In situ Performance and Numerical Analysis of Lining Systems for Waste Containment

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IN SITU PERFORMANCE AND NUMERICAL ANALYSIS OF LINING SYSTEMS FOR WASTE CONTAINMENT

By

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A dissertation thesis submitted in partial fulfilment of the requirements for the award of the degree Doctor of Engineering (EngD), at Loughborough University

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Dla mojego Dziadka
To my Grandpa

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ABSTRACT

Growing environmental awareness has led to developments within landfill engineering, increasing the amount of research with the aim of constructing safe, stable landfills with optimal geometry. EU member states are forced to improve waste disposal policies through directives (Council of the European Union 1999) enforced in member countries through local legislation (in the UK, The Landfill (England and Wales) Regulations 2002). This research focuses on several aspects of waste barrier *in situ* performance. A field study was conducted on a landfill side slope to investigate geosynthetics mechanical behaviour in service conditions and on a landfill capping to investigate capping geosynthetic drainage system performance *in situ* conditions and pore water distributions along the capping. Further site derived data were collected in order to validate numerical modelling approaches, to increase confidence in a design processes and to investigate mechanisms incorporated in the liner's performance. The side slope studies revealed an additional factor affecting lining components displacement along the slope: geomembrane and geotextile response to atmospheric conditions. The capping study allowed production of recommendations for future capping designs. These can be used to considerably enhance capping stability.

KEY WORDS

Landfill Lining System, Landfill Capping, Veneer Stability, Geosynthetics, Lining Integrity, Geosynthetic Capping Drainage System, Landfill Stability Risk Assessment.

PREFACE

This thesis is a result of the research conducted between 2009 and 2013, to fulfil the requirements of an Engineering Doctorate (EngD) at the Centre of Innovative Construction Engineering (CICE), Loughborough University. The research programme was supervised by CICE at Loughborough University and funded by the Engineering Physical Sciences Research Council as well as Golder Associates (UK) Ltd as sponsors.

The main aim of the EngD is to solve one or more significant and challenging engineering problems with an industrial context. The EngD is an alternative to the traditional PhD, as it requires the researcher to be positioned within a sponsoring organisation guided by an industrial supervisor. Academic support is provided by regular contact with academic research supervisors.

The EngD is examined on the basis of a discourse supported by publications or technical reports. This discourse is supported by three journal, and one conference paper.

The papers have been numbered 1-4 for ease of reference and are located in Appendices A to D to the discourse. While references are made throughout the discourse to the papers there are key reference points in chapter four where the reader is directed to read each paper in its entirety and then return to the discourse. This is intended to reduce the need for the reader to constantly refer to the accompanying papers while reading the discourse.

USED ACRONYMS / ABBREVIATIONS

CICE   – Centre for Innovative and Collaborative Engineering

EngD   – Engineering Doctorate

EU   – European Union

FCf   – Cuspated core geocomposite

FLAC   – Fast Lagrangian Analysis of Continua

Golder   – Golder Associates (UK) Ltd

GPT   – Non-woven needle punched geotextile with integral longitudinal band drains at regular spacing

HDPE   – High Density Polyethylene

HPS   – Non-woven needle punched geotextile

HSR   – Horizontal Submergence Ratio

PSR   – Parallel Submergence Ratio

VWC   – Volumetric Water Content

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LIST OF PAPERS

The following papers, included in the appendices, have been produced in partial fulfilment of the award requirements of the Engineering Doctorate during the course of the research.

PAPER 1 (SEE APPENDIX A)

Dixon, N., Zamara, K.A., Jones, D.R.V., Fowmes, G., 2012. Waste/Lining System Interaction: Implications for Landfill Design and Long-term Performance. Geotechnical Engineering Journal of the SEAGS & AGSSEA Vol. 43 No.3 September 2012 ISSN 0046-5828.

PAPER 2 (SEE APPENDIX B)

Zamara, K.A., Dixon, N., Jones, D.R.V., Fowmes, G., 2012. Monitoring of a landfill side slope lining system: Instrumentation selection, installation and performance. Geotextiles and Geomembranes 35(2012)1-13.

PAPER 3 (SEE APPENDIX C)

Zamara, K.A., Dixon, N., Fowmes, G., Jones, D.R.V., Zhang, B, 2013. Landfill side slope lining system performance: study case vs numerical modelling analysis. Draft, Submitted to Geotextiles and Geomembranes.

PAPER 4 (SEE APPENDIX CD)

Zamara, K.A., Dixon, N., Fowmes, G., Jones, D.R.V., Zhang, B., Sharley, M., 2012. Instrumentation measuring hydraulic performance of a landfill cap: Case study. The 5th European Geosynthetics Congress, Valencia 2012.

1 INTRODUCTION

1.1 RESEARCH CONTEXT

Modern landfill engineering in the UK involves detailed analysis of construction and environmental matters, in order to meet the requirements of the Environment Agency, European Union regulations and UK legislation. This is to minimise the impact on human health and ensure environmental safety. The EU Landfill Directive (1999) has been adopted in the UK through the Environmental Permitting (England and Wales) Regulations 2010 and aims '*to prevent or reduce as far as possible negative effects on the environment, in particular the pollution of surface water, groundwater, soil and air, and on the global environment, including the greenhouse effect, as well as any resulting risk to human health, from the landfilling of waste, during the whole lifecycle of the landfill*'. Restrictions have been introduced to minimise the amount of waste sent to landfills, *i.e.* liquid waste, corrosive, explosive or flammable waste, hospital and clinical infectious waste, whole used tyres (since 2003) and shredded tyres (since 2006). Waste and Landfill Sites are classified as being inert, non-hazardous or hazardous and any waste designated for landfill can only be sent to the appropriate landfill site. Since 30th October 2007, pre-treatment (including sorting) of waste going to landfill is required to encourage recovery and recycling. Also, EU member states are obliged to reduce landfilled biodegradable waste to 35% of the 1995 levels by 2016. Although reduction in waste volume / weight sent to landfills is significant, there is no doubt that landfill will still significantly contribute to the materials disposal sector in the UK and worldwide. Therefore, there remains a need to advance the engineered containment of landfill. According to the Council Directive 1999/31/EC (1999) required measures are associated with landfill emissions: leachate volume and composition, surface water composition, gas emission and atmospheric pressures and these are related to waste classification. Although geotechnical stability assessments are mandatory according to Environmental Permitting

(England and Wales) Regulations (2010), there is no formal requirement to monitor the mechanical performance of the lining system directly. Required monitoring parameters are related to the environmental impact of the liner's performance (*eg.* groundwater contamination) and reporting data in the case of exceeded limits may relate to an already damaged liner. Even if monitoring is required as a condition of the license for a particular landfill, data is rarely published. Information about lining system performance can be derived from back analysis of large scale landfill failures (Koerner and Soong 2000, Dixon and Jones 2003, Muhsiung 2005) but these cannot provide insight into in-service performance of nominally stable facilities. Current practice in landfill design typically involves applying limit equilibrium methods, and where necessary numerical modelling software is used, employing complex methods for predicting landfill lining system behaviour to evaluate displacements, strains, and tensile stresses resulting from waste body lining system interaction (Dixon and Jones 2003). However, there is still little attention given to in-service performance of landfill lining systems and interaction of the materials within a barrier system. Although in-service failure can lead to environmental damage, commonly used design approaches have not been verified through monitoring of liner behaviour during construction, filling and after closure.

In the scope of Eurocode7 design guidelines, to meet requirements for an environmentally safe landfill it is important to maintain stability (ultimate limit state) and integrity (serviceability limit state) of a lining system throughout the lifetime of a landfill. While loss of stability of the landfill involves large scale movements (*eg.* slope failure), integrity is related to overstressing of liner elements and consequently loss of original functions, according to which the liner was designed (*e.g.* low permeability barrier, protection layer). Figure 1.1 presents typical landfill cross section, with the main construction elements from the geotechnical point of view.
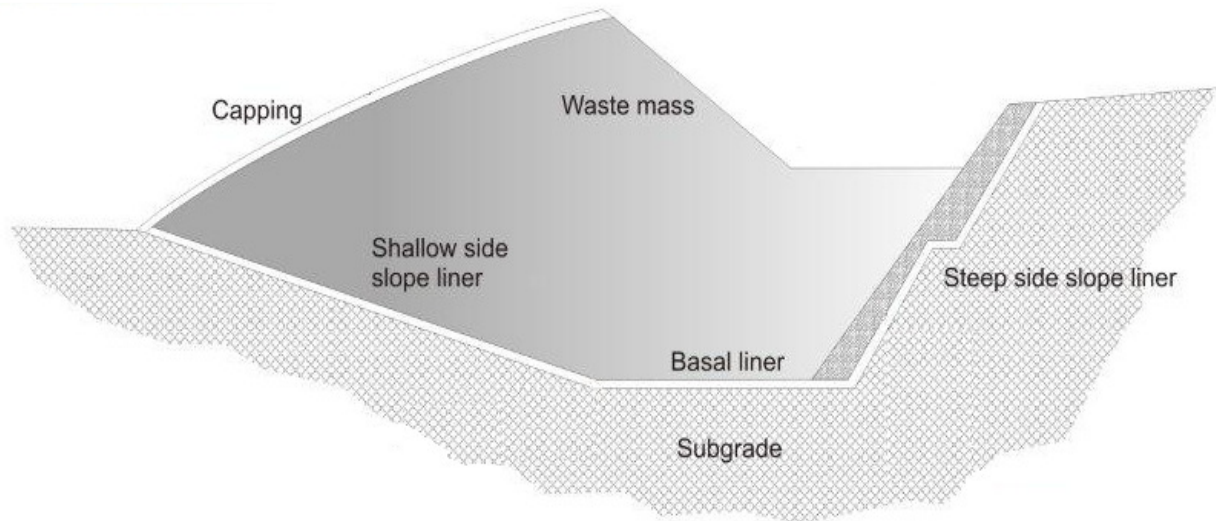
Figure 1.1 Landfill cross – section overview (after Fowmes *et al.* 2007).

Figure 1.2 presents the areas of concerns for a landfill design engineer, which are also listed below (after Fowmes *et al.* 2007):

(1)   Side slope (steep / shallow) stability / integrity;

(2)   Basal lining system stability / integrity;

(3)   Subgrade behaviour;

(4)   Groundwater behaviour;

(5)   Capping stability / integrity with potential seepage build-up;

(6)   Appropriate material selection for the barrier; and

(7)   Waste parameters.

It is of high importance to build environmentally safe landfill constructions and to assess adequately the performance of landfill lining systems and in particular, to predict stresses and strains in lining elements resulting from waste placement and settlement.

This research comprised two main topics which investigated both *in situ* behaviour and numerical processes involved in assessing geosynthetics performance on a landfill side slope and landfill capping. The first project investigated mechanical behaviour of a geosynthetic lining system, whereas the second project investigated hydraulic performance of geosynthetics drainage layers.
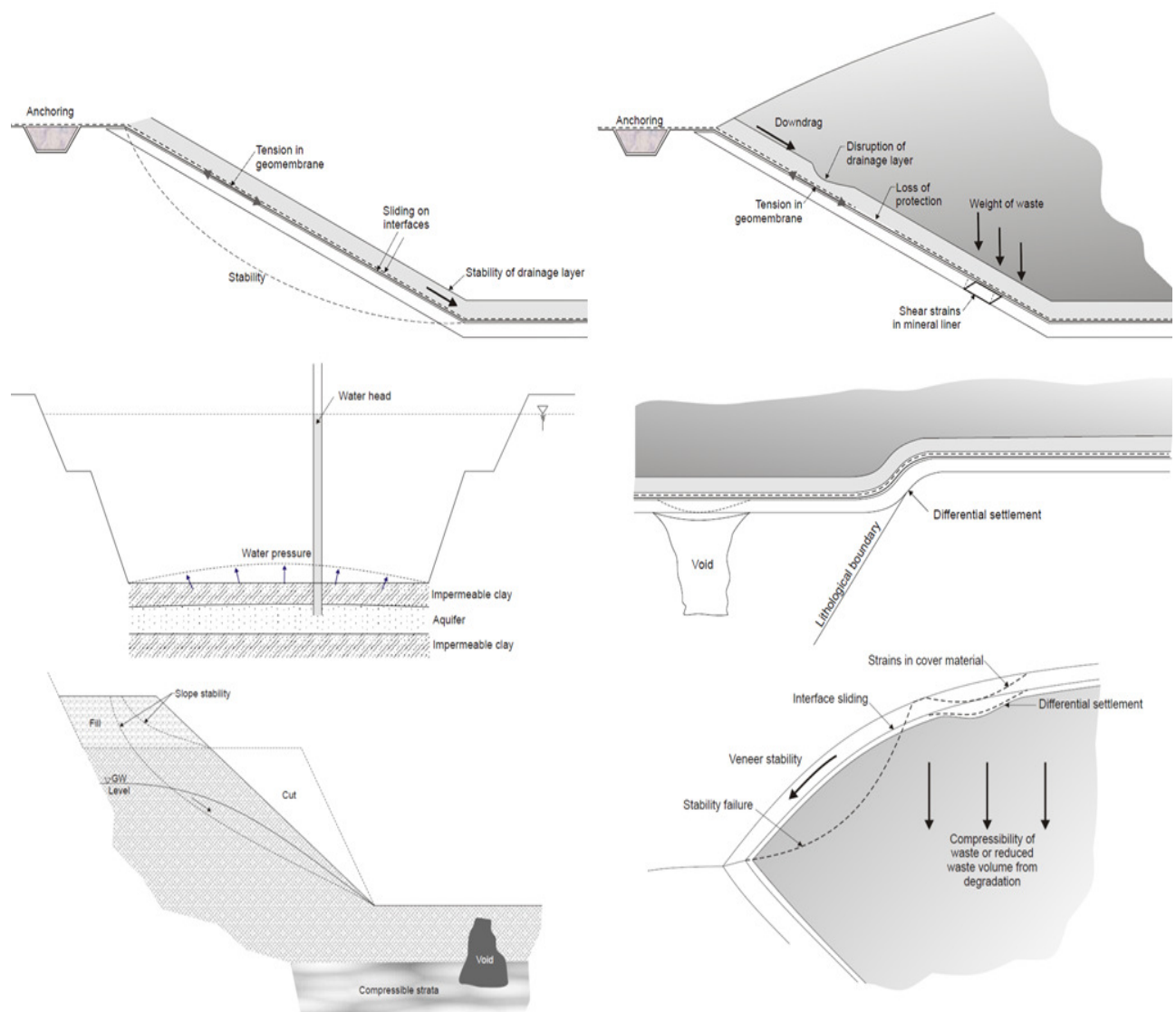
Figure 1.2 Issues related with landfill barriers performance (after Fowmes *et.al.* 2007).

1.2 **LOUGHBOROUGH WASTE GROUP**

Previous doctoral students at the University investigated topics related to landfill engineering. This project has been conducted as a continuation of a study carried out by former Engineering Doctorate student Gary Fowmes (Golder Assosciates (UK) Ltd project). The areas previously investigated by the group are as follow:

(1)   Interface shear strength of landfill lining system components (Jones 1998);

(2)   Quantification of interface shear strength variability and probabilistic design (Sia 2007);

(3)   Numerical modelling of landfill lining system behaviour and commercial implementation of design philosophy (Fowmes 2007);

(4)   Waste classification and assessment of waste shear and compression behaviour (Langer 2006); and

(5) Waste constitutive behaviour (Zhang 2007).

This research, through site investigations naturally follows and expands findings of the previous researchers, and identifies areas of further studies. Additionally, the study has been mentored by Prof. Neil Dixon - the co-author of UK landfill design guidance (R&D Technical Reports P1-385/TR2&TR2), with industrial sponsor Dr Russell Jones, Golder Associates (UK) Ltd.

1.3 **INDUSTRIAL SPONSOR - GOLDER ASSOCIATES (UK) LTD**

Golder Associates was founded by Hugh Golder, Victor Milligan, Larry Soderman and John Seychuk in 1960 in Toronto, Canada. The global consultancy company started by offering services related to soil mechanics and foundation engineering. In 1970s and 80s the service expanded to rock mechanics and groundwater expertise including contaminated soil and water. In UK the company was established in 1973, and ever since has delivered services to local and international clients.

Currently Golder is an employee-owned company, with more than 180 offices worldwide employing more than 8000 people, specialising in six broad areas:

(1)   Ground engineering;

(2)   Natural resources management;

(3)   Environmental and social assessment;

(4)   Environmental management and compliance;

(5)   Decommissioning and decontamination; and

(6)   Planning and design.

In the days of rapid information exchange, close cooperation between the branches take place on the national and international basis. Therefore the UK offices cooperate with branches all over the world, undertaking jobs across the globe.

This engineering doctorate was supported by the Nottingham branch of the company, where the team of UK geotechnical / landfill engineers is located. This provided the opportunity for the author to be involved in various (landfill and general geotechnical) commercial projects.

Furthermore Golder UK with Dr D. Russell V. Jones has been the leading group within the UK landfill engineering sector, which focused on the application of Integrated Pollution Prevention and Control Regulations, Environmental Permitting process, additionally co-authoring of UK landfill design guidance (R&D Technical Reports P1-385/TR1&TR2). Hence, this EngD project was well situated within industry to carry out landfill engineering study that would be of benefit to the company.

1.4 **OVERARCHING AIM**

The work described herein has two main aims:

(1)   To validate numerical modelling design approach used for landfill multi-layered lining systems (landfill side slope project); and

(2)   To investigate current landfill capping drainage system design approach (landfill capping project).

1.5 **OBJECTIVES**

The objectives of the landfill side slope project are:

(1)   To monitor integrity of a geosynthetic landfill multi-layered lining system in field conditions;

(2)   To analyse the monitored performance using advanced design methods;

(3)   To validate multi-layered lining system design tools against field data.

The objectives of the landfill capping project are:

(4)   To monitor hydraulic performance of landfill capping geosynthetic drainage materials in field conditions; and

(5)   To compare measured hydraulic performance to values predicted by numerical modelling software.

Figure 1.3 summarises the research objectives, completed tasks, methodology used and indicates papers in which the particular work has been published.
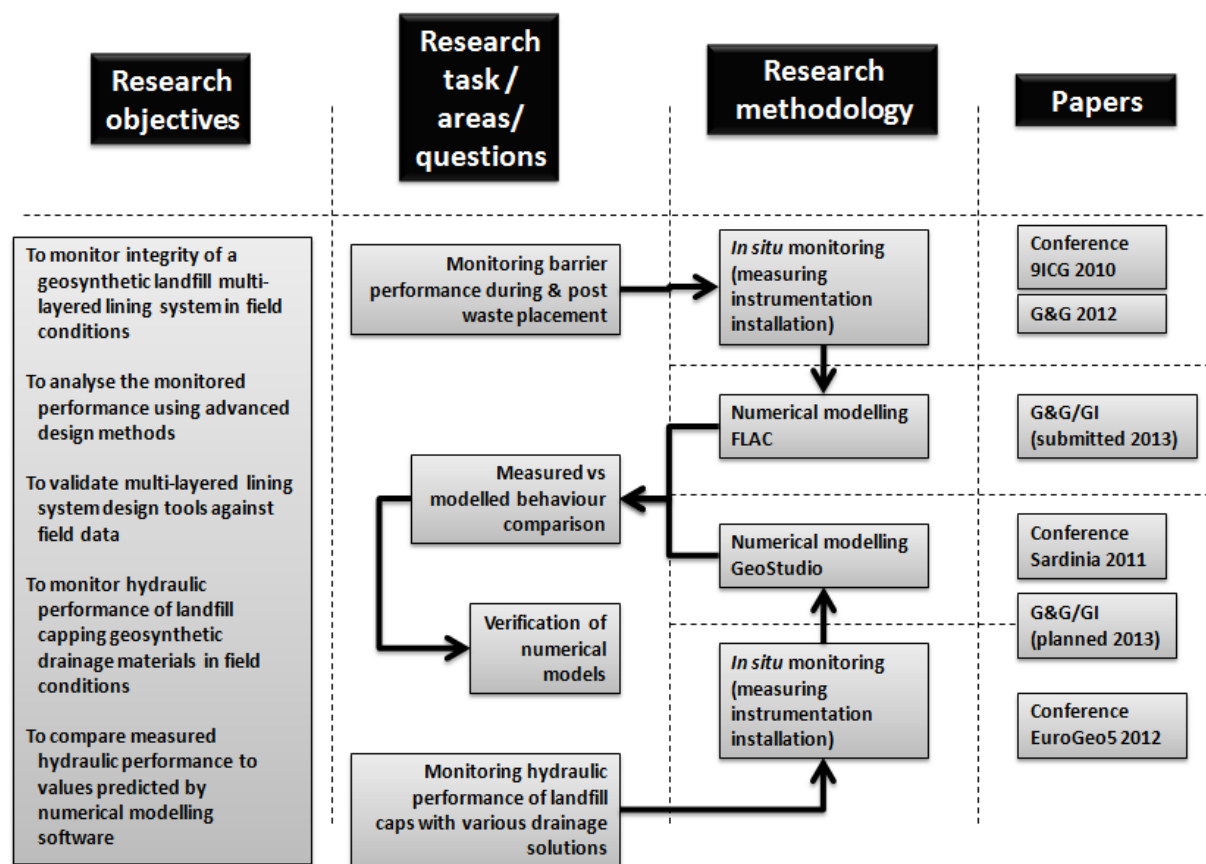
Figure 1.3 Research map (note: G&G – Geotextile & Geomembrane Journal, GI – Geosynthetic International Journal)

1.6 JUSTIFICATION OF THE RESEARCH

Although in developed countries waste disposal through landfilling is a declining market, the matter of safe, optimised landfill design will still be a valid topic in UK due to residual waste streams, non-recyclable / recoverable waste and also managing the legacy of the existing landfills.

While global stability of a landfill structure can be assessed through conventional limit equilibrium methods, it is insufficient for integrity analysis of the lining system components. Integrity analysis requires more sophisticated analysis, which can be derived through use of numerical modelling software. However, these must be calibrated using measured in-service performance. Within existing worldwide literature there is a lack of site derived data on geosynthetics *in situ* behaviour. Only limited information on numerical modelling calibration

against *in situ* performance is available, for both: landfill side slope and capping construction. The importance of field data on performance is acknowledged by many, as it is clear that it is challenging to represent *in situ* conditions in laboratory scale studies. Materials on site are exposed to the environment, various atmospheric conditions, soil erosion processes *etc.*, therefore long term monitoring provides complex information on ageing and actual in-service performance. Data from full scale experiments are of importance not only to verify numerical modelling approach but also to help understand the mechanisms of observed behaviour, to allow a greater understanding of geosynthetic performance, increase confidence in long term performance of containment systems and to allow design optimisation. This work examines commercially applied codes and design approaches against site derived data. Additionally, it provides recommendations for landfill side slope and landfill capping geosynthetic drainage system design.

1.7 STRUCTURE OF THE THESIS

First chapter presents an introduction to the problem. Talks about main aspects of the research, puts the research into a context by introducing the team among which the research was carried out, talks about the research aims and objectives. Most importantly justification of the research can be found in this chapter.

Second chapter gives a context to the engineering doctorate works, demonstrates the main aspects of the research, and extends the introduction with a focus on the main topics of the research.

