failure.

Indeed, prior to the development of Phase II as a landfill the perimeter side slopes suffered such a failure. This was due in part to the lack of design and poor construction control – the clay was merely pushed up the side slopes from the base by a dozer . To ensure that such a failure did not happen again, the side slopes of Phase II incorporates a geomembrane hydraulic barrier to prevent softening of the engineered clay liner.

3.3 *Separator geomembrane*

There is a need to separate the gravel leachate collection and removal system from the underlying clay liner. A geomembrane was chosen over the more traditional separator geotextile due to the concern over desiccation and erosion of the clay liner beneath a geotextile. However, since the separator would be in contact with 20 mm to 40 mm drainage gravel, damage to the geomembrane was considered to be a significant design issue.

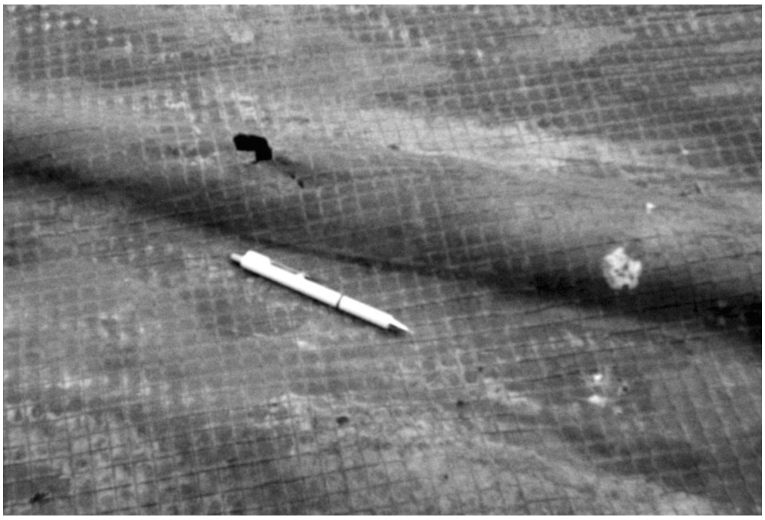

Figure 2. Example of geomembrane damage limited by reinforcement.

Damage to the geomembrane could have been limited by the use of a thick (say 3 mm) HDPE geomembrane however this was considered to be expensive and so another approach was used. The chosen geomembrane consisted of two layers of low density polyethylene sheet with built-in HDPE monofilament reinforcement on a 12 mm by 12 mm grid. The specification required a thickness of not less than 0.2 mm and tensile strength in excess of 3.5 kN/m. The design was based on allowing a certain number of defects in the geomembrane sheet due to puncture of the gravel particles, but tearing and large scale damage of he geomembrane was prevented by the reinforcing elements (Figure 2).

The material used was Netofol 200 supplied by Monarflex Geomembranes of St Albans, Herts.

3.4 *Geocomposite leachate drainage system*

3.4.1 *Introduction*

The design originally adopted for Sutton Wick for the leachate collection and removal system consists of a 300 mm thick drainage blanket, comprising rounded gravel, and 200 mm OD perforated HDPE drainage pipes over the base of the cell. The drainage blanket is then extended in stages, prior to waste lifts, to the crest of the side slopes. The drainage gravel is separated from the mineral liner by the separator geomembrane. Leachate is removed from the sump by means of a 500 mm OD upslope riser pipe.

During the construction of the first cell, the basal gravel was placed under the supervision of the CQA Engineer, as was the first lift (2 m vertically) up the side slope. The remaining gravel was placed by the landfill operator as the waste level increases. For the second and subsequent cell, the gravel leachate drainage on the slope was replaced by the installation of a geocomposite drainage layer. This material offered the following benefits; increased flow capacity for leachate drainage, ability to install the drainage system for the full slope in one operation and thereby ensure independent CQA, and the regular void structure within the drainage core of the geocomposite provides a higher resistance to clogging.

In addition, by specifying a cuspated core geocomposite, there was no requirement for a geomembrane separator since the cuspated HDPE sheet will prevent desiccation of the clay liner. The geocomposite material used was Pozidrain 6S500D/SF2 manufactured by ABG Ltd. of Meltham, Yorks.

The hydrogeological risk assessment assessed the pathways from the waste to the Terrace Gravel. One of the potential pathways identified was from perched leachate which may occur throughout the thickness of waste. The basal drainage blanket was therefore extended to the full height of the sidewalls to mitigate the potential for the development of a hydraulic gradient across the sidewalls from perched leachate to the Terrace Gravel.