Third chapter explains and justifies the methodology which was undertaken in order to investigate the main topics. Details about the projects and the way of handling the solutions are described in this chapter.

Fourth chapter summarises fulfilled tasks and describes details of the undertaken site, laboratory and modelling works.

Fifth chapter presents the crucial results and the main findings draw from this study. Discussion of the findings, contribution to the existing theory and practice, implication on the sponsor and the wider industry with recommendations for the industry and future research are presented, along with a discussion on a design process.

Sixth chapter concludes and briefly summarises the engineering doctorate projects and findings.

2 BACKGROUND TO THE RESEARCH AREAS

This chapter gives a context to the engineering doctorate works, demonstrates the main aspects of the research, and extends the introduction with a focus on the main topics of the research.

2.1 LANDFILL GEOTECHNICAL ENGINEERING - OVERVIEW

Within the last few decades much development has been observed within the waste sector. Increasing awareness of the waste disposal environmental implications resulted in increasing controls over the landfilled materials. It is estimated that 300 million tonnes of waste are produced annually in UK (Department of Environment, Food & Rural Affairs, 2011) but the volume of waste sent to landfill sites reduces year by year. Although much pressure is put to limit the amount of landfilled material, disposal of waste by burial still remains the most common method of waste treatment. Therefore, environmentally safe and financially optimised landfill sites still remain within the scope of commercial demand.

Appropriate landfill design is of importance, not only for the reasons of pollution prevention (DETR 2000) but also to ensure safety of the construction sites (CDM 2007). Although design methods to assess stability are well established (*e.g.* Koerner & Daniel 1997) still many landfill failures occur worldwide (Seed *et al.* 1990, Brink *et al.* 1999, Koerner & Song 2000, Eid *et al.* 2000, Jones & Dixon 2003). These include loss of integrity between geosynthetic / soil and geosynthetic / geosynthetics interfaces within base, side slope or capping (Stark & Newman 2010), or loss of stability of a waste body (reported especially within low and middle income countries where risk measures are reduced) and loss of stability of the subgrade or low permeability barrier layers. In general, landfill failures are not reported frequently in literature. Jones & Dixon (2003) reveal their relatively high occurrence with low severity in the UK. Fowmes (2012, personal communication) confirms that this still remains a current problem in the UK. In general, negative publicity is not desired by landfill

management, therefore little information can be found in the current literature. Also, there are few academics working with operators who would be likely to publish any information.

Engineered landfill often requires a complex analysis of all the construction components in order to meet requirements of the regulatory body, which is the Environment Agency in England and Wales. Due to application of modern materials in difficult subgrade conditions, numerical modelling software is sometimes used to evaluate geotechnical risks related to landfill construction. This includes lining system stability and integrity, underlying geological / hydrogeological strata, accommodating maximum amount of waste, waste slope stability, and after construction closure – *i.e.* capping stability and integrity. Often due to novelty of materials, design techniques are not fully developed / calibrated with numerical models and this is where research studies can be important bringing solutions or showing the direction of future work / practice to the industry.

Landfill lining system stability consideration is required as a part of the design and permitting process. The Environment Agency guidance: R&D Technical Reports P1-385/TR1&2 (Dixon & Jones 2003), was published to review and summarise current landfill practice for constructing engineered landfill in the UK. The Report No. 1 presents international literature review of landfill engineering practice, with focus on lining system stability and integrity. The Report No. 2 provides guidance on the landfill lining system design. Although almost a decade has passed since their publication, it still remains valid in the majority of its aspects and provides a good introduction to landfill engineering techniques. In general, landfill construction comprises: base, side slope (steep/shallow), capping (steep/shallow), leachate infrastructure and gas infrastructure (Figure 1.1). All of these often comprise multi-layered lining systems constituting arguably the most important component of the engineered landfill – the barrier layer. A typical landfill lining system comprises the following layers: barrier layers, protection layers, drainage layers, and reinforcement layers

(optional) and these are briefly discussed in the next section. Components and material properties of the layers are variable, depending on *in situ* conditions (subgrade category, groundwater level, *etc.*), design suitability in terms of materials stability and integrity, availability (a number of landfills have good quality materials available on sites), cost efficiency and simplicity of installation. Additionally, consideration has to be given to waste mass stability and its influence on landfill construction stability, lining system integrity, loadings from waste, settlements, deformations, and potential side slope lateral support. This research primarily concentrates on integrity related issues of the landfill side slope system and stability / integrity of capping systems. Therefore the foremost attention is paid to barrier, protection and drainage layers performance.

Barrier layers

The purpose of the barrier layer is to prevent subgrade and groundwater contamination by uncontrolled leachate infiltration to the adjacent environment (mainly local groundwater) and to prevent external water inflow into the system (which would cause leachate surplus). The current design practice in UK and EU member states was introduced by EC Landfill Directive 1999 and enforced through the Environment Agency (England and Wales) Environmental Permitting Regulations (2010). This states that for thickness and permeability for three waste categories is as follow:

(1)   Hazardous waste: $k \leq 1.0 \times 10^{-9}$ m/s, thickness ≥ 5m;

(2)   Non-hazardous (*e.g.* municipal solid waste): $k \leq 1.0 \times 10^{-9}$ m/s with, thickness ≥ 1m;

(3)   Inert waste: $k \leq 1.0 \times 10^{-7}$ m/s, thickness ≥ 1m.

Where the geological barrier does not meet the requirements of the above criteria naturally, it may be completed artificially and reinforced by other means providing equivalent protection, but in any such cases a geological barrier established by artificial means must be at least 0.5 m thick (if allowed by hydrological risk assessments).

The geological barrier must be in place in the bottom and along the side slopes of the landfill (Figure 1.1). A variety of materials can be used as barrier layers (Dixon & Jones 2003). Geological barriers can be constructed from engineered clay (compacted clay layer) or bentonite enriched soil. Artificial sealing liners can be constructed of polymeric geomembranes, geosynthetic clay liners, colliery spoil, processed fly ash, processed shale, dense asphaltic concrete, processed harbour sludge (material from the port of Hamburg reported by Tresselt *et al.* (1998), was installed as a barrier layer on a landfill capping). Municipal sludge was investigated by Zhang and Wu (2005), as a substitute to natural clay.

Protection layers

The main function of this layer is to protect the barrier layer from any mechanical damage during landfill construction, and during and post waste placement (*e.g.* geomembrane cutting, tearing, clay desiccation). Protection layers are typically used with geosynthetic artificial sealing liners and comprise geotextile materials and sand layer. Thick (10-20 mm) non-woven needle punched geotextiles with a sufficient puncture resistance (*i.e.* up to 40 kN) and push through displacement (*i.e.* 65 mm) are commonly applied as protection layers. Jones *et al.* (2000) report results of tests carried out on three different geotextiles. These revealed that the selection based on a material unit weight is not relevant without considering other properties. Typically, static puncture strength (CBR), dynamic perforation resistance, tensile strength provide sufficient information to describe materials protection performance.

Drainage layers

Drainage layers enhance control over leachate hydraulic regimes within a landfill construction, as it maintains low leachate levels above the base and transports contaminant liquids to the location of its collection. Drainage systems along landfill caps reduce pore water pressures and enhance cap stability. Additionally, drainage systems can be installed to control groundwater below a barrier. According to EU requirements, a drainage layer along the

landfill base with a thickness of 0.5 m minimum is compulsory for most of the sites, which can be reduced to 0.3 m with the inclusion of pipes. There is an increasing number of materials used to form drainage layers *e.g.*:

(1)   Conventional gravel layers;

(2)   Geocomposites (cuspated core geocomposite, geonets with geotextile filters) or both;

(3)   Recycled aggregate;

(4)   HDPE pipes;

(5)   Shredded tyres;

(6)   Whole tyres (baled tyres);

(7)   Glass cullet (Ellard *et al.* 2005); and

(8)   Recycled rubble (Palmeira *et al* 2007).

Geosynthetic drainage liners are widely applied on landfill caps due to ease of transport, storage and deployment; however their application requires much attention in terms of analysis of their internal shear strength, interface shear strength with adjacent materials in response to applied loadings, flow capacity in relation to long term durability, potential clogging and general material properties in terms of ageing in the in-service environment.

Reinforcement layers (optional)

Reinforcement layers provide additional support to ensure stability of the construction. These commonly comprise geotextiles or geogrids. Applications consist of veneer cover support on side slopes, landfill base, enhancing stability of landfill leachate pond, capping or occasionally within the waste body to increase its global stability (Bouazza and Gassner 2005). The later is rarely practiced in the UK.

Conditions imposed on the liners

Landfill liners are subjected to diverse conditions, depending on their location within the structure. Table 2.1 presents general comparison between conditions imposed on a landfill side slope and landfill cap lining systems.

It is clear that conditions imposed on an in-place lining system: side slopes and landfill final covers are very different. Stresses acting on a capping system are insignificant compared with the side slope. The hydraulic regime of the capping might be very unstable and change on a daily basis, while side slope hydraulic and thermal conditions respond to overall waste state. Tensile forces within the geosynthetic capping system can increase due to cover deformation triggered by differential settlement of the underlying waste. On a side slope the material is exposed to downdrag from the adjacent body (waste or drainage layer), overloading or inadequate loading placement (*i.e.* direction of protection layer, drainage layer or waste placement).

Conditions imposed on liners and soil covers		
Parameter	**Side slope**	**Capping**
Total Stresses	0 - > 100 kPa	0 to 40 kPa
Hydraulic Pressures	0 - 300 kPa	Suction 0 to > 1000 kPa Pore Water Pressure < 20 kPa
Temperature	Slow Increase and Decrease 10 to 50 °C	Highly Variable (Typically Between -10 / +30 °C)
Degree of Saturation	100%	0% to 100%
Water Phase	Liquid Only	Multiphase Vapour, Liquid, Ice
Hydraulic Regime	Downward Gradient	Infiltration, Runoff, Evapotranspiration, Change in Storage
Environment	Isolated Engineered	Microbial Communities, Plants, Roots and Fibre, Animals, Living Systems
Contamination	Leachate	Undesirable (/Landfill Gasses)

Table 2.1   Comparison of conditions imposed on liners and soil covers for a ready construction (modified after Wilson *et al.* 2003)

2.2 **LANDFILL BARRIER INTERACTION**

Geosynthetic materials have been deployed on sites for over two decades and are currently widely applied worldwide on engineered landfills (Figure 2.1), but their *in situ* interface performance is not well documented. Numerical modelling codes have been developed with specific functions for analysing landfill site geotechnical problems (Fowmes *et al.* 2008, Villard *et al.* 1999) including staged placement of municipal solid waste, mobilised shear strength of the geosynthetic lining system interfaces (strain softening and progressive failure), tensile stresses in elements and representation of waste behaviour (Zhang 2007, Machado *et al.* 2002). Construction stages result in deformation of components and hence shear stresses develop between and within materials and consequently formation of tensile stresses in the geosynthetic elements. The importance of several factors has been established and these should be considered when designing a landfill lining system. They include consideration of progressive failure through strain softening of interfaces between geosynthetics and geosynthetics / soil materials, staged placement of the waste body, consideration of tensile stresses in geosynthetics, and assessment of waste unit weight and its change due to ageing, deformation and settlement.

An important challenge is the selection of peak or residual interface shear strength parameters for use in limit equilibrium analysis or numerical modelling of multi-layered lining systems. The importance of accurate prediction / design of engineering aspects of landfill behaviour is self-evident. The current state of research and practice of engineering issues in the landfill environment is summarised in Appendix A / Paper 1, Section 2 to 4 provide state of knowledge/research whereas section 5 lists design recommendations.

The occurrence of landfill failures triggered research in landfill engineering. This refers to geosynthetics interface shear strength, experiments on various aspects of landfill lining systems, barriers interactions, numerical modelling of the landfills and waste behaviour

but only few full scale experiments monitoring lining systems performance are reported (Gourc *et al.* 1997, Bouthot *et al.* 2003, Nakamura *et al.* 2006). However, a limited number of long term monitoring experiments with back analysis have been reported (Najser *et al.* 2010, Villard *et al.* 1999). There is an increasing number of laboratory projects investigating lining system behaviour (Gourc *et al.* 2010, Koerner & Daniel 1997), attempts to develop software calibration have been undertaken (Vilard *et al.* 1999, Fowmes *et al.* 2008), and modelling of lining materials interface behaviour from test results have been carried out (Anubhav & Basudhar 2010, Bacas *et al*. 2011, Khoury *et al.* 2011). However, there is still an urgent need for information on in-service physical performance of barriers. Data from full scale experiments on lining system in-service performance are difficult to obtain, as landfill operators are afraid of studies that may reveal failure on their sites further costs implication.
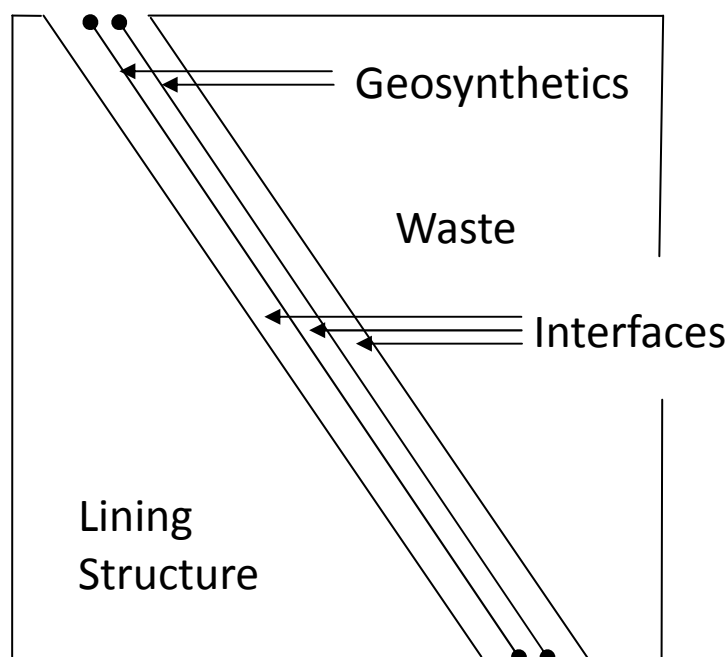
Figure 2.1 An example of a typical lining system (after Fowmes 2007).

Landfill is a very specific work environment and hence it requires robust instrumentation, which might be exposed to contaminated leachate and eventually buried under waste. Additionally, it takes years to investigate liner *in situ* performance due to slow

depletion of its properties caused by exposition to leachate or response to waste settlements. Although many laboratory tests are conducted in order to accelerate 'natural' processes (*i.e.* geosynthetics submerged in leachate – ageing), and to estimate liner durability, lifetime, in-service properties when exposed to atmospheric conditions or leachate, they still require comparison with *in situ* performance. The relevance of laboratory tests might be of question, as all the conditions are highly controlled and commonly no environmental aspects are considered (*i.e.* daily / seasonal temperature changes, microbiologic life). However, environmental factors are of significant importance in terms of investigating the actual mechanisms controlling processes that are highly complex in *in situ* conditions.

Additionally, much attention has been given to monitor landfill temperature and assess landfill lining system durability depending on liner temperature (Rowe *et al.* 2002, 2008, 2009). Studies of waste body parameters and mechanical properties have also been reported (Fassett *et al.* 1994, Gotteland *et al.* 2002, Dixon and Jones 2005, Stoltz *et al.* 2010). Nevertheless, limited information exists about geosynthetic liner performance throughout cell operation and after closure.

2.3 LANDFILL CAPPING SYSTEMS – HYDRAULIC REGIME

Landfill final cover is an important component of an engineered landfill construction. The main purpose of the landfill covers are: minimising exposure of the waste body, preventing water infiltration which determines production of leachate, controlling gas emission from waste body and optionally creating a land surface that can support vegetation or be used for further lightweight constructions to support new land use (golf courses, car parks *etc.*).

A capping system construction might be simple, typically comprising of regulating soil overlaid by a low permeability barrier (compacted clay or geosynthetic) covered by a soil veneer/top soil with vegetation (Figure 2.2a), or complex comprising of several soil and geosynthetic layers (Figure 2.2b). Additional reinforcement or drainage layers might be in

place underneath a low permeability barrier to control gas. However, these topics are not discussed in this study. Low permeability materials installed on a waste body accelerate production of landfill gasses, this is often used for commercial purposes and when extracted reduce time for which a landfill has to be monitored for gas emissions.

a)
1000mm RESTORATION SOILS
PROTECTOR GEOTEXTILE
1mm THICK DOUBLE TEXTURED LLDPE GEOMEMBRANE LINER
300mm REGULATION LAYER
WASTE

b)
300mm TOPSOILS
700mm SUBSOILS
GEOGRID
GEOCOMPOSITE DRAINAGE LAYER
1mm THICK DOUBLE TEXTURED LLDPE GEOMEMBRANE LINER
LOWER PROTECTOR GEOTEXTILE
300mm REGULATION LAYER
GEOCOMPOSITE GAS LAYER
WASTE
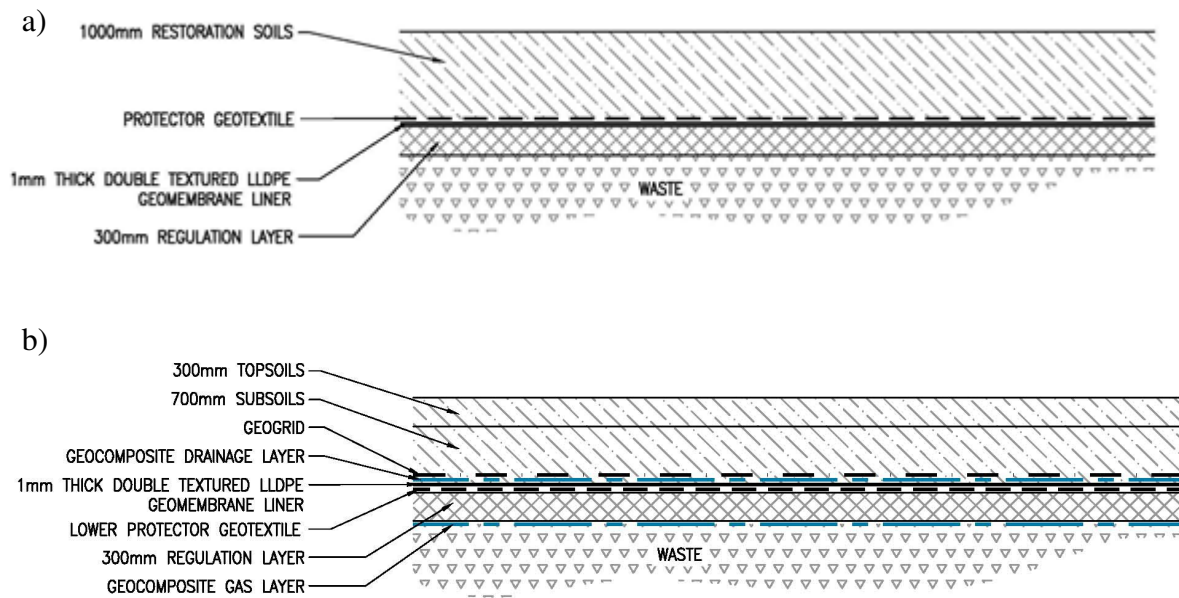
Figure 2.2 Landfill capping lining systems examples: a) simple approach, b) complex configurations.

In general, final covers might fail because the top soil layer slides on the low permeability barrier due to insufficient interface shear strength, due to increased stresses from uncontrolled pore water pressures (Stark & Newman 2010), or because of wash out of the soil surface due to increased precipitation and low internal strength of the veneer material. Therefore it is of importance to design covers with fully controlled water table and adequate management of surface flows. To date little attention has been given to analyse pore water pressure distributions above the low permeability liner along landfill cap slopes. There is currently very little site derived data about groundwater behaviour within landfill cover systems or pore water pressure distributions along cap slopes in response to geometric, soil, atmospheric and drainage conditions. Commercially applied methods of design use parallel submergence ratio (PSR) or horizontal submergence ratio (HSR) to define water pressures

used to evaluate landfill slope stability or to design a cap drainage layer (Figure 2.3). This might be considered as a conservative method due to lack of consideration given to the hydraulic parameters of materials, system mechanism or atmospheric conditions (*i.e.* it is intuitive that steeper slopes will cause higher surface water runoff, or higher permeability soil will act as a drainage layer by itself and no water head build up will be possible).
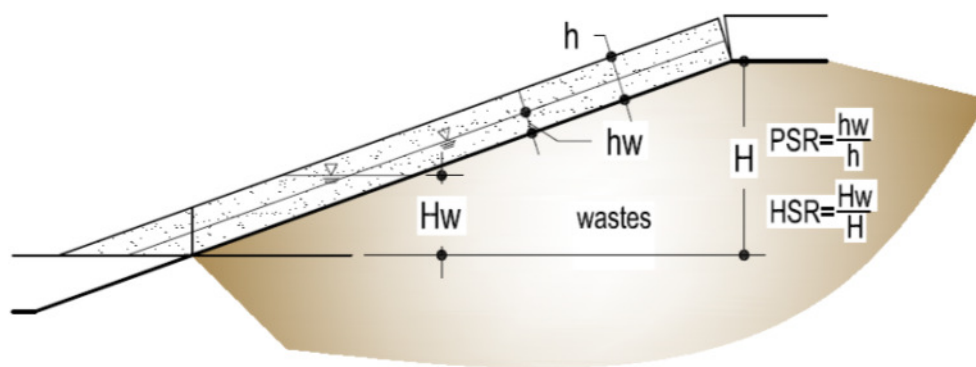
Figure 2.3 PSR and HSR ratios definitions (after Koerner and Daniel 1997).

The HSR / PSR approach is based on the Koerner & Daniel (1997) limit equilibrium method. This standard design approach assumes that if the cover system is not stable, with any pore water pressure conditions, the inclusion of drainage beneath the cover soils would typically be specified to reduce the level of pore water pressures and therefore increase the factor of safety against both stability and integrity failures. However, little attention is given to the actual drainage requirements, in terms of flow capacity required to reduce the pore water pressure within the slope. Calculation methodologies are based on assumptions that may not necessarily provide an optimal solution, *i.e.* to Authors' knowledge it has never been reported, whether full saturation of a soil cover develops or to what extent and for which conditions (atmospheric, geometric) geosynthetic drainage systems minimise development of pore water pressures. In order to verify the assumptions, improve the calculation methodology and ensure optimal, cost effective drainage system design, there is a need to monitor an

existing landfill cover system under operating conditions. This provides an opportunity to obtain valuable information to aid the design of future landfill capping drainage systems and to assess performance of existing ones.

Despite its importance, there is a lack of comprehensive information to investigate hydraulic performance of the capping system. Existing research focuses on vertical infiltration, overall water balance and so-called percolation studies with lysimeters installed on sites (Strunk *et al.* 2009, Henken-Mellies & Gartung 2004, Cazaux & Didier 2000), and capillary barrier effect (Aubertin *et al.* 2009) but little attention is given to analyse groundwater behaviour in relation to slope geometry, atmospheric and soil conditions at various locations along the slope (*i.e.* surface runoff and infiltration rate vary as a function of location on a slope), and water table behaviour and its variability along the slope. This information is important in order to better understand mechanisms controlling capping water budget, to justify current commercially applied approaches and to validate complex numerical modelling software.