The use of the geocomposite served the same purpose as the extended basal drainage blanket. Indeed, calculations suggest that it will have a greater flow capacity than the 300 mm of gravel. It will prevent the development of a hydraulic gradient across the sidewalls from perched leachate to the Terrace Gravel and thereby the leakage of leachate via this pathway. The conclusions of the hydrogeological risk assessment with respect to the risk posed by perched leachate to controlled surface and ground water were therefore unaffected by this design change.

3.4.2 *Flow capacity*

A comparison of the flow capacity of the gravel drainage blanket and the geocomposite is given below. This comparison is applicable for the short term and does not include any allowances for long term clogging or compression of the geocomposite.

If it is assumed that the hydraulic conductivity of the drainage gravel is $1\text{x}10^{-4}$ m/s, and the drainage blanket is 300 mm thick, then its flow capacity, or transmissivity, is $1\text{x}10^{-4}$ m/s x 0.3 m = $3 \text{ x } 10^{-5}$ m^2/s. The flow capacity of the geocomposite was measured in the laboratory as 0.9 l/m/s, or 0.9 x 1 x 10^{-3} m^3/m/s = 9 x 10^{-4} m^2/s. It can therefore be concluded that the flow capacity of the geocomposite is significantly greater than the original approved design of 300 mm of drainage gravel.

3.4.3 *Resistance to clogging*
Clogging/blocking of landfill leachate drainage layers in the long term can develop from a combination of mechanisms. These are generally seen to be physical, chemical and biological processes. A comparison of the resistance to clog for gravel and geocomposite drainage layers is presented in Table 1 below.

Table 1. Summary of Clogging Potential.

Process	Gravel	Geocomposite
Physical Clogging	Irregular void geometry leads to variable inter-particle voids. Greater potential for progressive blockage of void area by particulates at the particle contact points.	Regular void structure is less likely to block.
Chemical Clogging	Depends on the chemical composition of the gravel.	HDPE resistant to chemical clogging.
Biological Clogging	Microbes can be attracted to the relatively rough surface texture of most gravels.	Difficult for organisms to attach to the smooth surface of HDPE.

Based on the above observations, it is therefore possible to conclude that HDPE geocomposite material is likely to perform better, in the long term, than the proposed gravel drainage blanket for side slope leachate drainage.

3.4.4 *Geocomposite compressive strength*
The specified geocomposite had a compressive strength in the region of 500 kPa. The anticipated maximum loading on the geocomposite was likely to be around 130 kPa, and there is therefore a short-term factor of safety in excess of 3.5 against compressive failure of the side slope geocomposite drainage layer.

3.4.5 *Reduction in flow capacity due to geotextile deformation*
Deformation of the geotextile into the HDPE drainage core can lead to blocking, if the geotextile is loosely laid on the surface of the core. Since the geocomposite used comprises two layers of geotextile, bonded to the HDPE core, this will not to occur. Manufacture of the cuspated geocomposite drainage layer ensures that the uppermost layer of geotextile is bonded to each of the protrusions (cusps) of the core. Comparative testing has demonstrated a significant improvement in the performance of bonded geocomposite over unbonded.

The specified in-plane water flow of 0.9 l/m/s (equivalent of m^2/s) is achieved under a compressive load of 240 kPa (corresponding to around 20 m of waste), using a soft rubber foam above the geotextile which simulates a soil.

3.4.6 *Reduction in flow capacity due to compression of the geocomposite*
Structured polymeric materials such as geocomposite undergo a reduction in thickness under compressive loading. It has been observed that after an initial compression, it is usually followed by creep compression, i.e. a time dependent change in thickness of the material when subjected to a constant loading. Loading tests to 150 kPa on the geocomposite have been carried out and the test results are shown in Figure 3.

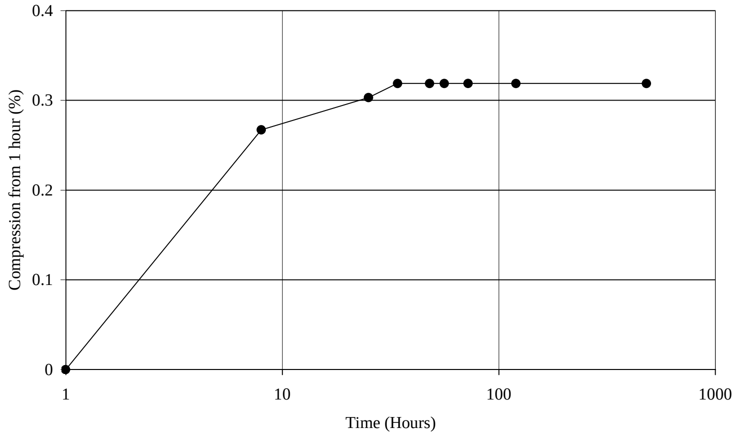

Figure 3. Results of compressive creep test.