Commercially applied design methods for stability of capping systems adopt simplified analysis of ground water behaviour, as it is a complicated hydrological problem with an extensive list of variables (*e.g.* root zone depth, soil moisture content, saturated / unsaturated hydraulic conductivity, porosity, field capacity, wilting point, initial moisture storage). Although a considerable attention has been given to the height of water table above a drainage liner (Koerner and Daniel 1997, Richardson *et al.* 2002), up to date design methods consider different scenarios of water table location, rather than mechanisms of their formation. Stability of a landfill final cover and water table within it has been described by Koerner & Daniel (1997). Applying PSR or HSR ratios also for geosynthetic drainage solutions is commonly addressed in commercial design. However, adequacy of the approach within sections of transient weather conditions at a site is not fully acknowledged as important

mechanisms are not included (*e.g.* increased soil moisture of a thin soil layer above geosynthetic drainage). Understanding transient analysis is crucial for optimising design. Conditions with varying water table level are of importance to reliably define a water table level and consequently stability of a capping slope.

The value of water table obtained indicates whether a drainage layer is needed or not. Also it is essential in estimation of optimal drainage layer thickness / capacity. The water table above the barrier also controls the factor of safety and hence safe constructed slope inclination. There have been several studies on water head estimation within landfill barriers. Richardson *et al*. (2002) reviewed design concepts commonly used for lateral drainage systems. The research concluded with development of the solution for a traditional - soil drainage layer and geonets drainage.

In addition, studies on the same subject were carried out by Qian *et al*. (2004) and comparisons of four analytical methods were presented (Moore 1980 (U.S. EPA 1980, 1989), Moore 1983 (U.S. EPA 1983), Giroud 1992, McEnroe 1993). These explicit formulas assume steady-state flow conditions and through different approaches, estimate maximum liquid head in a drainage layer. It was concluded that the most suitable and reliable methods of evaluating water head above a drainage layer are propose by Giroud *et al.* (1992) and McEnroe (1993). The other two greatly underestimate or overestimate the head. It should be pointed out that the methods assume steady state conditions within the cover layer and constant rate of inflow. To the Author's knowledge only limited experimental data is available to validate these methods. It is unlikely to consider atmospheric impacts on the landfill capping condition in slope stability risk assessments. Full saturation is sometimes considered in the design but only as the worst case scenario. Factors of safety are calculated for partial or full saturation but limited consideration is given to the fact whether the full saturated condition will develop within the particular capping. This might not always be the optimal approach but from the

industry point of view understandable as carrying out numerical modelling to investigate water regime within the capping (and pore water pressure development) is relatively expensive, complex and results are not definite. This requires an experienced engineer as selection of representative parameters for the analyses is highly complex and acquiring data is time consuming. Often soil used for the capping layers is chosen from the materials available on site. Therefore, very high spatial variability in geotechnical / hydraulic parameters of the capping soil can occur.

2.3.1 CAPPING GEOSYNTHETIC DRAINAGE SYSTEM

Traditionally, coarse sand or gravel have been applied to control pore water pressure distributions within various earth structures. However recently it is more common to deploy geosynthetic materials due to simplicity of installation and lower cost. Currently there is a variety of geosynthetic drainage materials available on the market. These might comprise a single layer geotextiles, geocomposites such as: geotextiles with inserted drains, cuspated core drainage with filter layer over the top, or multi-layered / multifunction geocomposites (*e.g.* Figure 2.4). Usually geosynthetic materials applied on the capping systems are less robust with lower strength properties and thickness, comparing to bottom or side slope materials because applied stresses are lower.
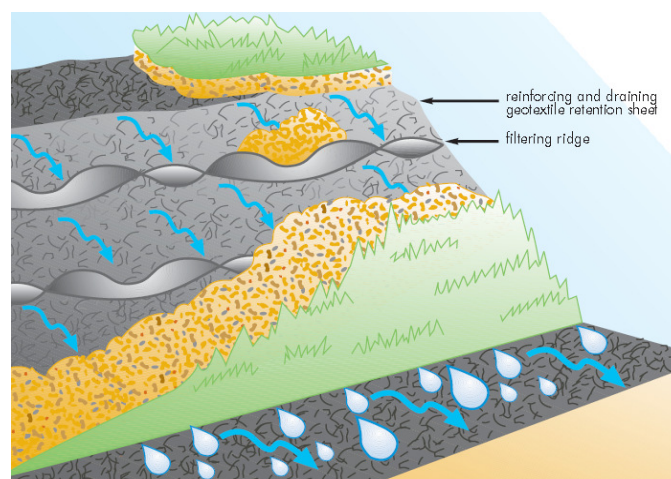
Figure 2.4 Multifunction geocomposite cap layer (Teragéos 2012).

Hydraulic properties of drainage materials in laboratory conditions have been investigated widely (*i.e.* McCartney *et al.* 2005, Bouazza *et al.* 2006, Nahlawi *et al.* 2007). Methods of property measurement are standardised (EN ISO 12958:1999, EN ISO 11058:1999) and results are available from suppliers. However, only a limited number of studies on *in situ* performance and verification of laboratory test results of geosynthetic drainage systems are available. Siemens and Bathurst (2010) reported numerical validation attempts of one dimensional sand-geotextile column tests. In order to suit column tests results it was required to reduce modelled geotextile permeability up to two orders of magnitude to match measured ponding head and water front progression. This was suspected to be a consequence of sand particle intrusion into the geotextile, which was not captured by the standard laboratory permittivity tests. Iryo and Rowe (2005) conducted laboratory studies simulating, in controlled conditions, infiltration through an inclined cap. These tests were performed with a sample size: 5 m long with 0.3 m thick and geonet underneath the sample. Their further simulations revealed good correlation between laboratory results and numerical modelling calculations.

2.3.2 CAPPING WATER BALANCE STUDIES

Distribution of pore water pressure along the landfill capping impermeable layer (*i.e.* the upper surface of the barrier), is complex and an under researched area. Accurate estimation of water rate infiltrating into a landfill cover is a difficult task because it depends on many variables. Estimation of water table behaviour along the landfill capping slope has been investigated by researchers worldwide trying to determine and quantify water balance components (eq. 2.1.)

Precipitation = Runoff + Infiltration + Evaporation + Transpiration + Storage Change (t)

(eq. 2.1)

In order to quantify components of the water balance equation for a landfill cap, lysimeters have been used by Henken-Mellies & Gartung (2004) and Albright *et al.* (2006). Lysimeters can deliver data on evapotranspiration and infiltration but only in one dimension. They give limited information about capping performance in two dimensions (*i.e.* how slope length and gradient control surface runoff, pore water pressure distribution related to slope length and gradient). For a safe, reliable and optimal design of landfill cover layers, complex and detailed water balance analysis is of importance. Detailed analysis of particular capping design should consider a list of variables (Table 2.2) as a function of time and site geographical location. This type of study is time consuming and hence commercially not applicable. Since landfill capping soil layers are commonly thin (0.3 m to 3 m for tree planting areas), hydraulic processes occurring within it are categorised as unsaturated flow. These depend not only on soil type, suctions that are generated by wetting and drying (*i.e.* weather), but also on the undergoing erosion processes and cracking (formation of preferential flow paths).

Some commercially available numerical modelling software allows detailed analysis of seepage infiltration *i.e.* water balance analysis. However for such a study, comprehensive data are necessary, often beyond the resources of a designer. Complex numerical modelling software requires knowledge of numerous variables. These are summarised in Table 2.2 and are divided in three main groups: water balance parameters, soil parameters, slope geometry.

Precipitation

Precipitation is the driving force of the hydraulic processes occurring within the cap. To the Authors' knowledge this parameter is rarely considered when assessing capping stability and drainage requirements. This is understandable due to high variability of this parameter not only across the world (Figure 2.5a) but also across a single country (Figure 2.5b). However, to select an optimal solution for the capping drainage design, consideration should be given to

drainage capacity design in terms of 'storm events' occurring in the region, their return period, and also to average daily precipitation throughout the year and 'semi steady state' conditions that a capping soil layer might develop in a 'wet / winter season'.

no	Water Balance Parameters	Soil Parameters	Slope Geometry
1	Precipitation /Snow cover	Permeability vs suction function	Slope length/height
2	Temperatures	Field capacity	Slope inclination
3	Wind speed	Wilting point	Soil cover thickness
4	Humidity	Porosity	Site latitude
5	Solar radiation	Initial moisture storage	Drainage thickness/ capacity
6	Vegetative growth	Evaporation – evaporative zone depth	
7	Plant transpiration (leaf area index)	Volumetric water content vs suction	
8	Evaporation	Ground surface temperature	
9	Surface runoff	Thermal capacity	
10	Root depth	Thermal conductivity	
11	Barometric pressure		
12	Length of growing season		
14	Time of occurrence for a transient analysis modelling the whole event scenario		

Table 2.2 Parameters required for numerical modelling water balance analysis.
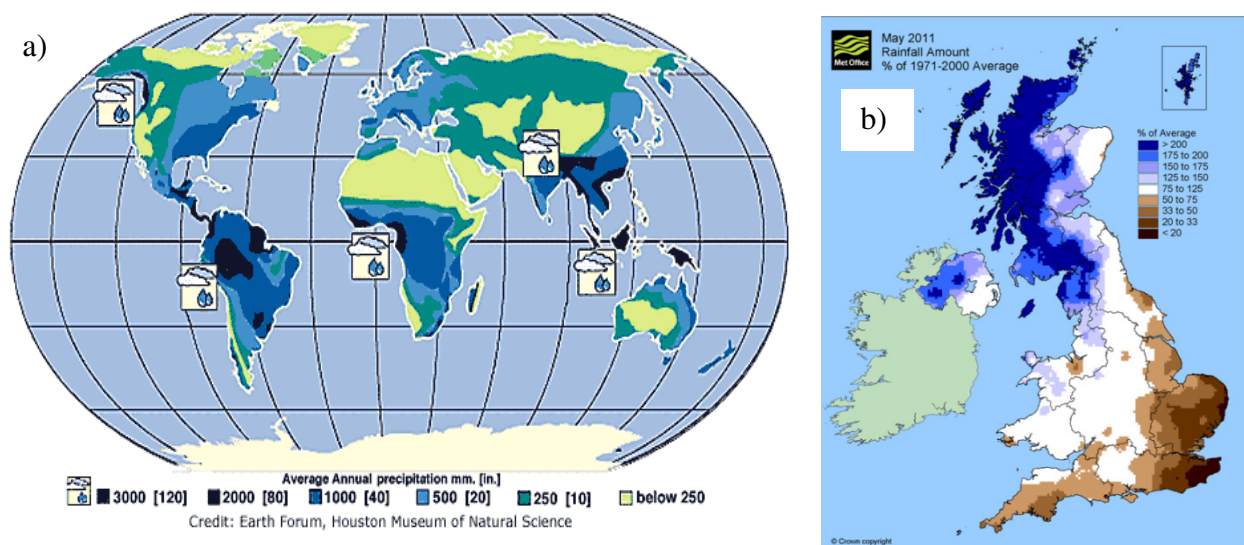
Figure 2.5 Rainfall averages: a) worldwide (USGS 2012), b) UK (MetOffice).

Surface runoff

While precipitation is a common and relatively simple parameter to measure (or obtain information about), estimation of the remaining components of the capping water balance equation are particularly problematic. Water surface runoff (which is directly related to infiltration) is dependent on precipitation amount and length of its occurrence (storm event or long time rainfall), on the type of surface soil, its condition (saturated / unsaturated, permeability coefficient, degree of erosion, desiccation), vegetation (leaf area) and slope inclination and length. Even though the parameter is complex, it is often proposed to assume a value taking into consideration only slope inclination and soil type. For example Koerner and Daniel (1997) after Fenn *et al.* (1975) proposed:

Description of soil	Slope	Runoff coefficient
Sandy soil	Flat ($\leq 2\%$)	0.05-0.10
Sandy soil	Average (2-7%)	0.10-0.15
Sandy soil	Steep ($\geq 7\%$)	0.15-0.20
Clayey soil	Flat ($\leq 2\%$)	0.13-0.17
Clayey soil	Average (2-7%)	0.18-0.22
Clayey soil	Steep ($\geq 7\%$)	0.25-0.35

Table 2.3 Runoff parameter values for varied slope inclinations.

This not only gives a small range for slope inclination variability, but also assumes very low values of runoff coefficient, while other sources: (*e.g.* Mining Department NCB 1982) provide completely different values (Figure 2.6). Although there are a differences in runoff coefficient values both approaches have a rising tendency for run off with increase in slope angle value as expected.
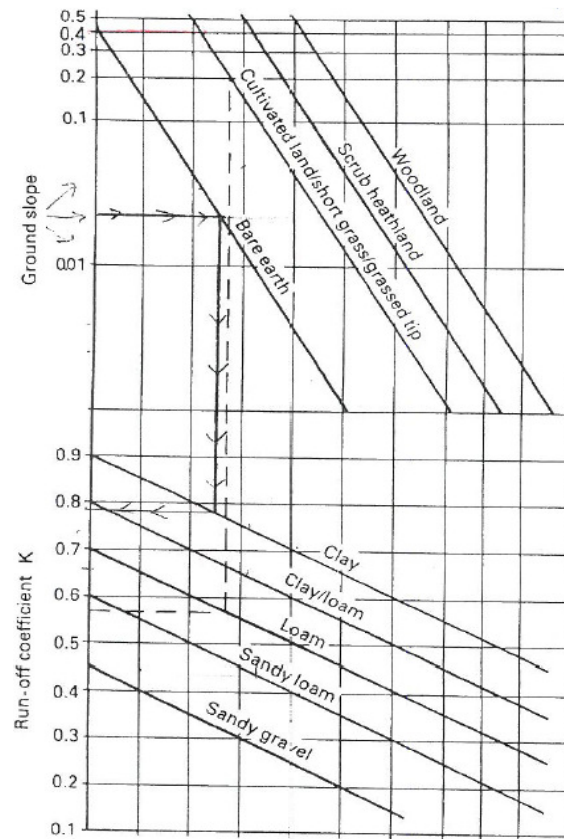
Figure 2.6 Nomogram to determine the runoff coefficient (after Mining Department, 1982).

Infiltration

The downward speed of infiltrating water depends on the texture, structure, stratification (heterogeneity) of the soil, the soil moisture content and the groundwater level. The infiltration capacity of a soil decreases rapidly over time during infiltration. The initial infiltration capacity in dry ground is high, which is caused by high suction of the soil. The infiltration capacity decreases usually until it reaches a constant value approximate to the saturated hydraulic conductivity (the enclosure of air bubbles during infiltration prevents maximum saturation). Some factors affecting the infiltration capacity at the soil surface are: soil compaction caused by ruts, washing of fine particles into surface pores, cracks and fissures (macro pores). In addition, vegetation and soil cultivation will affect infiltration capacity.

Drainage materials

Geosynthetic drain's hydraulic properties are normally available from suppliers in their material specifications. Hydraulic properties describing drainage materials are as follow:

(1)   Flow [$l/m^2/s$] BS EN ISO 11058 – which describes material flow capacity normal to the plane direction;

(2)   In-plane flow capacity [l/s/m width] EN ISO 12958 - which describes flow capacity within the plane of the material for a constant head water flow condition (for hard or soft platens for various pressures, as discussed below);

(3)   Transmissivity [m^2/s]   EN ISO 12958 – which describes flow capacity within the plane of the material under a given head for a particular cross-section area in an isolated condition;

(4)   Water permeability [m/s] EN ISO 11058 - which describes material permeability characteristics normal to the plane direction;

(5)   Permittivity [s^{-1}] ASTM D 4491 – which is an indicator of the quantity of water that can pass through a geotextile under a given head over a particular cross-section area in isolated conditions;

(6)   Coefficient of permeability [m·s] ASTM D 4491 – which is obtained by multiplying permittivity times the nominal thickness of the geotextile.

Often values of in-plane flow are delivered from tests with hard platens. This should be taken into consideration when adopting values for calculations. It was widely discussed that often soft platens or a combination of hard / soft platens represents better *in situ* conditions (*e.g.* Bamforth 2008), and influences the obtained tested values, by decreasing the flow capacity (N.B. tests with hard platens are much more repeatable in the laboratory conditions, and may be more adequate for a quality control tests). Soft platens replicates better behaviour of soil,

intruding the filter into core, but specification for soft platens is not standardised. Additionally, permittivity (eq. 2.2) and transmissivity (eq. 2.3) are often used to describe hydraulic properties of drainage geocomposites (after Koerner 2005):

Permittivity – cross plane permeability of a filter

$$\Psi = \frac{k_n}{t} \ [\text{s}^{-1}] \text{ where} \qquad\qquad (\text{eq. 2.2})$$

k_n - permeability coefficient, normal to the plane direction (m/s)

t   - thickness of geosynthetic (m)

Transmissivity – in-plane permeability of a drainage

$$\Phi = k_p \times t \ [\text{m}^2/\text{s}] \text{ where} \qquad\qquad (\text{eq. 2.3})$$

k_p - permeability coefficient, in-plane direction (m/s)

t   - thickness of geosynthetic (m)

These parameters were developed to improve assessment of drainage capacity. Material thickness can decrease under increasing normal stresses.

Flow in geocomposite

Suppliers provide basic hydraulic parameters for particular materials that are used in hydraulic applications. However, for numerical modelling software, key required parameters are permeability coefficients: k_p and k_n [m/s]. These are derived from the following flow equation:

(1)   Cross-plane flow through geocomposite:

$$Q = k_n \times A \times \frac{\Delta h_g}{T_g} \quad [\text{l/s}], \quad \text{where} \qquad\qquad (\text{eq. 2.4})$$

k_n - permeability coefficient, normal to the plane direction (m/s),

A   - discharge area (m^2),

$\Delta h_g/T_g$ - hydraulic gradient across the geotextile

Hence:

$$k_n = \frac{Q}{A \times i} \qquad [\text{m/s}] \qquad \qquad (\text{eq. 2.5})$$

(2)   In-plane flow through the geocomposite:

$$Q = k_p \times B \times t \;\; \times \frac{\Delta h_g}{L_g} \qquad [\text{l/s}], \quad \text{where} \qquad (\text{eq. 2.6})$$

k_p - permeability coefficient, in-plane direction (m/s),

B   - width of the geocomposite (m)

t   - thickness of the geosynthetic (m)

$\Delta h_g / L_g$ - hydraulic gradient in plane of the geocomposite.

Hence:

$$k_p = \frac{Q}{B \times t \times i} \qquad [\text{m/s}] \qquad \qquad (\text{eq. 2.7})$$

2.4 STATE OF KNOWLEDGE

Commonly, to assess stability of a landfill base, side slopes or capping, well established limit equilibrium methods are used. However it is a geotechnical designer's responsibility to select parameters which accurately reflect *in situ* conditions and to consider aspects that could contribute to construction or in-service failure. For the last two decades knowledge about interface shear strength of lining components (consideration of interface shear strength softening from peak to residual strength), geosynthetic tensile behaviour and waste properties and behaviour, has increased significantly along with understanding of mechanisms resulting from construction. Nevertheless, most reported results are based on laboratory experiments or numerical modelling back analysis of landfill failures and rarely is actual information on mechanical performance of lining components from *in situ* conditions available. Therefore accurate representation / prediction of parameters describing landfill construction components

and their behaviour is still a challenging task with possible severe consequences if wrong parameters and analysis are selected.

In order to assess landfill integrity, limit equilibrium methods are insufficient to deliver complex information about lining component material displacements, deformations, strains and axial forces occurring during and after cell construction. Consequently in recent years attention has focused on developing / improving numerical modelling approaches. However, there is still a lack of validation / verification / calibration against full scale monitored results. Although geosynthetics interface shear strength softening behaviour has been widely investigated, it still remains of question which shear strength value (peak, residual or factored) is mobilised for a certain liner configuration. This depends on site specific conditions such as cell geometry, lining system coverage / exposure, waste properties (unit weight, stiffness and compression due to degradation), method of waste (and drainage layer) placement. There are still significant uncertainties related to many of the material parameters and processes required to analyse and design landfill lining system interacting with waste.

In terms of landfill capping systems, there is a lack of full scale monitoring programmes to verify well established stability calculation methods. Although the main water flow principles are well developed, the capping is a sensitive part of the landfill. Where changes in water regime might alter rapidly and depend not only on capping geometry and soil parameters but also on soil / vegetation / drainage interaction and atmospheric conditions (hence geographical location).

The commonly used capping design process assumes full saturation of the top soil layer, however, there is a lack of actual *in situ* data on pore water pressure distributions along and within the cap and understanding of its relationship to key variables such as geometry, soil properties and weather conditions. Even though geosynthetic drainage materials are

widely applied within landfill capping structures, performance of such materials and correct selection of required parameters are still uncertain and there is a lack of site derived verification. This research broadens knowledge on geosynthetic *in situ* performance.

3   METHODOLOGY AND RESEARCH TASKS

3.1   INTRODUCTION

This chapter explains and justifies the methodology which was undertaken in order to investigate the main topics. Details about the projects and the way of handling the solutions are briefly described in this chapter, since this part of the research has been published, and relevant papers with comprehensive information are included in Appendixes B / Paper 2 and Appendix C / Paper 3. Full references are made to the papers in the text. Since only limited information about the capping project has been published to date (Appendix D / Paper 4), attention has been given to describe the main aspects of this study in the thesis.

3.2   OVERVIEW

This study constitutes in part **qualitative** research. Through experimental approach it aims to investigate material in service performance and reaction to imposed forces/conditions, and hence to deliver improved understanding on the materials *in situ* performance. However, the part of the study related to numerical modelling with numbers of calibration and sensitivity analyses constitutes **quantitative** research**.**

Much effort was given to collect reliable data from installed instrumentation in order to understand mechanisms involved in the processes occurring on the sites. Further analysis was conducted in order to investigate and examine this behaviour in numerical modelling. As a result of developments in the technology and computing areas, numerical modelling is applied to assess the performance of landfill lining systems where the standard approaches are insufficient. In particular, it is used to predict stresses and strains in lining elements resulting from waste placement and settlement. In terms of landfill capping system designs and adequate drainage selection, methods are based on a limit equilibrium analysis. The accuracy of the approach is questionable as capping failures keep occurring and little attention is given to research in this area. It has been emphasised by many researchers that there is a lack of site

derived data regarding geosynthetic performance. Therefore, to understand better the main mechanisms controlling materials performance it was decided to conduct site experimental studies. Two full scale experiments are the main pillars of this research; these were supported with laboratory investigations with the aim of collecting data for use in further numerical modelling. Through installation of various measuring instrumentation, data on mechanisms and factors controlling specific material behaviour were collected. This work comprises two main topics: investigation of geosynthetics mechanical behaviour on a landfill side slope and investigation of geocomposites hydraulic performance on a landfill capping. Both of the projects involved some laboratory investigations, also both projects involved numerical modelling investigation and comparison between predicted and measured behaviour.

Figure 3.1 presents the research process related to both studies. The main methodology approach is similar for both projects: through full scale site investigations, supported with laboratory tests of material properties, a numerical model was set up and data obtained from the *in situ* measurements and the model were compared. Conclusions on factors controlling behaviour have been obtained, discussion on design process / criteria was carried out and areas for further research specified.
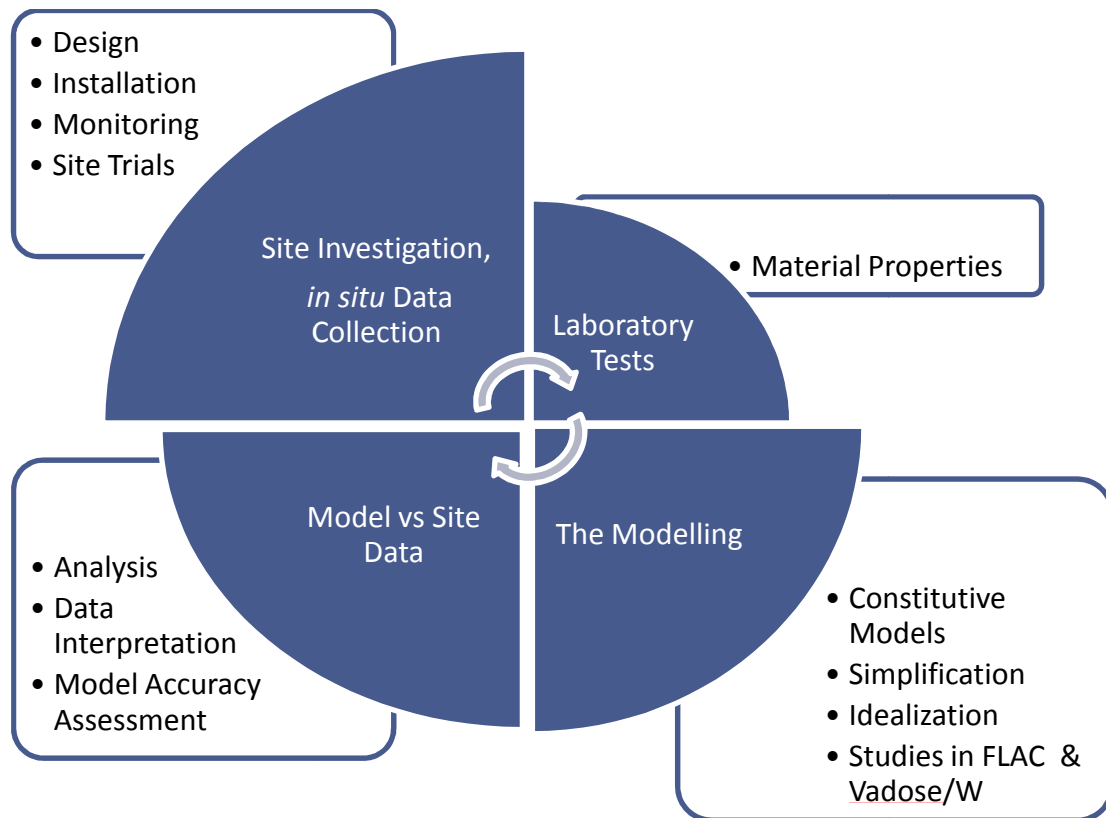
Figure 3.1 General schematic presentation of the projects process / stages.

3.3   RESEARCH TASKS

Table 3.1 presents the summary of the research task methodology and related objectives, with where applicable published papers identified. The tasks are divided in two main groups: 'M' related to the side slope Milegate project, and 'C' tasks related to the capping project.

Research task	Research methodology	Objective	Paper
Literature review, state of the art, of the research	Literature review	1	Dixon *et al.* 2012 (Paper 1 / Appendix A)
M:Design field instrumentation	Literature review / Experimental research	1	Zamara *el al.* 2012 (Paper 2 / Appendix B)
M:Instrumentation installation	Field works / Experimental research	1	
M:Instrumentation monitoring, Maintenance works	Field works / Experimental research	1	
M:Field trials (sand veneer)	Field works/ Experimental research	1	
M:Laboratory testing programme	Experimental research	2	Zamara *el al.* 2013 (Paper 3 / Appendix C)
M:Data analysis	Experimental research	3	
M:FLAC modelling results analysis	Experimental research	2	
M:HDPE thermal behaviour / wrinkles investigation	Literature review	3	
C:Literature review, capping water balance	Literature review	4	Zamara *et al.* 2011
C:Data collection for Vadose/W analysis	Literature review	4	
C:Vadose/W PSR modelling	Experimental research	4	Zamara *et al.* 2011
C:Design field instrumentation	Field works/ Experimental research	5	Zamara *et al.* 2012b (Paper 4 / Appendix D)
C:Instrumentation installation	Field works/ Experimental research	5	
C:Instrumentation monitoring, Maintenance works	Field works / Experimental research	5	
C:Laboratory tests	Experimental research	6	
C:Field trial (bowser)	Field works / Experimental research	5	
C:Vadose modelling results analysis vs monitoring data	Experimental research	7	

Table 3.1 Research programme (note: M – side slope project tasks, C – capping project tasks).

3.4 SIDE SLOPE LINING SYSTEM INVESTIGATION - MILEGATE EXTENSION LANDFILL SITE PROJECT

This part of the research focused on collecting information on mechanical performance of geosynthetic materials. Figure 3.2 presents a scheme of the main aspects of this part of the study. Collection of site derived data constituted the main part of this project, supported by laboratory works and followed by numerical modelling. The numerical model was built using a commercially applied method developed by Fowmes *et al.* (2007).
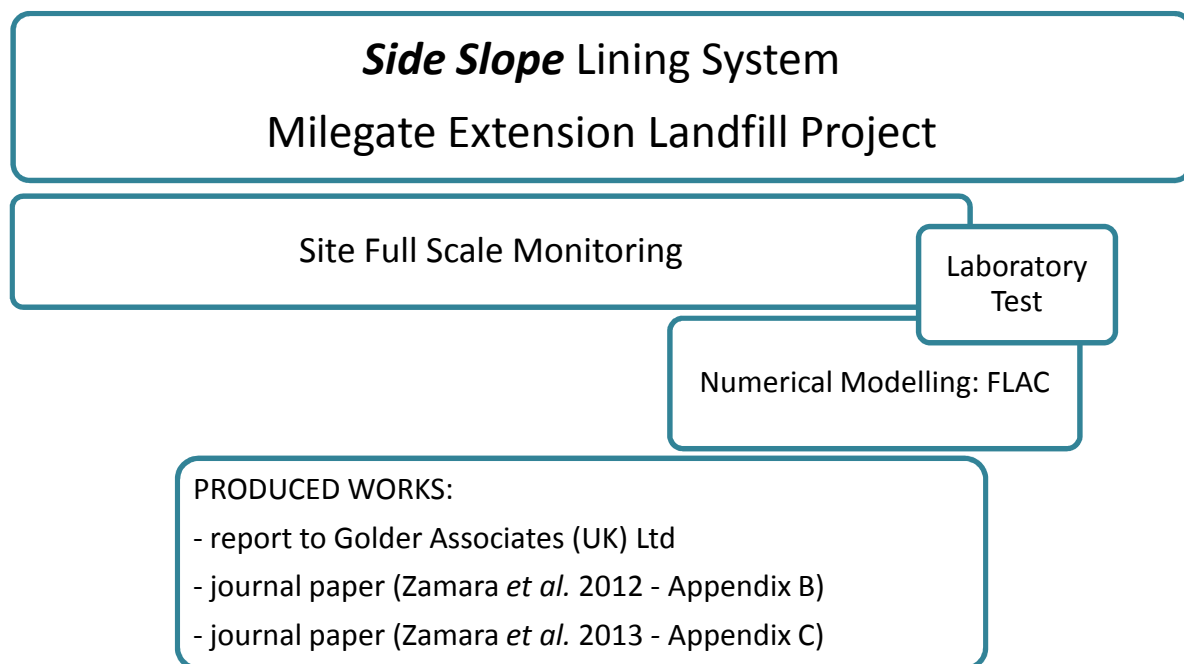
Figure 3.2 Schematic representation of the main stages of the landfill side slope studies.

The investigated site is located at Milegate Extension Landfill Site, Yorkshire, UK. The study was carried out in cooperation with:

(1) Sandsfield Gravel Company Ltd – owner and operator, access to the site & technical assistance;

(2) Golder Associates (UK) Ltd – project consultancy and EngD sponsor;

(3) GeoFabrics Ltd - geotextile supplier;

(4) Veolia Environmental Ltd – geomembrane donor.

3.4.1 SITE INSTRUMENTATION

To meet the research objectives, various instruments were deployed on the side slope lining system. The key parameters calculated in the design process that constitute the main controlling factors of the design monitored on the site are:

(1) Geosynthetics displacements (and relative displacement in between materials);

(2) Geosynthetics strains;

(3) Loadings imposed on the liner;

(4) Temperature - throughout the project, high importance of temperature factor was observed, hence additional measurement of temperature was carried out.

Installed instrumentation needed to be robust and reliable. It was known that hardly any reparation or amendments would be possible after instruments installation due to waste coverage. In order to collect reliable and consistent data it was important to provide cross checking of measurements between methods. This allowed comparison of the readings measured using methods with associated various resolutions and accuracies. This helped to determine error readings or inaccuracies related with instrument performance. Paper 2 included in Appendix B details philosophy of instrumentation selection, measured parameters, methodology and discusses quality of data obtained from monitoring which complements information included herein.

3.4.2 LABORATORY TESTS

Laboratory works comprised an important part of this study, as it produced information on site specific interface shear strength properties between the lining system materials. These were further used in the numerical modelling analysis. It is of significant importance to use site specific parameters in design process, hence the interface properties were investigated using the most common and widely used method - large scale direct shear box machine.

Within the last decade emphasis had been placed on the importance of using site specific parameters of geosynthetic / geosynthetic or geosynthetic soil interfaces in the design process. Although many interface test results are reported in the literature (Dixon *et al.* 2006), variability of materials available on the market is still significant and implementation of these in a specific design is incorrect and not recommended.

3.4.3 FLAC MODELLING

The aim of this study was to validate / verify the numerical modelling design approach that is commercially applied. The importance of the numerical modelling analysis in landfill engineering, along with up to date development and undertaken research within the area of landfill modelling validation is presented in Appendix A / Paper 1, Section 3 and 4.

In this study commercial program FLAC (Itasca 2002) has been used to analyse lining system performance. FLAC has a proven applicability within landfill geotechnical engineering topics through several previous studies (*e.g.* Fowmes *et al.* 2007, Arab 2011, Sia 2007). Due to a feature that enables modelling of materials that undergo large strains, FLAC can be used for modelling landfill geotechnical problems (*e.g.* waste body deformation, geosynthetics strains). Additionally, this software allows creation of user-defined sub-routines that can incorporate formulations. This can add flexibility and accuracy to the analysis and hence design.

The FLAC solution procedure consists of the following steps:

(1)   From the applied forces and stresses, velocities and displacements are calculated with equilibrium equation (equation of motion);

(2)   Strain rates are evaluated again using equilibrium equation (equation of motion);

(3)   Consequently new stresses and forces are calculated from the strain rates using stress-strain relationships (constitutive equation).

Since waste / barrier interaction is a key landfill design concern, the primary aim of this study was to better understand mechanisms influencing geosynthetic performance, to validate commercially applied numerical modelling software and verification of ability to reproduce the *in situ* material behaviour (see Section 1.4).

The methodology describing modelling process is summarised in Figure 3.3, and was developed in accordance with the following stages:

(1)   Basic approach

A standard design method was used that is commonly applied within commercial project. This models the interface shear strength as constant values: peak or residual, with no progressive failure considered. In addition waste is represented by Mohr-Coulomb criteria, with strength values derived from the literature;

(2)   Tension/compression subroutine

Geosynthetics tension/compression stiffness moduli were incorporated into the model. It has been recommended by Villard *et al.* (1999) and Fowmes *et al*. (2007) to represent geosynthetics in compression with a decreased stiffness value (ten times lower than modulus in tension) to express compressive behaviour (wrinkle formation). Nevertheless, this is still a simplistic approach, which potentially requires improvement;

(3)   Waste volumetric hardening subroutine

This subroutine takes into account waste stiffness increase with depth and compaction under self-weight. This method simulates in a limited manner the aspect of landfill waste behaviour during waste placement;

(4)   Interface strain softening subroutine

An interface strain softening code fully dependent on the relative displacement between lining components was implemented. Input parameters obtained directly from the laboratory test

results were applied in the model. The FLAC procedure was based on the approach developed by Fowmes *et al.* (2007);

(5)   Additional studies

Due to limited and inconsistent agreement between measured and computed lining system displacements, additional analyses were undertaken to better understand the mechanisms controlling *in situ* behaviour. These include a basic thermal analysis of high density polyethylene (HDPE) geomembrane compression / extension behaviour and wrinkle formation (detailed information on geomembrane and geotextile thermal response is included in Appendix C / Paper 3, Section 5). In addition, modifications were made to the basic model to investigate the sensitivity of in-put key material properties (*e.g.* geotextile tensile properties, sand stiffness, clay stiffness) and interface shear strength properties between the liner components.
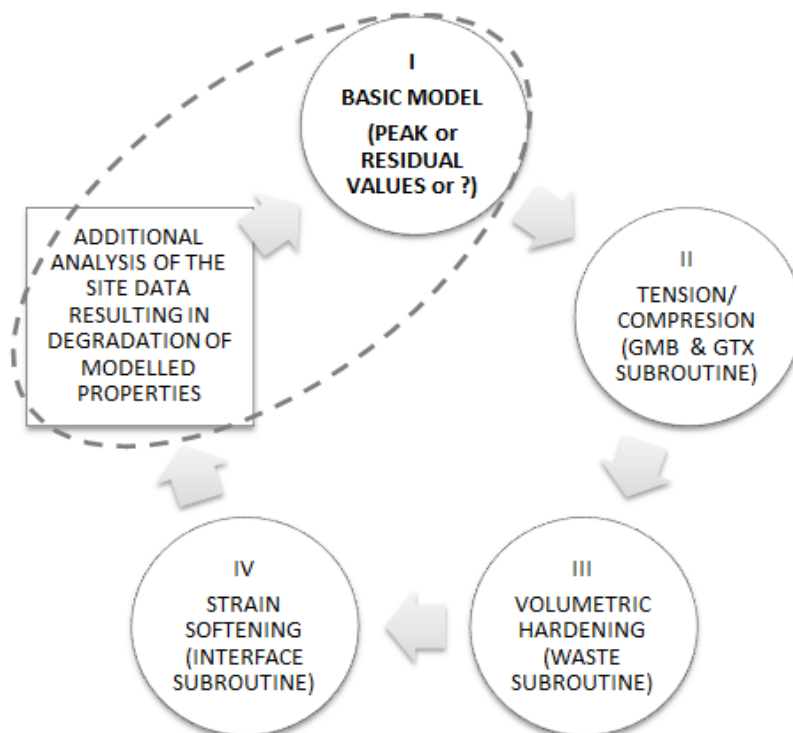
Figure 3.3 Numerical modelling approaches undertaken to reflect Milegate lining system displacement mechanism.

3.5 **LANDFILL SITE CAPPING SYSTEM STUDIES**

This part of the study was focused on investigating soil pore water pressure distributions along landfill capping liners and its influence on the stability and drainage system requirements. Figure 3.4 schematically presents the main stages of the work involved in the landfill capping studies. Initially a site trial was unattainable, therefore the study was focused on capping water balance analysis, review of the crucial factors and analytical considerations.

Further geometrical sensitivity analyses were carried out in numerical modelling software Vadose/W (component of GeoStudio 2007 programme). Vadose/W was selected due to its features allowing simulations of unsaturated water flow in two dimensions and application of user defined climatic boundary conditions along with providing tools for slope stability analysis. Critical variables for the simulations were selected and their importance in terms of pore water pressure development was assessed. Attention was paid to atmospheric conditions, commonly designed slope inclinations, soil hydraulic properties and drainage material properties.

These analyses were further supported with a field trial, which provided the possibility of verifying the modelled behaviour with *in situ* performance. Again, much effort was expended to deliver valuable and reliable data on hydraulic performance of various drainage geosynthetic materials under in service conditions. Additional site and laboratory investigations were carried out in order to deliver additional information about the capping material properties, required by the numerical modelling.

Landfill *Capping Drainage System* Investigation

Geometrical Sensitivity Analisis in Vadose/W

Site Full Scale Monitoring Trial

Laboratory / Site Tests

Numerical Modelling: Vadose/W

PRODUCED WORKS:

- report to Golder Associates (UK) Ltd

- conference paper (Zamara *et al.* 2011)

- conference paper (Zamara *et al.* 2012 - Appendix D)

(- planned journal paper 2013)
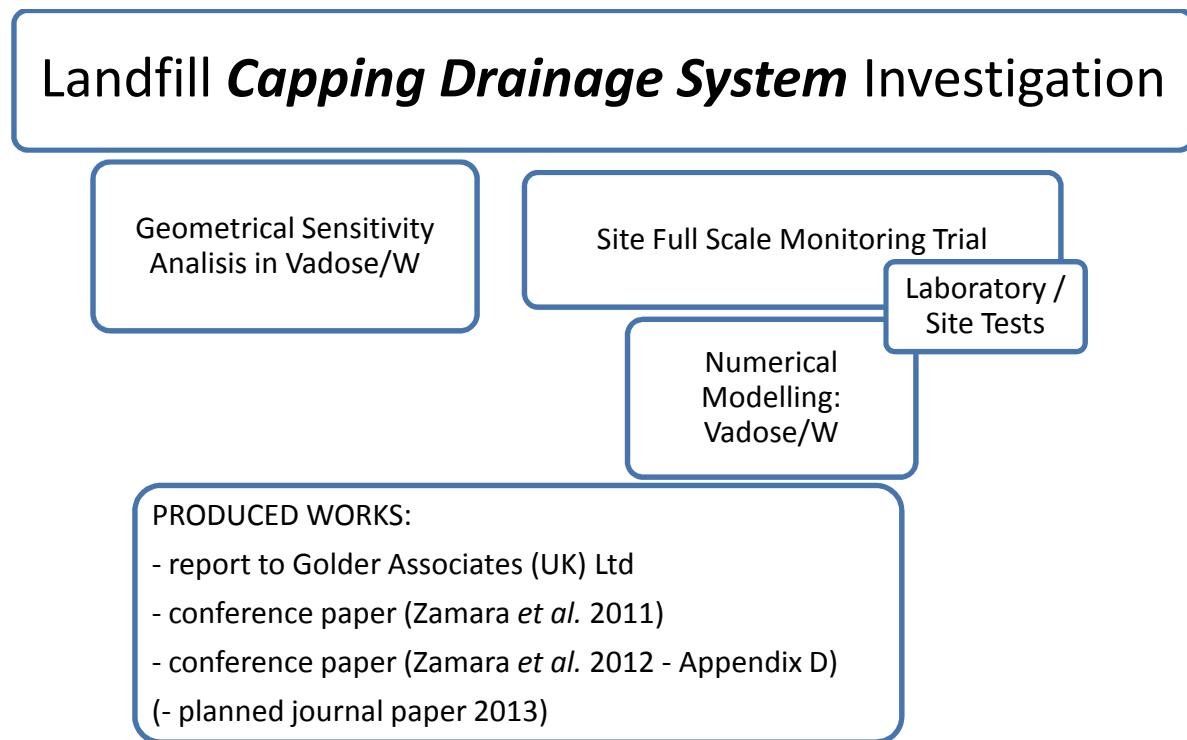
Figure 3.4 Schematic representation of the main stages of a landfill capping studies.

The capping full scale monitoring project is located at Bletchley Landfill, UK site and was carried out in collaboration with:

(1) FCC Environment Ltd – owner and operator, access to the site & technical assistance;

(2) Golder Associates (UK) Ltd –EngD sponsor;

(3) GeoFabrics Ltd – drainage material supplier and equipment sponsor.

3.5.1 NUMERICAL MODELLLING WITH VADOSE/W

It was decided to investigate hydraulic regimes of capping systems using Vadose/W, which is a component of GeoStudio (2007) - software for geotechnical and geo-environmental analysis. Vadose/W allows analysis of the flow through unsaturated zones due to surface transient boundary conditions *i.e.* fluxes dependent on atmospheric and surface conditions (*e.g.* vegetation - root transpiration, surface evaporation and weather conditions – temperature, humidity wind speed and precipitation). In general Vadose/W is based on Darcy's law for the saturated flow, but modified unsaturated flow using the Richardson

equation, where permeability is directly related to soil water content and pore water pressures (GeoStudio 2007).

The study of landfill capping hydraulic regime started with general water balance analysis based on Koerner and Soong (1997) approach. However, this could only provide general information about amount of water entering the system. No consideration to pore water pressures development and distribution along the capping was given,

Next it was decided to investigate HELP model (The Hydrologic Evaluation of Landfill Performance Model) which is often applied to estimate amount of leachate produced within landfills However again this quasi two dimensional model delivers data on overall landfill water balance, actual water accumulation and pore water pressure build up along capping is beyond the scope of the software.

Further it was decided to investigate the subject in numerical modelling software. The study on SEEP/W - GeoStudio (2007) component started. GeoStudio is a widely known and commercially applied programme. This user friendly software allows to investigate groundwater seepage, pore water pressures development and dissipation. Moreover, the software allows analysis of saturated and unsaturated conditions in steady state or transient conditions. Water entering the capping system is described in terms of inflow, which for transient analysis requires a function of water inflow versus time. However, it was not possible to investigate components of water balance equation such as: evaporation and run-off.

Hence, it was decided to use Vadose/W which allows applying atmospheric conditions directly to the model and all of the components of the water balance can be directly computed.

Bohnhoff *et al.* (2009) reported Vadose/W verification/comparison of modelled and monitored water balance parameters (data from a test section located in a semiarid climate simulating a monolithic water balance cover) over a period of time. These provide general

information on software adequacy in terms of replicating *in-situ* conditions, degree of accuracy of the computed values (under / over estimating certain values) and soil water content due to atmospheric conditions changes. Also much attention has been given to model capping with a capillary break effect (*e.g.* Aubertin *et al.* 2009) in Vadose/W. To the Authors's knowledge no data has been reported in the literature on modelling of field scale capping system performance including geosynthetic drainage layer responding to specified atmospheric conditions.

For permanent / steady state water conditions, it is possible to produce an optimal design of drainage systems. However, in terms of optimising capping design, highly transient processes including increased unsaturated flows and transient analysis should be represented, in order to provide realistic pore water pressures distributions, which consequently controls slope stability. Vadose/W allows transient simulations through use of time steps and selection of input data variable in time.

The analysis was carried out within top soil placed on an impermeable material forming the barrier (*i.e.* compacted clay layer or geosynthetic membrane). It is understood that infiltration into the landfill might occur on-site but the processes related to the underlying waste behaviour (*e.g.* percolation, settlements, waste temperature effect) are not considered in this study and are expected to have a minimal influence.

3.5.2 FULL SCALE EXPERIMENT – BLETCHLEY LANDFILL SITE PROJECT

It has been highlighted that data acquired from the field study, of the capping system in-service conditions, is of the significant importance. It is not only important for numerical modelling verification but also specifically to understand water table behaviour, mechanisms of its development and driving forces of capping pore water pressure distributions. This part of the project was focused on monitoring geosynthetic drainage performance at the Bletchley

Landfill which is located near Milton Keynes, Buckinghamshire, UK. The site is currently an active landfill but the study was carried out on the newly built cap of a recently closed cell.