Testing of the transmissivity of the geocomposite showed that under a pressure that is roughly equivalent to 20 m waste, the flow capacity is 0.9 l/m/s. However, the creep compression testing of the material found that over long time periods the material may be compressed by a further 0.2 %.

If it is assumed that the 0.2 % compression results in a reduction in thickness of the open area of 0.2 % (the mode of compression of the material suggests that this is pessimistic), then the open area will be reduced by the 0.2 % x 0.2 % - i.e. 0.04%. Darcy's Law states that flow rate is proportional to open area (as well as hydraulic conductivity and hydraulic gradient). It can therefore be expected that the long term flow capacity of the material is reduced by about 0.04% - i.e. to approximately 0.8996 l/m/s.

A flow capacity of 0.8996 l/m/s is equivalent to 0.0008996 m^3/m/s. The transmissivity of the geocomposite material is therefore 0.8996 $x10^{-3}$ m^2/s.

The transmissivity of the drainage blanket as calculated above is 3 x 10^{-5} m^2/s or 0.03 x 10^{-3} m^2/s. The transmissivity of the geocomposite is therefore approximately 30 times greater than the transmissivity of the drainage blanket.geosynthetics installation

4 INSTALLATION OF THE GEOSYNTHETIC MATERIALS

4.1 *Geomembrane hydraulic barrier*

The hydraulic barrier was placed onto a 1 in 1.5 slope cut into the clay fill material and anchored in trenches at the crest and toe. Hot wedge fusion welding was used for all seaming operations. Placement and compaction of engineered clay in front of the geomembrane (Figure 4) was carried out carefully under the strict supervision of a CQA Engineer. Due to the CQA regime and good supervision from the contractor, no geomembrane damage was observed during the placement of the clay barrier.

4.2 *Separator geomembrane*

The separator geomembrane was brought to site in large pre-fabricated panels that were deployed manually on site. This reduced the amount of site welding required. The material was placed on the perimeter side slopes, base, temporary bunds and intercell bunds. It was anchored at the perimeter slope in an anchor trench some 600 mm deep. All geomembrane field overlaps were a minimum of 150 mm between adjacent panels and all joints were heat bonded by a hot air welding machine. All separator geomembrane was successfully installed without any problems.

Figure 4. Placement of clay liner in front of geomembrane hydraulic barrier.

4.3 *Geocomposite leachate drainage system*

At the base of the perimeter side slopes, the geocomposite was run-out for a distance of 1 m horizontally along the base from the toe of the side slope. This was to ensure that leachate from the geocomposite will be directed towards the leachate collection and removal pipes in the base of the cell. Above this run-out a 300 mm thick layer of drainage gravel was placed.

At the top of the slope, the geocomposite was anchored in a trench excavated with a 45° front face, to ensure that the geocomposite was not subjected to sharp edges. This anchor trench was backfilled with clay placed and compacted in layers. Each panel of geocomposite was overlapped, and the upper geotextiles were sewn together at each joint location. Rows of sand bags at 2 m centres, tied to ropes were also placed on top of the geocomposite at 20 m intervals across the slopes to prevent uplift due to wind.

Figure 5. Geocomposite side slope leachate drainage layer.

5 SUMMARY

This paper has presented a case study of geosynthetics use in landfill applications that are not common. The use of a geomembrane barrier beneath a single clay liner to prevent softening of the clay, and subsequent instability has been described. Also, the use of a thin reinforced geomembrane as a separation layer between the clay liner and a gravel drainage layer has been discussed.

This paper has also detailed the use of a geocomposite drainage material on the side slopes of Sutton Wick Landfill, and has demonstrated the benefits of its use. Equivalency between gravel and geosynthetic leachate drainage systems have been discussed and a summary of site specific testing carried out for this project has been given.

The last cell in Phase II of Sutton Wick Landfill was constructed this Summer. The performance of the landfill and in particular the geosynthetics has been exemplary.

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