Instrumentation was designed and installed on the site in a manner to monitor parameters of the capping water balance. Additional information regarding site instrumentation is presented in Appendix D/Paper 4.

Figure 3.5 describes the philosophy of instrumentation selection. Parameters of IN-PUT and OUT-PUT categories were considered as essential to monitor in order to answer the main research questions, while MECHANISM category parameters were advantageous to monitor, but were considered secondary.
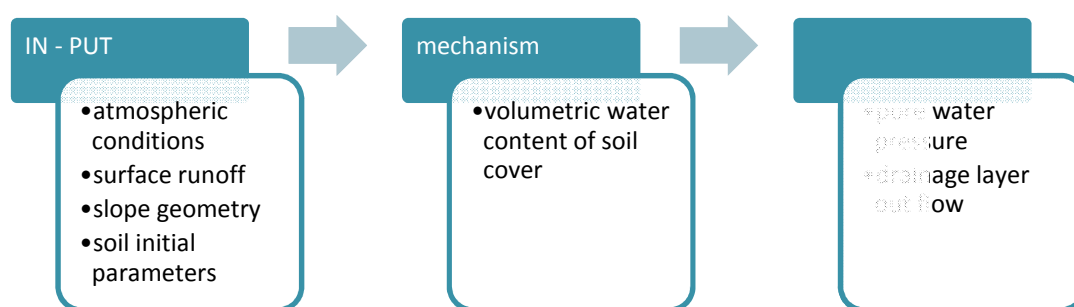
It was decided to install instrumentation on the site in three phases, this was due to the expected low permeability of the cover soil. It was not certain how the capping soil and drainage material would respond to atmospheric conditions. The aforementioned three stages comprised:

(1)   Instrumentation required in order to assure the stability of the capping soil cover (pipe system collecting drained water).

(2)   Instrumentation monitoring water table (stand pipe piezometers) and soil water content (volumetric water content sensors).

(3)   Instrumentation logging water draining from the drainage layers (considered were V-notch weirs with water table loggers or ultrasonic Doppler instruments).

First stage instrumentation was compulsory to deploy due to risks involved in the capping stability, and this was carried out immediately after geocomposite installation. It was not desirable to allow uncontrolled water flow from the drainage panel to lower parts of the capping slope, which could cause increased saturation of the capping lower sections. A system of pipes and water containers was installed to divert drained water from the bottom edge of geocomposite to the toe of the slope.

Second stage instrumentation allowed monitoring of the mechanism involved in the geocomposite in service performance, soil water regime and pore water pressure condition above the drainage layers.

Knowledge of the pore water pressures acting on the low permeability barrier is critical when estimating veneer cover stability. Current design calculations often assume the worst case scenario (this usually assumes a fully saturated cover layer) and target a satisfactory factor of safety against slippage. Second stage instruments were installed shortly after geocomposite installation on the site and comprised of standpipe piezometers and volumetric water content sensors (water content reflectometers, model CS616, Campbell Scientific). The volumetric water content reflectometer sensors are widely used in agriculture and soil sciences, although their accuracy depends on a number of factors: method of insertion, maintaining the geometry of the device, surrounding soil conditions, inserting sensors without air void generation, maintaining original soil density and removing stones (Campbell Scientific 2011). Nevertheless, data obtained from these sensors can provide useful information about the soil response time to atmospheric conditions, moisture translocation across the soil layer and soil moisture conditions above geosynthetic drainage materials.

Decision on the **third stage** instrument installation was made after a period of observation and the instrumentation suitability was confirmed (*i.e.* if no drainage outflow was observed, there would be no need for flow measurement installation). A significant period of the monitoring was during the driest winter season for decades, reported by MetOffice (2012): "Many counties in eastern and southern England and eastern Scotland had a dry January and a particularly dry February. Over England, it was the driest February since 1998 generally, and East Anglia and Lincolnshire had one of their driest Februarys on record".

Initially there was no need to install a logging system for the flow. It was decided to monitor drainage performance on days with increased precipitation using a standard manual method: volume per time unit, measurements. In further studies, difficulties relating to data collection were faced, as a result only limited data on the *in situ* drainage system performance were collected manually.

3.6 **SUMMARY**

This chapter summarises the methodology adopted to meet the main objectives of this study. Two full scale experiments were designed, and conducted on the sites in order to install instrumentation which can monitor geosynthetics performance.

Through the Milegate Landfill full scale monitoring programme and additional laboratory tests, information on waste barrier in-service performance was collected. This allowed further numerical analysis of site specific data using commercially applied design approaches used in landfill engineering sector.

The capping studies were undertaken in Vadose/W to investigate pore water pressure distribution above the capping barrier layer. A full scale study at Bletchley landfill, allowed monitoring of in-service performance of three different drainage materials. This allowed investigation of the monitored behaviour in Vadose/W and enabled formation of conclusions based on the monitored and modelled results, which are discussed in the Chapter 5.

4 THE RESEARCH UNDERTAKEN

This chapter summarises fulfilled tasks and describes details of the undertaken site, laboratory and numerical modelling analysis.

4.1 OVERVIEW

Both of the site experiments: Milegate Extension Landfill (side slope) and Bletchley Landfill (capping) in general present similar philosophy of works undertaken. Nonetheless, timescale and tools needed to carry out successfully both tasks were completely different. Table 4.1 briefly summarises / compares key aspects of this research. In general, the scope of works undertaken for both studies is as follows:

Site works

(1) Design

This comprised consideration of the material locations, configuration, stability risk assessments, instrumentation selection and design, consultation with suppliers, laboratory testing and preparations.

(2) Materials installation

This comprised of liaising with site management and contractors about timescales and general workforce availability. Liaising with material suppliers and coordinating material deliveries.

(3) Instrumentation installation

This comprised of installation of a wide range of instrumentation for both projects and liaising with contractors regarding assistance.

(4) Maintenance works and sample collection

This included regular site visits to ensure adequate performance of instrumentation and additionally soil sample collection for laboratory works.

(5)   Additional trials

Liaising with management / contractors in ordered to undertake additional works, acquiring equipment (excavator / water bowser) with operators and materials (soil / water).

Data collection

This included regular trips to the sites in order to take readings, including all day trials.

Laboratory works

(1)   Milegate Extension Landfill Project - Investigating the interface shear behaviour between barrier components;

(2)   Bletchley Project - Investigating soil permeability tests, volumetric water content, and other parameters.

Data analysis

Comparison of results from different instruments, temperature corrections where applied (*i.e.* extensometers wire exposed to three different temperatures: exposed wire, wire under sand layer and wire under sand and waste), analysis of parameter relationships, trends and tendencies.

Numerical modelling

Analysis in order to better understand the mechanisms, driving forces incorporated in the system performance along with verification and validation of the numerical models:

(1)   Milegate Extension Landfill Project – FLAC;

(2)   Analyses were carried out in Vadose/W to investigate pore water pressure behaviour for various conditions;

(3)   Bletchley Project – Vadose/W.

	Milegate Extension Landfill	Bletchley Landfill	Vadose/W studies
Landfill construction section	Side slope	Capping	
Investigated properties	Mechanical	Hydraulic	
Software used for modelling	FLAC 4	Vadose/W	
Laboratory works	Interface shear strength	Hydraulic properties of soil, geotechnical parameters of soil	Parameters based on literature
Measured parameters	- Geosynthetic displacements, - Relative displacements, - Geosynthetic strains, - Normal stresses imposed on the liner, - Geosynthetic temperature.	- Piezometric level, - Soil volumetric water content, - Atmospheric conditions, - Surface runoff, - Drainage out-flow.	- Pore water pressure.

Table 4.1 Summary of the project approaches.

4.2 LANDFILL SIDE SLOPE LINING SYSTEM – MILEGATE EXTENSION LANDFILL PROJECT

4.2.1 MONITORING PROGRAMME

The materials and monitoring instrumentation was installed on the site in the first year of the research in 2009 and readings were collected from this period until the end of the 2012. Figure 4.1 presents the process of materials and instrumentation installation and the final result – the slope general view. This has produced data for 3.5 years of monitoring. The cell was filled slowly and became full only in the final months of the study. Details of instrumentation installation, location, accuracy of the collected readings, issues related to instrumentation performance, along with details regarding materials and history of undertaken works are reported in Appendix B / Paper 2, Section 2 to 5. Figure 4.2 presents schematic stages and timing of the landfill cell filling process.

Milegate lining system comprises 1 m thick clay barrier layer. One of the originally designed and installed 5 m wide geocomposite panels was removed and in that place a 2 mm

double textured HDPE geomembrane overlaid by a nonwoven, needle-punched geotextile HPS 14 were installed. The geosynthetic lining system deployed during the experiment was placed in addition to the clay liner, and therefore is an additional, and hence sacrificial, layer that does not form part of the approved containment system.

Geosynthetics were covered by a sand layer 0.5 m thick, which simulates placement of a drainage layer over the side slope. Figure 4.3 presents schematically configuration of the instrumented lining system. The landfill side slope was instrumented with sensors monitoring geosynthetics displacements, strains, stresses imposed on the lining system and temperature at the clay surface and geomembrane/geotextile interface. Figure 4.4 schematically presents instrumentation locations along the slope.
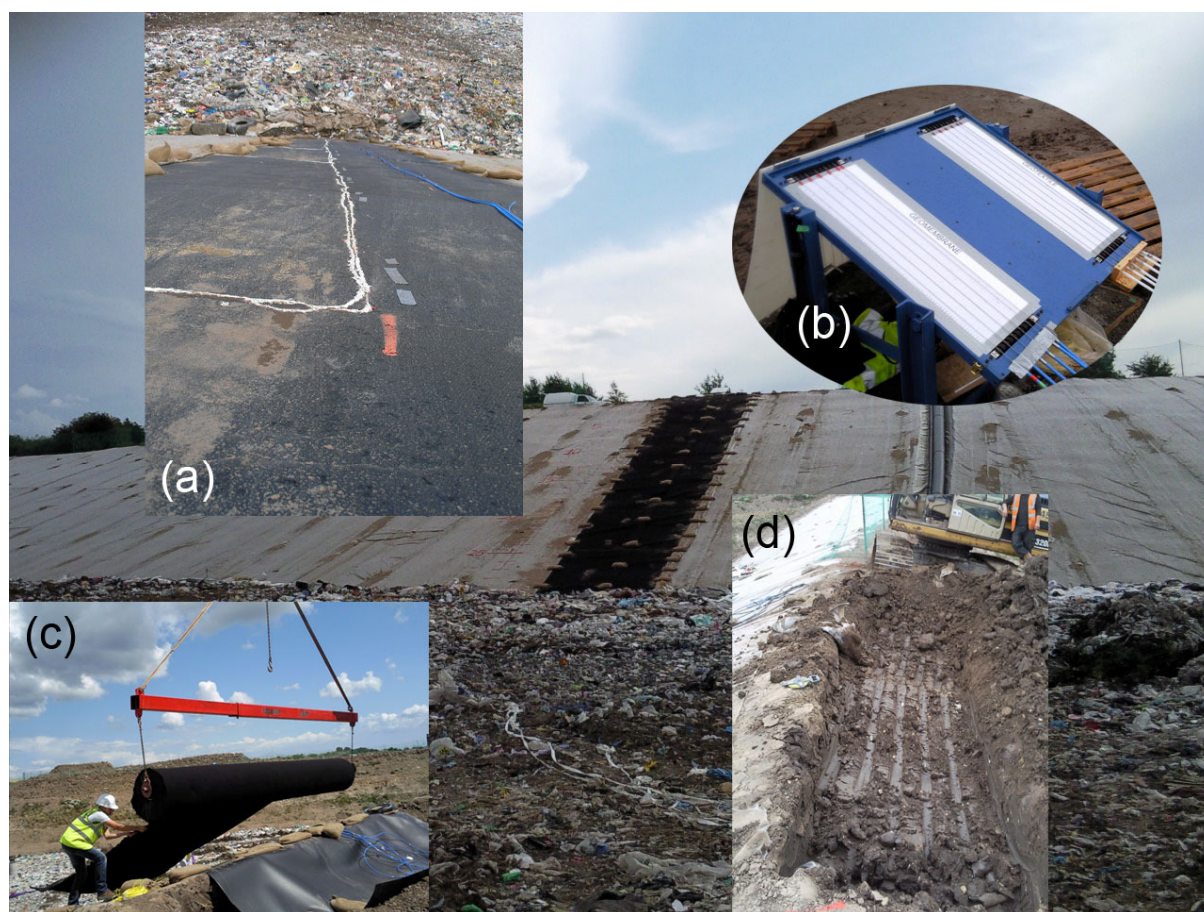
Figure 4.1 Installation of materials and monitoring instrumentation (a) slope general view-instrumentation on geomembrane; (b) measuring station; (c) geotextile deployment; (d) excavation of anchored trench.

To measure geosynthetics displacements and relative displacements between materials, wire extensometers were installed on the geomembrane and geotextile. This method was previously reported by Bouthot *et al.* (2003)*,* Gourc *et al*. (1997), and Fowmes *et al.* (2007). Additionally, measurements of geosynthetic displacements provide information on magnitude of strains.

Strains were also measured by fibre optic Bragg gratings (middle and lower section of the slope) and Demec strain gauges (top section of the slope). Although fibre optic strain gauges are considered to be brittle sensors, their applicability was demonstrated in previous studies (*i.e.* Nakamura *et al.* 2006, Nancey *et al.* 2007). Fibre optic sensor configuration, location and protection on the slope was directly consulted and installed by the Photonics Group from Cranfield University. Details regarding optic sensors installation are widely described in Appendix B / Paper 2, Section 4.3.2. Additionally, the collected information is discussed in the same paper in Section 4.3

Strains were also measured using Demec strain gauges. Due to the nature of Demec gauges (direct readings at the location of its installation for which access is required), they were installed only in the top section, which remained uncovered by overburden loadings for most of the monitoring period. Demec strain gauge measurement method is widely adopted for strain measurements of concrete structures. This is a simple and cost effective method of data acquisition. To the Author's knowledge this is the first time that this method has been used in a landfill environment to measure geomembrane strains. The method is described wider in the Paper 2 / Appendix B, Section 4.3.3. Simplicity and the cost of the method is an advantage, however the method requires direct access to the monitoring points and might be applied only within exposed sections of a geomembrane.
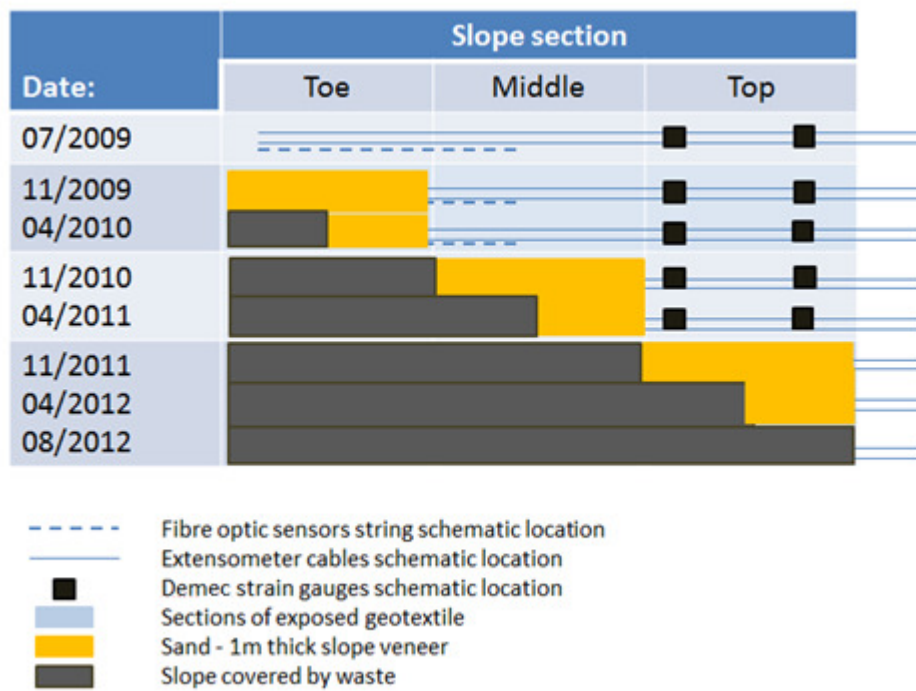
Figure 4.2 Schematic representation of the slope coverage timescale.
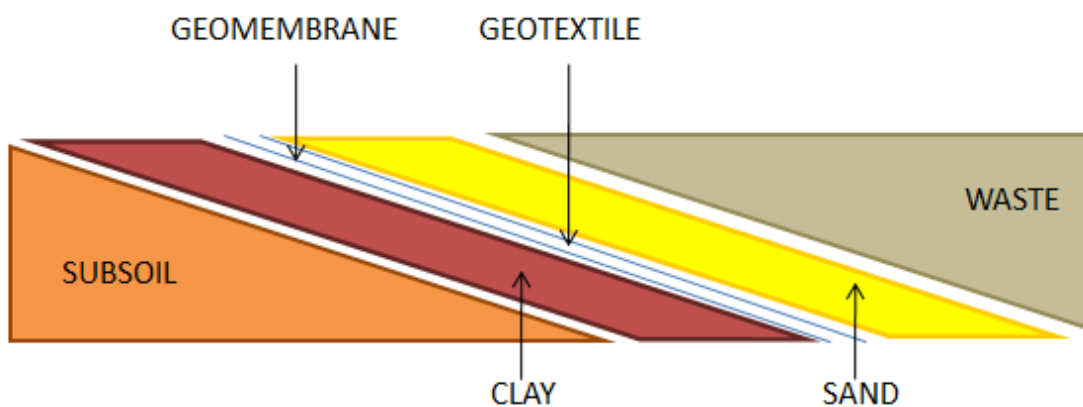
Figure 4.3 Milegate lining system schematic view (after Zamara *et al.* 2012).
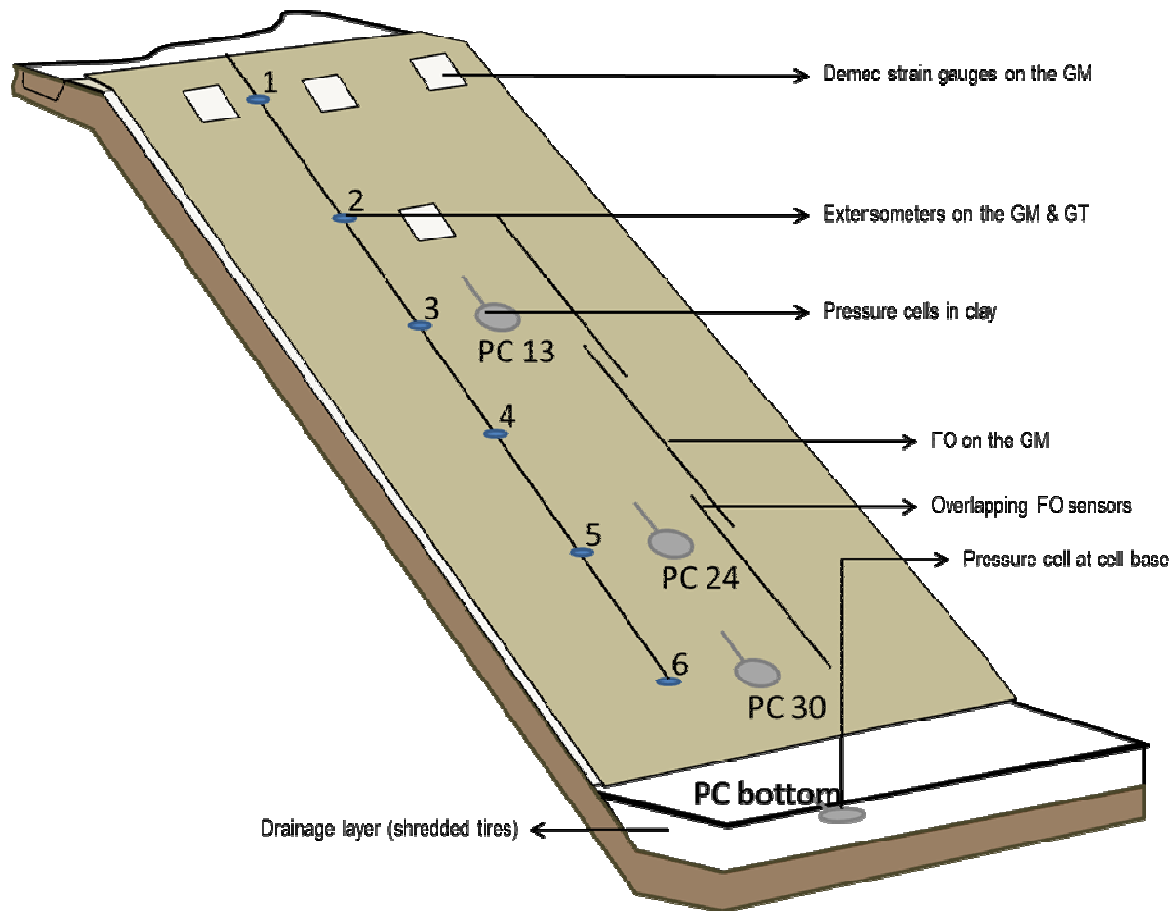
Figure 4.4 Milegate instrumentation – schematic view (after Zamara *et al*. 2012); note: GM – geomembrane, GT – geotextile, FO – fibre optics, PC – pressure cell.

Loadings imposed on the lining system were measured by pressure cells installed in three shallow excavations along the panel centre line in the top of the clay layer. Pressure cells are commonly used to measure distribution, magnitude or direction of total stresses in soil constructions, optionally they may include thermistors measuring temperature. It was decided to install vibrating wire pressure cells with thermistors on the slope to not only monitor loadings imposed on the lining but also to measure temperature at the clay / geomembrane interface. More information about the instrumentation performance is included in Appendix B / Paper 2, Section 4.1 and Section 5. Whereas, all the monitoring data collected throughout the period of the study are discussed in Appendix C / Paper 3, Section 4.1 and presented in Appendix C / Paper 3, Figure 4. This shows good agreement between

measured and modelled response, what indicates adequate waste loading representation in the model.

In early stages of the project the importance of air and geosynthetic temperature logging was acknowledged, hence additional temperature monitoring on the geomembrane surface was carried out, primarily to allow instrumentation temperature corrections, but also to help interpret geosynthetic behaviour in response to thermal changes.

4.2.2 LABORATORY TESTS

In order to collect information on the interface shear strength between lining system components, direct shear box tests were carried out in a laboratory with controlled temperature ($21° \pm 2°$), using a large direct shear box device (300 mm x 300 mm x 100 mm). The tests with five different normal pressures (10, 25, 50, 100, 200 kPa) were undertaken on three interfaces: sand / geotextile, geotextile / geomembrane and geomembrane / clay. Tests were carried out with various conditions: sand / geotextile, geomembrane / geotextile – dry and submerged, geomembrane / clay – undrained and drained. Undrained samples were sheared with speed of 1mm/min. Drained tests on clay / geomembrane interface were simulated by the extended time (24h) of initial sample loading and shear speed of 0.01mm/h. Each test was repeated a minimum of three times on a new geosynthetic and soil samples. Figures 4.5 and 4.6 present an example of data derived from series of tests performed on dry geomembrane / geotextile interface. Figure 4.5 presents the strain softening behaviour of the geosynthetic / geosynthetic interface. After reaching peak, shear strength decreases to a residual value. Figure 4.6 presents Mohr-Coulomb envelopes for both peak and residual shear strengths. The selection of right pair of parameters (peak or residual) in a design process is under ongoing discussions. It has not been clearly defined which site specific conditions (slope geometry, lining system, waste height *etc*) trigger peak or residual displacements. Table 4.2 summarises all the results obtained from laboratory tests. This information was

further used in numerical modelling as properties defining particular interfaces. This is further discussed in Paper 3 / Appendix C, Section 3). The interface shear strength parameters include friction angle and adhesion. It was found that the weakest plane was the geomembrane / geotextile interface in dry conditions for both peak and residual displacement. It is suspected that geomembrane's co-extruded regular pattern versus geotextile fibers provide in overall less interlocking and friction on the interface, than the interface incorporating soil. Soil provides friction within the whole plane of the direct contact, additionally in terms of geotextile/sand interface, non-woven fibres allow for more sand penetration and interlocking, which potentially increase overall interface strength.
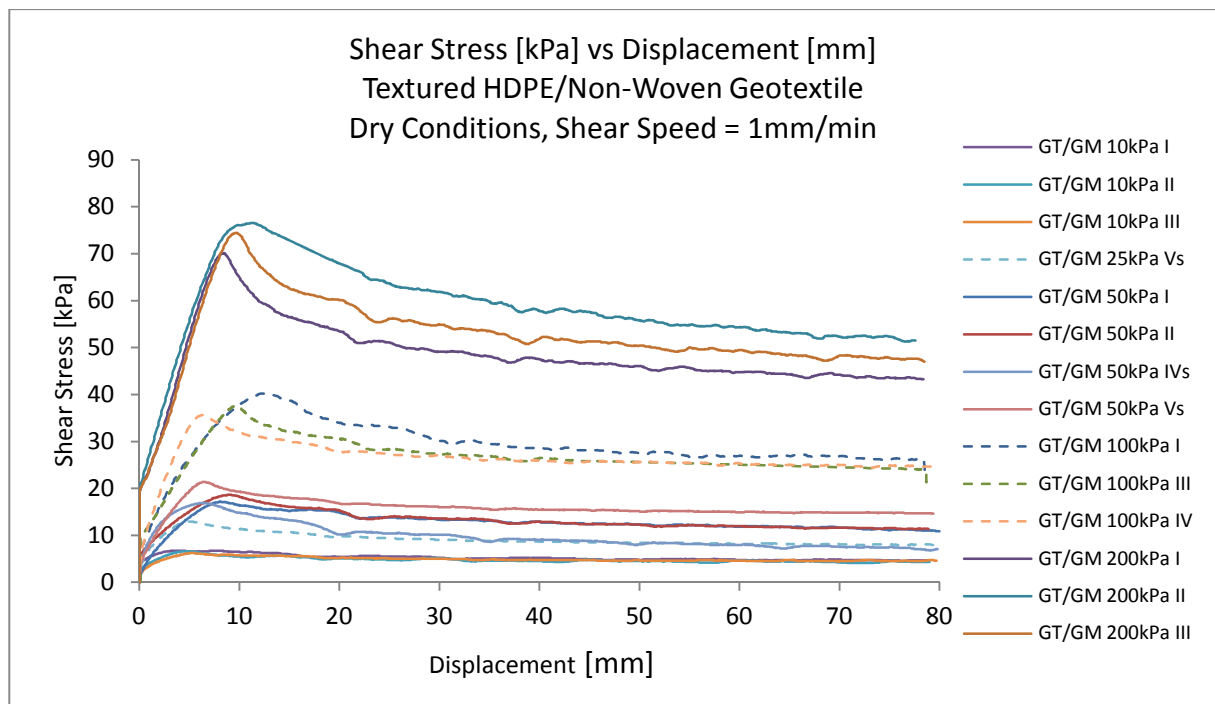
Figure 4.5 An example of shear stress vs displacement plot derived from the laboratory tests.
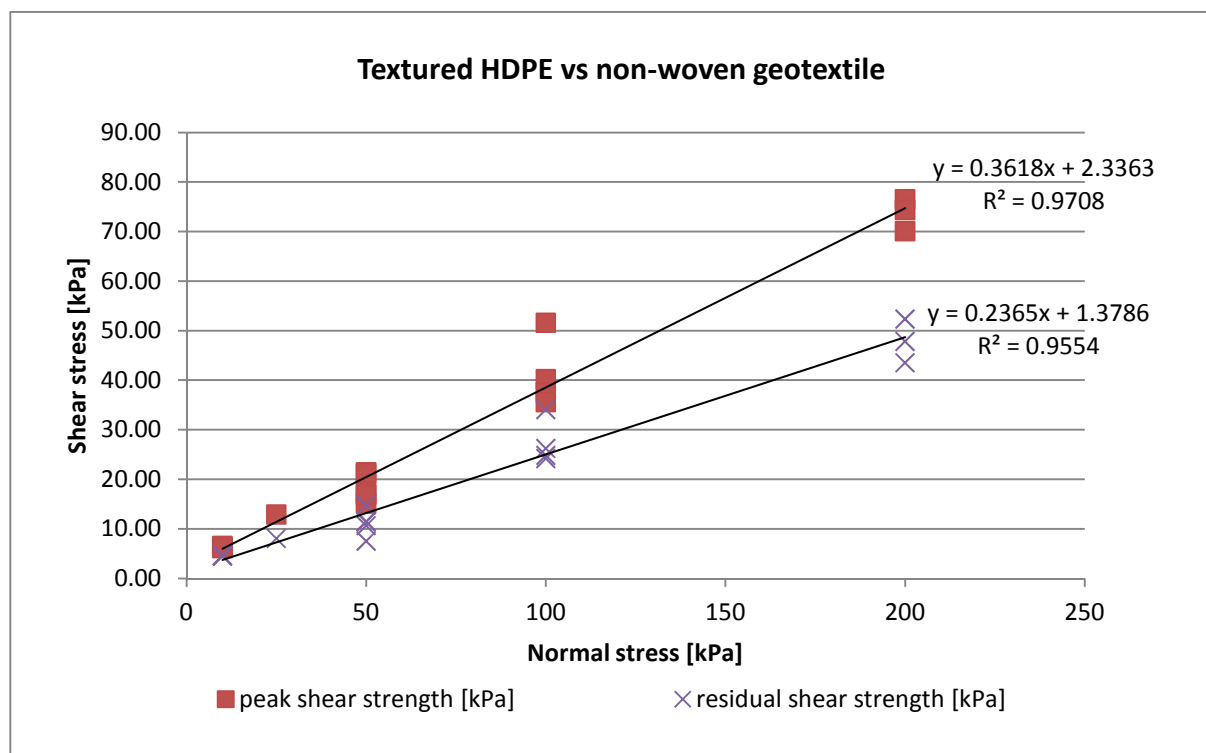
Figure 4.6 An example of Mohr-Coulomb envelope derived from the laboratory tests.

	Peak Values			Large displacement			Normal stiffness [kPa/m]	Shear stiffness [kPa/m]
	Interface friction angle δ [°]	Interface adhesion α [kPa]	R^2	Interface friction angle δ [°]	Interface adhesion α [kPa]	R^2		
GT/Sand dry	29.9	6.3	0.99	29.6	1.8	0.99	10000	4500
GT/Sand wet	29.6	3.2	0.99	29.9	1.3	0.97	10000	4500
HDPE/GT dry	19.9	2.3	0.97	13.3	1.4	0.95	10000	4500
HDPE/GT wet	20.8	4.0	0.99	14.7	2.9	0.99	10000	4500
GM/Clay undrained	31.1	7.6	0.99	25.1	3.2	0.99	10000	5500
GM/Clay drained	22.0	8.0	0.85	-	-	-	10000	5500
Waste/sand*	20	5		20	5		10000	5000

Table 4.2 Summary of the laboratory interface shear strength parameters and interface stiffness values (*after Jones & Dixon 2005).

4.2.3 NUMERICAL MODELLING FLAC

FLAC analyses were based on a landfill design procedure commercially applied, developed by Fowmes *et al.* (2007) with sensitivity analysis added. In general the Milegate geometry was modelled with a clay layer on the slope and base. Geosynthetics were modelled as beam elements with anchorage at the top. Interfaces between each component were modelled with interface shear strength constant value (*e.g.* peak or residual), or as strain softening interfaces. Staged sand placement (three lifts) and staged waste construction – twelve lifts were simulated.

Commercially applied numerical modelling design approach has been used in this study; details of the model design and the philosophy of the undertaken approach are reported in Appendix C / Paper 3, Section 3. In terms of further analyses, the most comprehensive and consistent site data were derived from the extensometer measurements of geomembrane and geotextile displacements. Therefore, the first attempt was to compare numerical modelling outputs with the site data on geosynthetic displacements.

Multilayered geosynthetic lining system

The geosynthetics lining elements were placed along the side slope. The *in situ* material comprised 1 m thick compacted clay liner over *in situ* strata with high strength and stiffness properties, therefore clay behaviour was not monitored and for the modelling approach internal behaviour of the subgrade and clay was not considered to produce any significant effect on the lining system deformations. It should be noted that initially high stiffness values were assumed for the clay as no movement was expected within the compacted clay layer. However, in further sensitivity analyses the clay stiffness was reduced to investigate the potential influence on geomembrane displacements.

Four interfaces between lining components were defined: clay / geomembrane, geomembrane / geotextile, geotextile / sand, and additionally the sand / waste interface with waste strength properties.

Information on geosynthetics tensile behaviour was provided by the material suppliers: geomembrane thickness is 2 mm and Young secant modulus E= 338 MPa (for 5% strain), geotextile thickness is 7.8 mm and Young modulus E=120 MPa (for 5% strain). Geosynthetics were not expected to fail through excessive tensile deformations (latterly proven by both field measurements and results from the analyses), therefore secant modulus values for 5% strain were used to generate conservative strains.

Soil and waste materials were represented by Mohr –Coulomb failure criterion and the properties assigned to the materials are given in Table 4.3. Waste properties are based on data available from the literature (Jones & Dixon 2005).

Material	Model	Density [Mg/m^3]	φ' [°]	c' [kPa]	Young's Modulus [MPa]	Poisson's Ratio [-]
Waste	Mohr - Coulomb	1.0	25	5	0.5	0.3
Sand layer	Mohr - Coulomb	1.7	35	0	70/20*	0.4
Clay layer	Mohr - Coulomb	1.7	23	5	150/50*	0.3

Table 4.3 Materials properties applied in FLAC models (note: *additional analysis, considering decreased values, in order to trigger the geosynthetic movements in further analysis).

Interfaces

The importance of the interface strength parameters has been emphasised before by various authors (*i.e.* Filz *et al.* 2001, Jones & Dixon 2005) and various approaches have been applied in limit equilibrium techniques (peak, residual or both) to evaluate displacements and strains between geosynthetic materials. In general it is accepted that landfill side slope lining systems might undergo interface shear strength softening behaviour, however, often peak or residual values are considered for stability and integrity analysis. The peak values by some are considered to overestimate the strength of the interface whereas the residual values are considered to be too conservative.

The Milegate case modelled in FLAC allows the interface behaviour between elements to be modelled with certain properties (*e.g.* strain softening). Hence material interface shear strengths that were measured in the direct shear box during laboratory tests could be directly transferred into the model (Table 4.2). Waste / soil interface properties were not investigated in the laboratory and the values were based on the common approach of assigning the same waste material properties to interface strength properties. Additionally, four interface shear strength strain input approaches were examined with: peak, residual, strain softening procedure and degraded values (*i.e.* representing materials ageing).

Simulations

The model of the monitored slope evolved, starting from a relatively simplistic approach to more complicated solutions. The model was built taking into consideration staged construction: each sand veneer stage (0.5 m thick sand veneer was placed in three lifts, in 10 m long layers measured parallel to the slope) each followed by four waste lifts to cover the sand veneer. In total, the model computed sixteen stages of material placement (1st clay, 2nd sand veneer, 3-6th waste lifts, 7th sand veneer, 8-11th waste lifts, 12th sand veneer, 13-16th waste lifts). Figure 4.7 presents intermediate and final stage of the modelling process. Each waste lift was approximately 1 m high and 0.5 m thick (two grid elements high and two elements wide). A number of simulations were run in order to examine suitability of various design approaches *i.e.* application of specific lining materials interface shear strengths. These included properties investigated in the laboratory tests: peak, residual and strain softening approach, as well as behaviour of degraded interfaces. In the latter reduced interface values were used in order to investigate *in situ* degradation, ageing and material weathering. The justification for the degraded properties is further discussed in the Appendix C / Paper 3, Section 5.

Initial analysis revealed significant influence of the geosynthetic prolonged exposure to atmospheric conditions on downward slope displacements. Therefore, in order to reproduce *in situ* records further investigations were carried out. These included analysis of sand, geotextile and clay properties in an attempt to increase geotextile displacements (to those measured). The sensitivity analyses regarded materials stiffness, as this has been observed to trigger geosynthetics displacements significantly. This aspect is described in detail in Appendix C / Paper 3, additionally Table 4 of the paper presents significant influence of clay stiffness on geomembrane displacements. Decreasing stiffness of the clay enhances down slope movement of the modelled geomembrane.
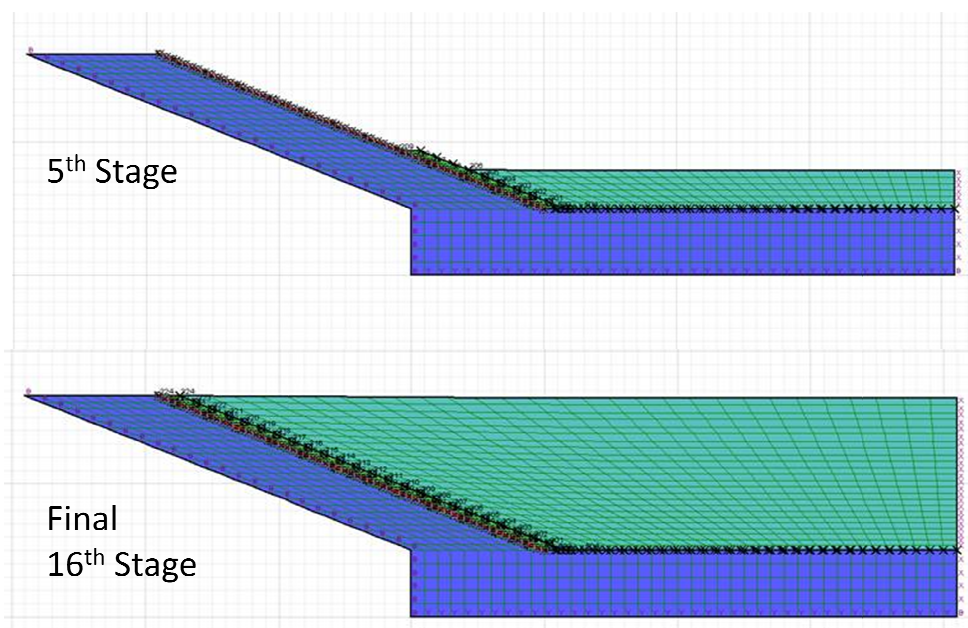
Figure 4.7 An example of modelling process stages.

4.2.4 **RESULTS**

This section provides a brief description of the project results, as these are reported and widely analysed in detail in Zamara *et al.* (2013), which is included in the Appendix C / Paper 3, Section 4.

Appendix C / Paper 3, Figure 5 presents an overview of all the displacements recorded by extensometers through the three year monitoring period. These present significant movements of the geotextile, within the upper section on the slope. These large displacements were triggered mostly at the time of second sand veneer placement. While geomembrane measurements gradually increase for locations further down the slope with increasing distance from the slope top, only limited displacements occurred directly in response to veneer placement.

The numerical modelling results using the standard analysis approach produced limited agreement with the measured behaviour, especially in terms of geotextile displacements. This is presented in Appendix C / Paper 3, Figure 5, plot FLAC_MIN. In general, for the standard approaches it was impossible to replicate geotextile movement in the middle/top section of the slope, where the largest movement in the model occurs within the toe section. Therefore, further investigations were undertaken to identify the mechanism of such behaviour. The study of photographs taken at various stages of the project revealed large wrinkle formation on the geotextile material (*e.g.* Appendix C / Paper 3, Figure 11). This resulted in carrying out simulations for interfaces with reduced properties, based on an assumption that wrinkle formation reduces the area of direct contact between the geotextile and geomembrane). It should be emphasised that the model does not include HDPE thermal behaviour.

However, geomembrane overall movement is represented in FLAC with relatively good agreement. Overall, similar ranges of displacement were computed in FLAC for stages of construction. Thermal analysis of HDPE geomembrane and geotextile displacements allowed

definition of a mechanism leading to increased geotextile displacements down the slope. It is thought that cyclic deformation of the exposed geomembrane, due to temperature changes, causes geotextile stretching. Elastic geomembrane recovers its 'original' length when temperature decrease, whereas geotextile deformations are not recovered leading to cumulative thermal related permanent wrinkles. The principles of the mechanism are presented in a schematic manner in Appendix C / Paper 3, Figure 12. In a simplified evaluation of the geosynthetic thermal *in situ* behaviour, HDPE thermal expansion was calculated for the geomembrane in conjunction with ranges of temperature seasonal changes recorded directly on the site and in the region. The results of this simplified analysis are presented in Appendix C / Paper 3 Table 5, where they are compared with the down slope deformations of the geotextile measured during placement of the 2nd sand veneer. It can be observed that magnitude of the elongation is highly comparable. It is concluded that existence of wrinkles in the exposed section of geotextile allowed rapid down slope displacements to occur during loading from the sand. This resulted in relative shear displacements between the geomembrane and geotextile and has implications for mobilisation of interface shear strength and hence for integrity of the side slope lining system. Further discussion and explanation are reported in Appendix C / Paper 3, Section 5.

4.3 LANDFILL CAPPING SYSTEM STUDIES

4.3.1 GEOMETRICAL / PARAMETRICAL ANALYSIS IN VADOSE/W

The capping soil cover cannot be well represented through the use of steady state analysis. A typical thickness of the top soil covers on landfills varies between 0.5 m - 1.0 m. Seasonal changes in temperatures and precipitation have crucial roles on the water regime within the cover soil. These might change rapidly and hence capping soils do not reach steady state conditions. Vadose/W allows comprehensive analysis which includes variables related to

atmospheric conditions and therefore seasonal changes within the cover soil properties are acknowledged. Below are described key boundary conditions of the capping model.

Atmospheric conditions

Atmospheric conditions based on average UK rainfall for three regions were considered 600 mm (Midlands, South and East Anglia), 1024 mm (average UK) and 2000 mm (Wales and Scotland, MetOffice 2012). This was due to difficulties related to acquiring comprehensive data. Each simulation period was run three times using the same input data – to see if the initial conditions had any effect on the capping performance. Additional data required for the analysis were: temperature max and min on a day, humidity max/min, wind speed and time of rainfall occurrence throughout the day. Precipitation of 1024 mm per annum was selected for further analysis as a representative average for UK conditions. Sensitivity analyses were also carried out on lower and higher precipitation amounts *i.e.* 600 mm and 2000 mm per year. These developed subsequently lower or significantly higher saturation of the cover soil.

The atmospheric input data were applied in Vadose/W on a daily basis. The required information for the software, are as follows:

(1)   Daily air temperature (min and max);

(2)   Humidity (min and max);

(3)   Wind speed;

(4)   Precipitation (mm) - only one precipitation event can be specified per day; with

(5)   Indication of the event occurrence (0:00 h-24:00 h).

Soil model

Defining the cover soil properties for the numerical analysis was a challenging task. Spatial variability in permeability of the site soil and macro scale permeability of fractures and preferential flow paths are difficult to represent in the model. Soil models implemented in the analysis Vadose/W, require parameters that are often not available for cover soil (*i.e.* the

upper layer that does not constitute a barrier). Typical soil parameters used in the software are as follows:

(1)   Water retention curves

It is common to represent hydraulic properties of soil with van Genuchten (1980) or Fredlund & Xing (1994) method. The empirically derived parameters predict a soil water retention curve (function of volumetric water content versus negative pressures).

(2)   Hydraulic conductivity functions

It should be emphasised that to predict accurately water infiltration within the cap it is crucial to consider soil hydraulic permeability as a function of negative pressures, as most of the time the landfill cover soil is subjected to unsaturated flow with reduced permeability and increased suction pressures. Vadose/W allows calculation of a hydraulic conductivity function based on water retention curves, soil saturated permeability and coefficient of volume compressibility (the slope of the volumetric water content function).

(3)   Soil thermal properties

Soil *thermal conductivity* (this describes ability of soil to transport heat) and *volumetric heat capacity* (this describes quantity of heat needed to increase the temperature of the material by a degree) are required for the analysis. For the full thermal analysis it is required to implement thermal properties as a function of temperature. However, this approach was beyond the scope of the study therefore, it has been decided to use the simplified approach, which requires constant values describing thermal properties.

(4)   Gas diffusion parameters

Vadose/W has a built-in function referring to chemical transport in response to concentration gradients in the air phase. These functions were disabled in the capping analysis, and considered insignificant for this study.

Soil type	Description		k, m/s	Drainage
Cobbles and boulders	Flow may be turbulent, Darcy's law may not be valid		I	Very good
Gravels	Coarse Clean	Uniformly graded coarse aggregate	10^{-1} 10^{-2} 10^{-3}	
Gravel sand mixtures	Clean	Well graded without fines	10^{-4}	Good
Sands	Clean, very fine Silty Stratified clay/silts	Fissured, desiccated, weathered clays Compacted clays – dry of optimum	10^{-5} 10^{-6} 10^{-7}	
Silts	Homogeneous below zone of weathering		10^{-8} 10^{-9} 10^{-10}	Poor
Clays		Compacted clays – wet of optimum	10^{-11} 10^{-12}	Practically impermeable
Artificial	Bituminous, cements stabilized soil Geosynthetic clay liner / Bentonite enriched soil concrete			

Table 4.4 Classification of coefficients of soil permeability (Look 2007).

Three different soil permeabilities were analysed: 0.864 m/day, 0.0864 m/day and 0.000864 m/day (*i.e.* 10^{-5} m/s, 10^{-6} m/s and 10^{-8} m/s). Higher permeability soil drains water entering the system immediately and numerically this causes convergence problems related to calculation of unrealistically high negative pressures throughout dry periods. These selected permeabilities are representative of fine sands, silty sands with higher and medium permeability and clayey silt with lower permeability. Soil installed on landfill caps commonly comprises of cheap, low quality material with medium to low permeability, hence the selected permeability values for the analyses were thought to comprise a good approximation of *in situ* conditions. Additionally, Vadose/W requires comprehensive fully defined functions relating permeability with negative pressure and volumetric water content with negative pressure (this were derived by Vadose/W through van Genuchten approach – closed form expressions commonly applied and frequently validated for various soils *i.e.* Ghanbarian-Alavijeh *et. al.* (2010). The soil permeability curves are presented in the Figure 4.8.
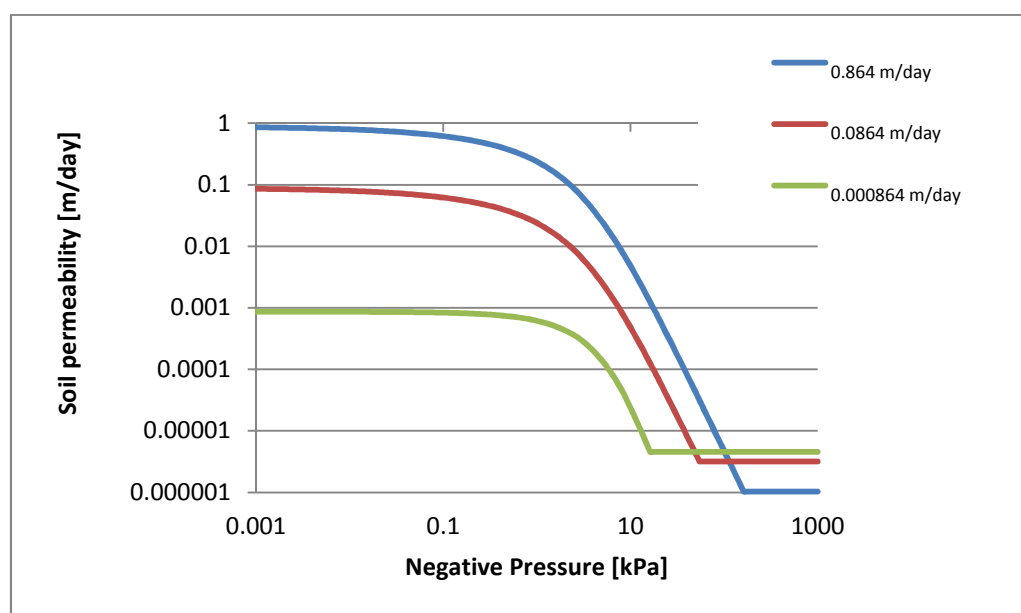
Figure 4.8 Permeability versus negative pressure curves adopted for the analysis.

Geometry and mesh

The analysed geometry included the top soil with / without an underlying drainage layer. Only the capping soil cover top layer was considered in this study. No further interaction between underlying materials was considered.

The applied conditions for the analysis were as follows. A 50m long slope was modelled (Figure 4.9), and five different slope angles were considered (inclinations of: 1v:2.5h, 1v:3h, 1v:4h, 1v:5h and 1v:8h). These are common inclinations for landfill capping designs (except for 1v:2.5h, which is relatively steep for applications without reinforcement). Meshes were created from rectangular elements (30cm), apart from surface layers with 3 elements 4 cm x 10 cm and layers using 10 cm x 0.3 cm representing geosynthetic drains.

Hydraulic boundary conditions

Two groups of analyses were conducted:

(1)   Simulations of the capping without a drainage layer:

For the conditions without a drainage layer, only a soil layer was modelled. A toe drain (pressure head equal zero) collecting water from the geosynthetic materials, was assigned to a

node corresponding to the location of the water collection pipe on the site, and modelled as a pressure head node equal zero. An additional gradient boundary condition was applied to the bottom line of the slope in order to remove excess water from the toe section and to stop water table build up.

(2)   Simulations of the capping with drainage layer:

For the conditions with drainage, a thin layer (3 mm thick - corresponding to commonly available geocomposite drainage material) with various permeability coefficients was modelled directly beneath the soil layer. These were analysed in order to present performance of a relatively low transmissivity layer and to investigate minimum drainage material requirements to reduce pore water pressure within a cap / cover soil. Specific parameters characterising the drainage material were derived from the literature (*e.g.* Iryo and Rowe 2005). The same toe boundary conditions were applied as for the analyses without a drainage layer.
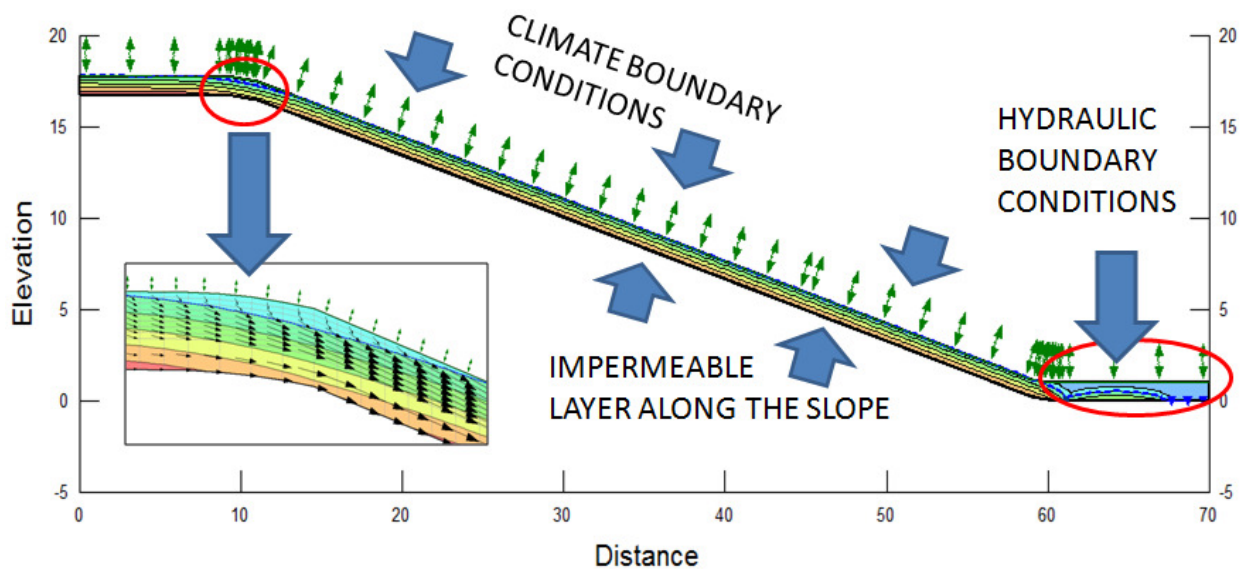
Figure 4.9 Vadose/W capping boundary conditions.

Vegetation, runoff and ground freezing

Vadose/W options allow application of vegetation, ground freezing, runoff ponding and gas transfer. Applying **vegetation** and root length functions in Vadose/W are optional. This study does not take into account the effect of vegetation growth within the capping soil. However it is acknowledged that plants can change water regimes in the cover and this increases factors of safety against stability. It is considered that a study neglecting vegetative growth is representative, since in the UK most rainfall occurs in the autumn / winter season, when vegetation influence is minimised.

Additionally, plant growth on the landfill cap is a delayed process as vegetation does not appear immediately on the constructed cap and once it appears stability issues related to increased water storage are improved. If vegetation growth is considered in Vadose/W, additional analysis would have to be carried out considering relationships between plants/grass size and soil classification (from an agricultural point of view) related to permeability and soil water content, along with vegetation size related to weather conditions. However this is beyond the scope of this study.

Runoff can be disabled in the Vadose/W analysis. This results in loss of water infiltration in further time steps of the analysis. When runoff is enabled, the runoff water is considered to infiltrate in the next time step in the lowest point of the defined geometry. This approach in the software is highly simplistic, but is considered to give a good estimation of the run-off volume (Bohnhoff *et al.* 2009). In general surface runoff is computed in a relatively simplistic manner (Equation 4.1).

Runoff = Precipitation – Infiltration – Evapotranspiration – Change in storage   (Eq. 4.1)

In terms of **ground freezing**, due to software limitations it was necessary to exclude ground freezing analysis. All the temperatures below zero Celsius were considered as zero (personal communication with GeoStudio Ltd technical support). Overall, for UK average

weather conditions this approach is an acceptable simplification since monthly averages do not fall below zero degree (Table 4.5).

England Weather Averages													
Month	January	February	March	April	May	June	July	August	September	October	November	December	Year
Average maximum temperature °C (°F)	6.6 (43.9)	6.9 (44.4)	9.3 (48.7)	11.7 (53.1)	15.4 (59.7)	18.1 (64.6)	20.6 (69.1)	20.5 (68.9)	17.5 (63.5)	13.6 (56.5)	9.5 (49.1)	7.4 (45.3)	13.1 (55.6)
Average minimum temperature °C (°F)	1.1 (34.0)	1.0 (33.8)	2.4 (36.3)	3.6 (38.5)	6.3 (43.3)	9.1 (48.4)	11.4 (52.5)	11.2 (52.2)	9.3 (48.7)	6.6 (43.1)	3.5 (38.3)	2.0 (35.6)	5.6 (42.1)
Sunshine hours	50.5	67.7	102.5	145.2	189.9	179.4	192.8	184.1	135.0	101.3	65.2	43.9	1457.4
Rainfall mm (inches)	84.2 (3.3)	60.1 (2.4)	66.5 (2.6)	56.8 (2.2)	55.9 (2.2)	62.9 (2.5)	54.1 (2.1)	66.7 (2.6)	73.3 (2.9)	83.6 (3.3)	83.5 (3.3)	90.4 (3.6)	838.0 (33.0)
Rainfall ≥ 1 mm days	13.4	10.4	12.1	10.1	9.8	9.8	8.5	9.4	10.2	11.8	12.5	13.1	131.2
Source: Met Office[1] (1971–2000 averages)													

Table 4.5 Average weather conditions for England (MetOffice 2012).

A number of simulations have been carried out in Vadose/W in order to investigate long term capping soil saturation scenarios and critical pore water pressure distributions within the capping soil along the low permeability layer that influence stability. Various conditions were considered in the initial study and as a consequence, critical governing parameters were selected and their influence analysed.

The average PSR ratio was calculated for each of the simulations and presented in a summary graph for different slope angle (Figure 4.11). The main attention was given to soils with permeability 10^{-5} and 10^{-6} m/s due to its specific behaviour on the slope. Higher permeability soils (10^{-4} m/s) were acting as a drain by themselves and lower permeability soils had high run off and low penetration of water (10^{-8} m/s). The selected soil permeability coefficients were chosen to present the essential aspects of the study – fully or partially saturated soil cover layer, with fully or partially developed PSR. To minimise the influence of the initial boundary conditions (starting steady state analyses) on the capping system overall hydraulic performance, annual weather conditions were applied at least three times to a single

simulation. This was to generate an average response of the system excluding potential adverse effect of the initial steady state setting. It should be highlighted that soil layers considered in this study did not incorporate surface cracks. It can be observed that Vadose/W to some extent presents intuitive solutions:

(1) Permeability of the soils for capping purpose might be divided in the following categories:

Unsaturated

- Highly permeable – soils with high permeability coefficients allowing rapid drainage, such that pore water pressures do not build up on the cover soil / cap barrier interface;

- Low permeability - soils with very low permeability coefficients (10^{-8} m/s), experience very limited water penetration with high surface run off, causing no changes in overall layer saturation and hence again no build-up of positive pore water pressures.

Semi saturated

- Pore water pressures develop in the cover soil, which vary as a function of the cap inclination (10^{-5} m/s). PSR are not observed for steeper slopes but develop for shallow cap covers;

Fully saturated

- Soils with permeability coefficient allowing full saturation (10^{-6} m/s), develop PSR=1, for all the analysed inclinations.

(2) The highest saturation of the cover soil and water storage occurs in winter / early spring seasons for the modelled annual weather conditions.

(3) Lower slope inclinations (1v:8h = 7.2°) for soils with permeability of 10^{-5} m/s and hence semi-saturated state, develop higher pore water pressures along the slope, while higher slope angles (1v:2.5h = 21.8°) have enhanced gravitational seepage preventing water accumulation and pore water pressure build-up. Additional inclination versus surface runoff

analysis could not be conducted in Vadose/W due to the simplistic runoff approach implemented in this software.

(4) Inclusion a drainage element at the base of the cover soil layer, even with only minimal transmissivity properties, triggers reduction of the pore water pressures within the fully saturated soil layer (Figure 4.10). This converts a fully saturated soil layer into a semi saturated soil. Increased drainage capacity reduces pore water pressures within the soil layer. These simulations present results for long term conditions but no individual studies on site specific conditions / weather events were undertaken. These generic analyses indicate possible scenarios of behaviour, however further studies revealed the influence of a storm event on a capping system condition. Therefore it should be investigated further if modelled semi steady state develops within capping cover.

Furthermore, it is acknowledged that such a novel implementation of numerical modelling solutions should be strongly supported with detailed site studies in order to verify assumptions, justify the approach, investigate simplifications and better understand site specific factors controlling behaviour.
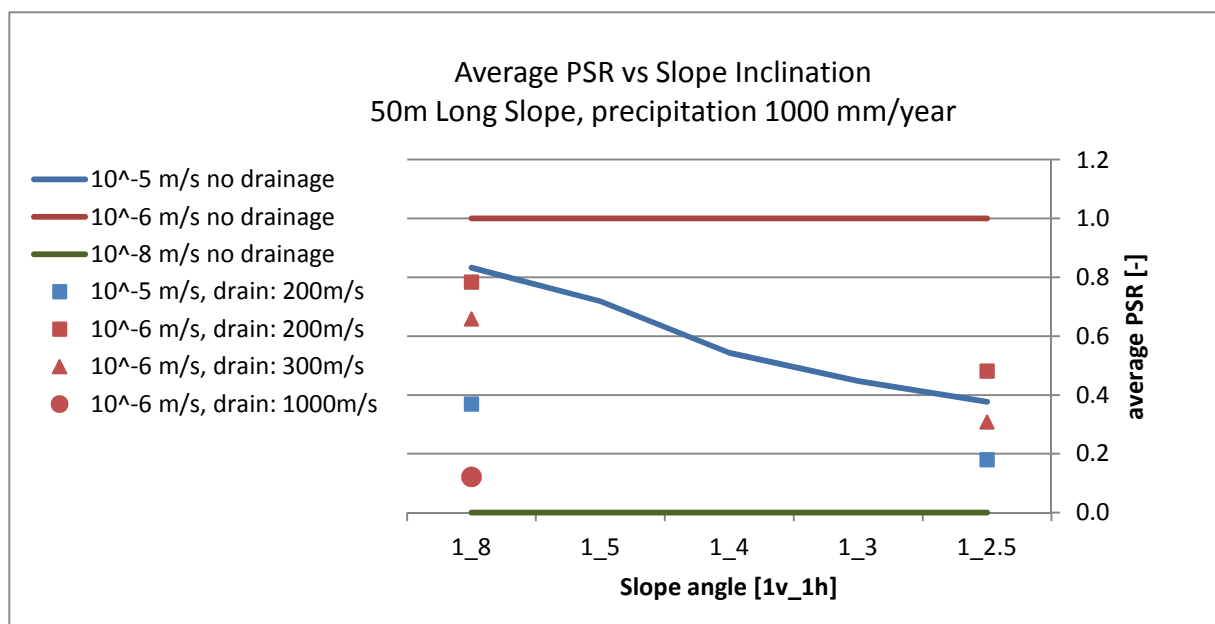
Figure 4.10 Average PSR calculated for various slope inclinations, permeability and drainage layer parameters.

4.3.2 FULL SCALE EXPERIMENT – MONITORING PROGRAM BLETCHLEY LANDFILL PROJECT

The Bletchley Landfill capping was constructed in autumn 2011. Drainage materials were installed on a low permeability clay layer 1m thick (Figure 4.11). Three different geosynthetics were installed in Bletchley landfill underneath the top soil layer: a cuspate core geocomposite (FCf6), a geocomposite with band drains (GPT5) along the sheet and a non-woven needle punched geotextile (HPS5). The length of geosynthetic drains on the monitored slope section was 40 m (FCf6) and 45 m (GPT5, HPS5), and the width of each panel 6 m. The geosynthetics were installed within the lower sections of a constructed cap. The average inclination of the slope was 7.2° (1v:8h). The thickness of the top soil was on average ~30 cm over the cuspated core geocomposite and ~40 cm on the other sections. Monitoring instrumentation was installed on each drainage panel and also in an area of slope without any drainage material. Figure 4.12 shows a schematic cross section of the capping system.

Figure 4.11 Initial stage of cuspate core drainage material installation and cover soil placement.
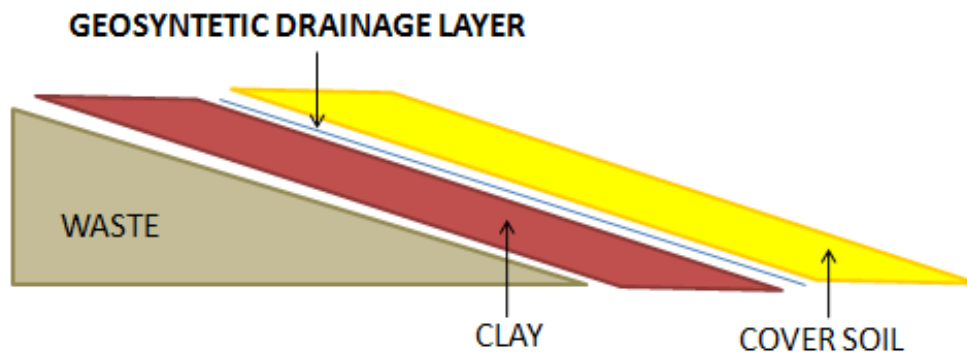
Figure 4.12 Schematic cross section through Bletchley capping.

The site instrumentation consisted of: a weather station, gauge monitoring surface runoff, standpipe piezometers, volumetric water content sensors and a system monitoring water outflow from the drainage layers. Additionally, for comparison the area without a drainage layer was instrumented. Standpipe piezometers were installed in the middle and lower section of the slope, with the bottom of the tube located on top of the geocomposite drain. Initial readings demonstrated acceptable performance of the monitoring system. Moisture content changes were observed in response to precipitation events and high drainage outflows were recorded in response to increased rainfalls in April and June 2012. The details of instrumentation installation, location and initial readings can be found in Appendix D / Paper 4, Section 5.

Volumetric Water Content sensors

Volumetric water content sensors were installed to monitor moisture change within the cover soil in response to precipitation events and drying periods. At each monitored section (nonwoven geotextile / band drain geocomposite / cuspate core geocomposite / soil), volumetric water content sensors were placed (20 m and 38 m down slope from the top edge of the geocomposite panels) at various depths that were determined by the thickness of the soil at the particular location (Figure 4.13). The sensors were installed in the cover soil above the geosynthetics / clay and parallel to the slope. The records were logged every half hour. To
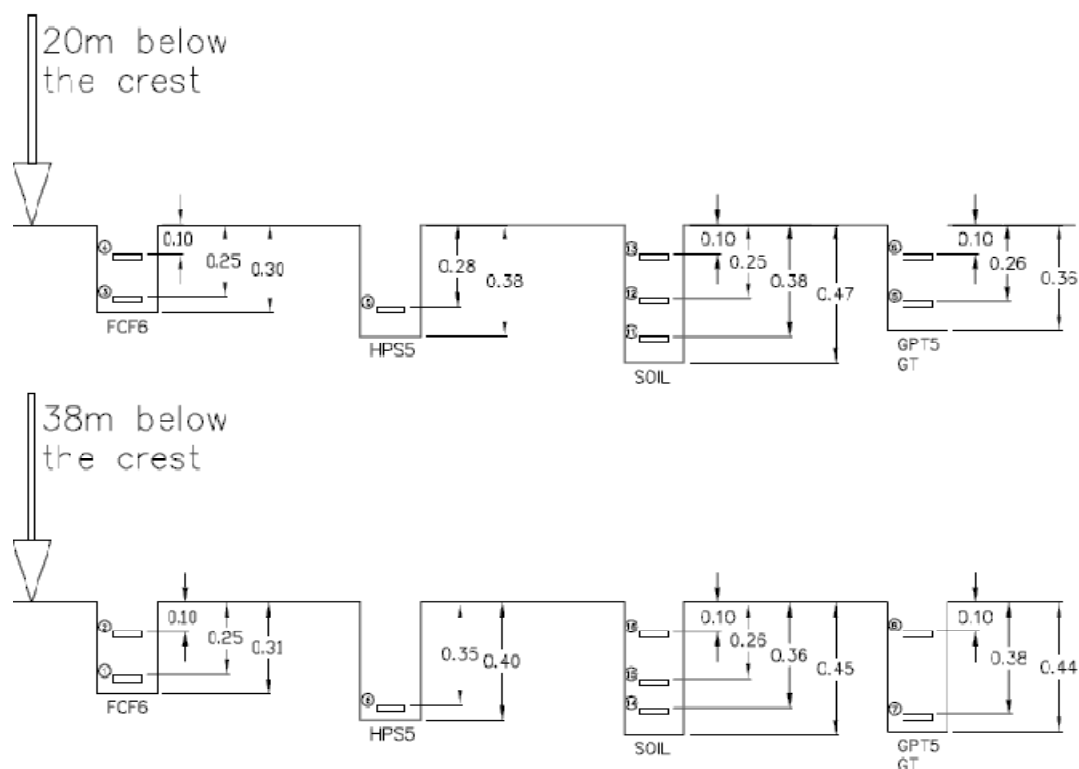
Figure 4.13 Schematic location of volumetric water content sensors within the soil layer.

allow comparison between materials performance, each panel was instrumented using the same configuration. 16 volumetric water content sensors from Campbell Scientific Int. (Figure 4.14) were installed within the soil layer. It is acknowledged that volumetric water content sensors might not present the actual values of the soil volumetric water content, (due to the sensors calibration issues), however this study focused on analysing the general response of the system to weather events and investigated trends, tendencies, relative changes and time scales related to the events.

Figure 4.14 Volumetric water content sensor installation.

Piezometric level

To measure pore water pressures, standpipe piezometers were installed directly above the drainage layer. Additionally, for the band drain material, pairs of standpipes were installed at two locations – first above a band drain elements and second in the middle of the geotextile section equidistant between two band drains. Piezometers were installed at the lower and middle part of the slope (Figure 4.15), at corresponding locations to the volumetric water content sensors. Standpipes were installed in boreholes and backfilled with gravel with a bentonite seal used in the top part. The accuracy of these instruments is uncertain, nevertheless, they delivered an indication of the pore water pressure magnitude at their locations. Figure 4.16 presents soil layer thicknesses at the standpipe piezometers locations.
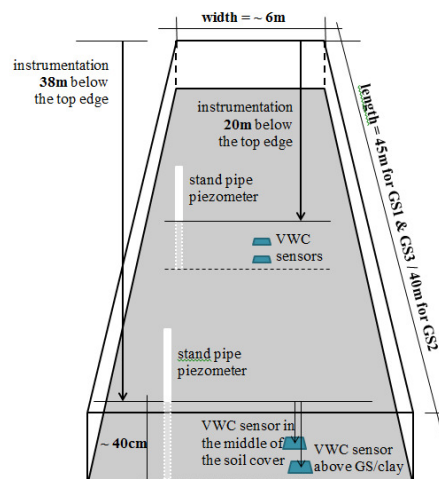
Figure 4.15 Schematic location of the measuring instrumentation (after Zamara *et al*. 2012).

Drainage outflow

A system of pipes (100 mm diameter guttering pipes) was installed along the lower edge of each drainage material panel to collect all out-flow from the drainage layer. The water flowed into a tank installed 5 m below the lower edge of each drainage material panel. The manual measurements of flow were taken during each site visit. The system allowing measurement of water outflow from the drainage materials is presented in Appendix D / Paper 4, Figure 4). Water leaving the tanks was transported by a system of pipes to the toe of the capped slope to prevent any stability concerns.

Capping soil property

The top soil overlaying the geosynthetic drains used in this project initially was considered as a low permeability material *i.e.* silty sand. However to adequately assess soil permeability it was decided to perform a falling head test (after Head 1982). This is a common laboratory test method used to determine the permeability of fine grained soils with intermediate and low permeability coefficients, such as silts and clays.
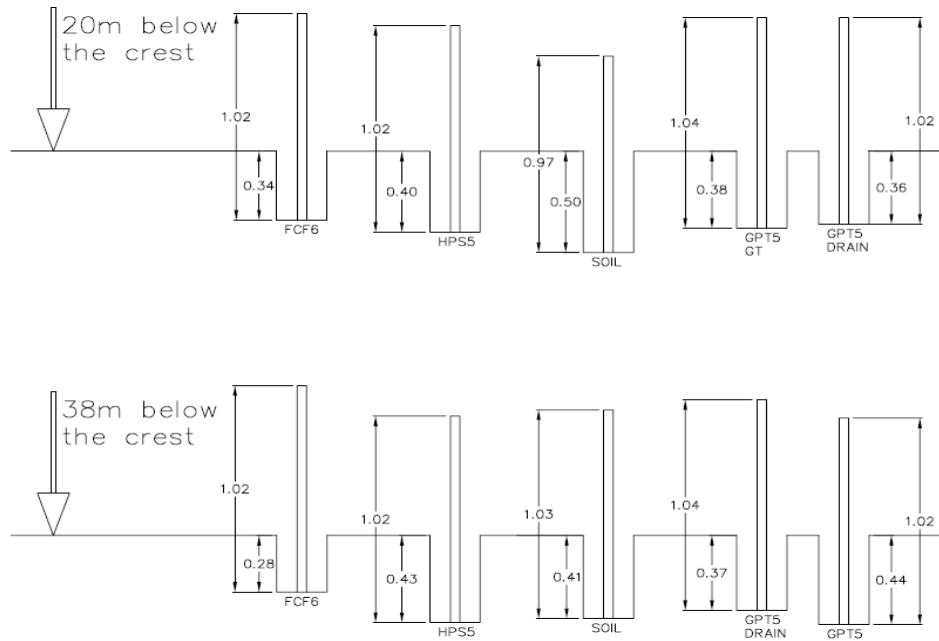
Figure 4.16 location of the measuring instrumentation within a soil layer.

It was found that micro scale tests do not reflect correctly soil properties *in situ* such as macro porosity and fracturing scale conditions. Samples prepared in the laboratory were manually compacted and it was observed that *in situ* compaction was difficult to replicate and very low soil permeability was determined through laboratory tests (*i.e.* after hours of being submerged the water did not penetrated the sample (this indicates permeability of 10^{-8} m/s or lower). Even though very low soil permeability was assessed in the laboratory, water was recorded to penetrate the cover on the site relatively quickly. It was observed that the drains respond directly to rainfall events within a relatively short period of time (hours). Therefore it was decided to carry out simple *in situ* measurements to better assess soil permeability. An infiltrometer was used to assess the soil *in situ* permeability (Figure 4.17). The test indicated the *in situ* soil permeability is on the order of 10^{-5} m/s. However, additional calculations of flows observed on the site due to a rainfall events, indicate that macro permeability of the soil layer (*i.e.* including shrinkage, cracks and fractures) is in order of 10^{-4} m/s.

Figure 4.17 Infiltormeter (Eijkelkamp, 2013).

4.3.3 GENERAL SITE OBSERVATIONS

After a year of monitoring it has been observed that:

(1)   Soil / drainage responds very quickly to occurring precipitation. It is considered that flow driven through cracks and fissures controls hydraulic permeability of the cover soil layer. This resulted in an unexpected high permeability of the soil cover;

(2)   By the end of the 2012 summer season (after 10 months of observations), the 0.4 - 0.5 m thick soil layer had a well-developed vegetation that resulted in low moisture penetration across the layer and insignificant increase in volumetric water content within the locations even following prolonged rainfall and intense events. Hence, no flow was observed for materials covered by ~0.4 m thick well vegetated soil cover layer, even for 24 hour precipitation of 16.3 mm;

(3)   It was challenging to collect comprehensive data on drainage outflow from the drainage layers, due to uncertainty related to weather forecasts. Additionally, in the initial stage of the project the UK experienced a prolonged drought season, and therefore data collection was ineffective.

The general conclusion drawn from the **volumetric water content sensors** are as follow:

(1)   All of the sensors installed below the surface of the cover soil (approximately 10 cm below) present similar trends and as expected, the highest spatial variability of moisture content ranges due to surface wetting / drying periods. Sharp peaks occur directly due to rainfall event and surface water penetration trough the cover.

(2)   Based on readings undertaken on 11th June 2012 (Figure 4.18) sensors installed above the cuspated core geocomposite returns to its initial state *i.e.* before the start of the event, in the shortest time and produces the lowest increase in soil volumetric water content for a given precipitation event. This indicates that water flows through the soil layer directly to the drain.

The general conclusion drawn from the **standpipe piezometers** are as follows:

(1)   All of the drainage materials develop piezometric water levels for a given rainfall events;

(2)   The most significant increase in piezometric water level above the drainage layers was monitored on the day with over 20 mm precipitation (11th of June 2012). The relatively high precipitation triggered development of a water table above the drainage materials.

Water table development within a capping cover comprises an undesirable effect. This normally is caused by drainage material properties related to break through head occurrence (or filter clogging). This for capping systems should be equal to zero, as any additional water pressures along the cap decrease its stability.
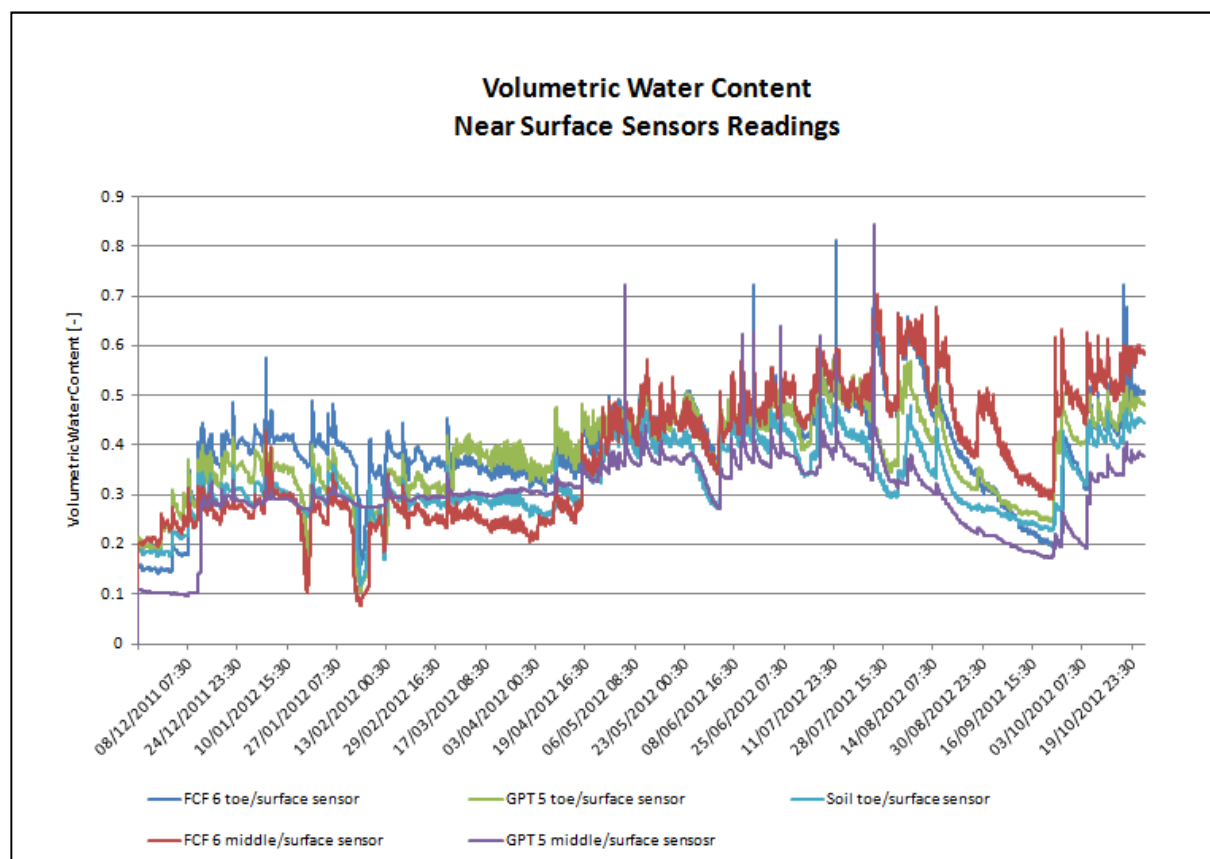
Figure 4.18 Volumetric water content sensors – readings from the near surface sensors.

4.3.4 MONITORING PROGRAMME RESULTS VERSUS VADOSE/W MODELLING

For the site specific analysis – Bletchley, simulations were carried out for the selected weather events in which water drained from the geocomposite was recorded. Complex information on the weather conditions was initially collected by the weather station installed on the site. Later when the station rain gauge failed, data from a local private weather station installed in Bletchley was obtained (WunderGround, 2012a & b).

The slope inclination in Bletchley is 1v:8h and length of 40 and 45m with clay layer above and silty soil below the monitored section (Figure 4.19). The average thickness of the soil layer above the FCf6 drainage layer was approximately 30 cm. The soil thickness above the HPS5, GPT5 and clay sections was approximately 40 cm. Vadose/W model was created

from a rectangular grid, with discretisation of surface and drainage layers. Drainage material was built with element size corresponding to the actual drainage material thickness.
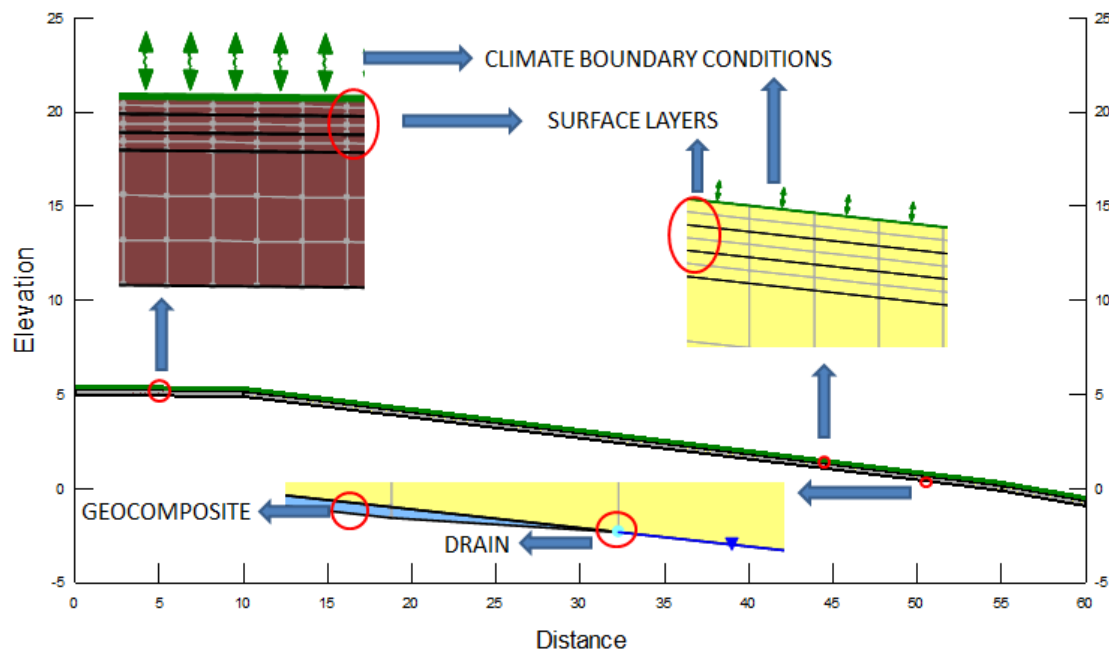
Figure 4.19 Vadose/W model of Bletchley study.

Analysis was conducted in Vadose/W to verify and calibrate the Vadose/W model against *in situ* measured data and to help establish properties for the numerical model of geosynthetic drainage materials used in capping applications. In this study three rainfall events were selected for detailed analysis ('A', 'B' and 'C', see Table 4.6 for details), these were days with high precipitation and with significant outflow recorded from the drainage materials.

A number of simulations were run in order to replicate *in situ* conditions during these events and to allow verification of recorded data. The calibration was conducted for event 'A', for which the most comprehensive data on water flow were collected. Further analysis and adjustments were carried out on the remaining events, in order to validate the approach.

Precipitation data

The site weather station on these days revealed technical issues, and therefore conditions from weather stations in the area were used to deliver information on atmospheric conditions (WG1 – located in Bletchley and WG2 – located in Milton Keynes, Woughton Park, WunderGround 2012a & b). Both of the stations are less than 5 miles distant from the site. However to better represent the event adjustments to events time of occurrence had to be implemented. Table 4.7 presents adjusted values that better replicate the observed behaviour.

Event	Temperature [°C]		Humidity [%]		Wind speed [m/s]	Precipitation Amount [mm]	Precipitation occurrence time	
	max	min	max	min			start	end
A.26/09/2012	14.2	9.6	95.0	79.0	0.7	4.0	14:00h	16:00h
B.24/09/2012	14.2	9.4	97.0	80.0	1.0	9.9	Several events throughout the day	
C.11/06/2012 12/06/2012	11.9	9.7	95.0	90.0	1.7	28.0	00:00h 03:30h	24:00h 07:30h

Table 4.6 Recorded atmospheric conditions.

Event	Temperature [°C]		Humidity [%]		Wind speed [m/s]	Precipitation Amount [mm]	Precipitation occurrence time	
	max	min	max	min			start	End
A.26/09/2012	14.2	9.6	95.0	79.0	0.7	4.0	14:00h	15:00h
B.24/09/2012	14.2	9.4	97.0	80.0	1.0	5.8	11:00h	13:00h
C.11/06/2012 12/06/2012	11.9	9.7	95.0	90.0	1.7	28.0	02:00h	14:30h

Table 4.7 Adjusted atmospheric conditions implemented in the model.

Drainage properties

Drainage properties of each geosynthetic layer used in the analysis were based on the approach described in Section 3.5.2. The size of a roll of drainage material installed on the site and hence panel dimensions were 40 m x 5.75 m (FCf6) and 45 m x 6 m (GPT5, HPS5). The adjacent sections of the slope were covered by the original clay / soil layer. Geosynthetic

drainage materials where modelled in the same manner as the soil layer, with thickness and permeability corresponding with the reported parameters. Hydraulic properties of the geosynthetic drainage materials are presented in Table 4.8.

	$K_{saturated}$ [m/day]	$VWC_{saturated}$ [-]	α [kPa]	n [-]	m [-]	Thickness [mm]
Initial analysis	300	0.92	3.03	3	0.667	3
FCf6	7200	0.49	3.03	3	0.667	5
GPT5	370	0.7	3.03	3	0.667	6
HPS5	300	0.92	3.03	3	0.667	6

Table 4.8 Hydraulic properties of geosynthetic drainage materials and van Genuchten parameters selected for the analysis. Note: volumetric water content (VWC).

4.3.4.1   **Event 'A'**

Only the cuspated core drainage layer responded to event 'A' that occurred in September 2012. It is suspected that vegetation (increased water storage capability, which can be reflected in Vadose/W modelling) and thicker soil layer on the two other materials retarded water transport within the other sections. No water table level was detected during this short rainfall event. Figure 4.20 presents the drainage layer flow data collected during and after the rainfall event and attempts to calibrate the model. Adjustments were implemented into climatic boundary conditions (time of the event occurrence shortened but precipitation amount unchanged), soil properties (permeability increased) and geocomposite properties (retention function sensitivity runs) in order to replicate *in situ* behaviour. Geosynthetic drainage material retention function is not among the standard tests carried out by manufacturers. Hence, for this study data was based on the values reported by Iryo & Rowe (2005).
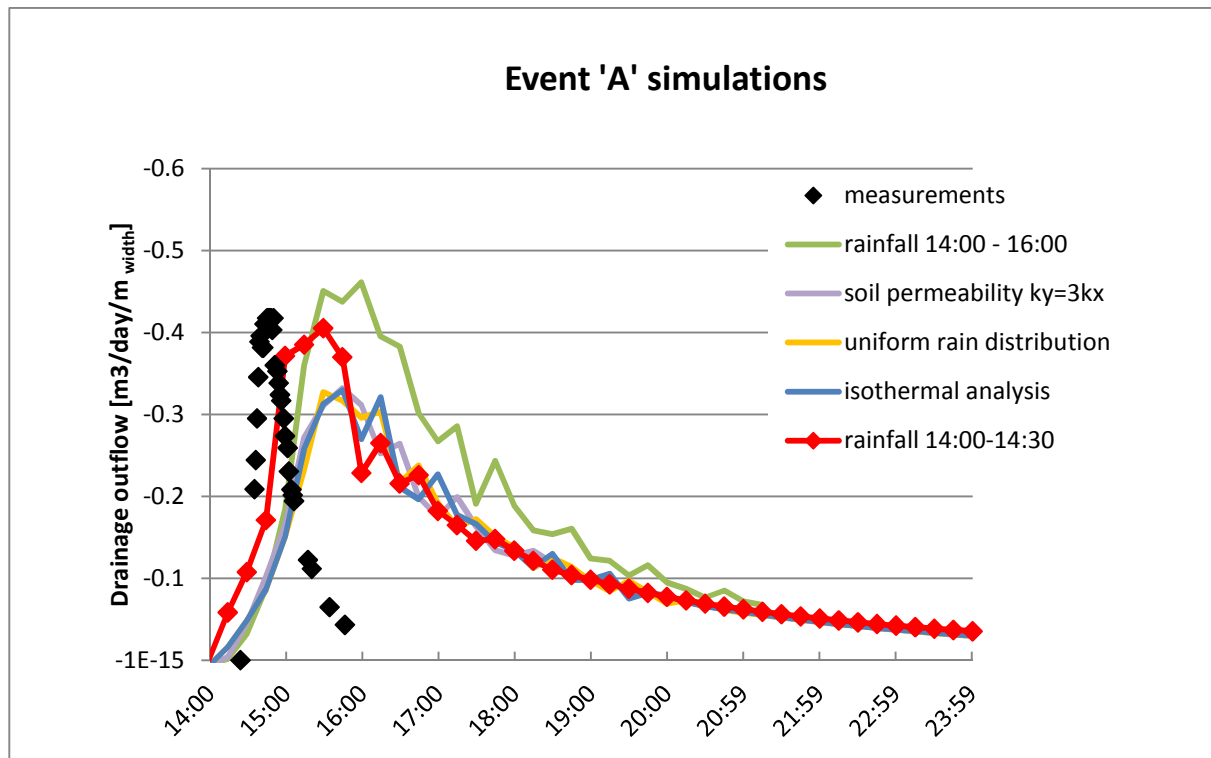
Figure 4.20 Example of the model calibration attempts (per meter width) against the recorded Event 'A' (note: the legend indicates a simulation key specification).

It was observed that the macro scale permeability of the cover soil layer is much higher than indicated by laboratory soil permeability tests or *in situ* infiltrometer tests. The monitored water flow was controlled by the permeability of soil cracks and fissures through preferential flow paths. Therefore, while part of the water entered the system of preferential paths, the remaining water infiltrated into the soil mass and increased its general volumetric water content (Figure 4.21). The modelled quantity of water leaving the drainage system (area under the curves) is much higher for the Vadose/W outputs than the monitored *in situ* flow. The software does not allow differing or scaling of material spatial permeability. This is acknowledged as a limitation of the model. Hence, replication of the site results was limited. However, the model gives a valid indication about the response time and peak flow rate occurring in certain atmospheric conditions. This confirmed that for carefully selected input data the model provides valid information on drainage performance. Hence, the conclusion

was drawn that the numerical simulations in Vadose/W incorporating geosynthetic drainage materials and its response to storm events still brings valuable information. Hence further simulations were carried out.
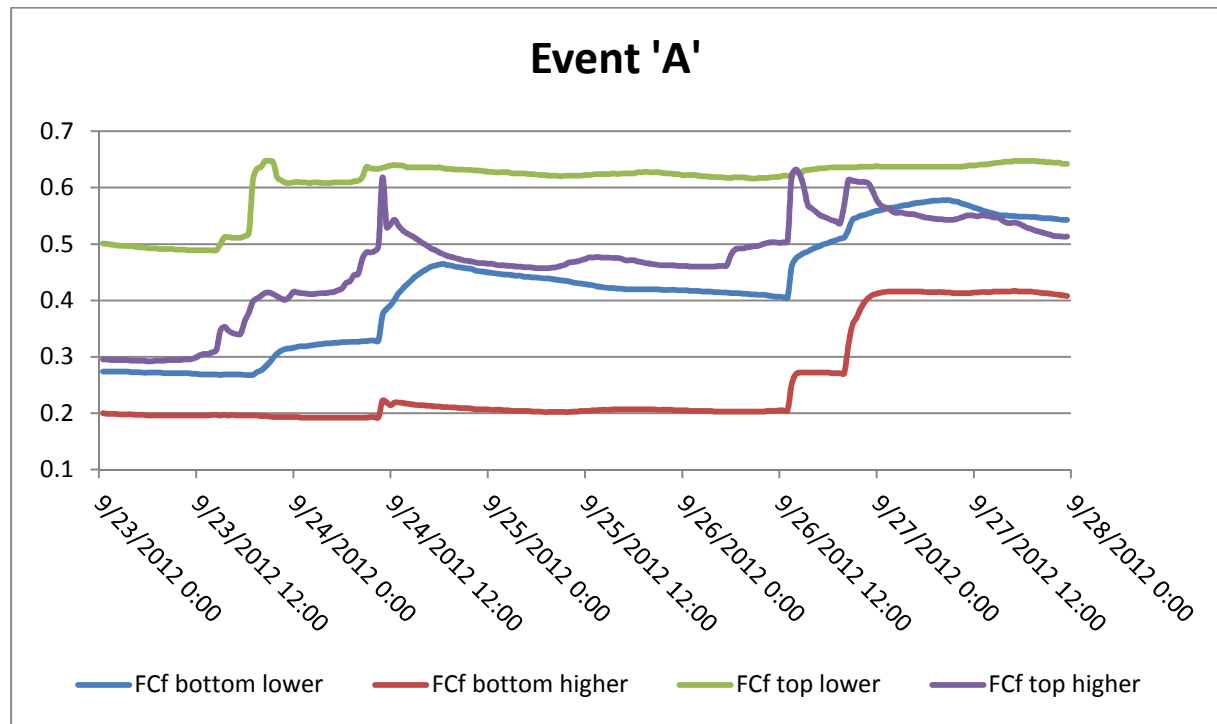
Figure 4.21 Volumetric water content readings (note: index 'bottom' indicates sensors 38m below the crest, 'top' indicates sensors 20m below the crest).

4.3.4.2   Event 'B'

Event 'B' occurred on the 24th of September 2012. This was a day with high precipitation throughout the day and flooding risks throughout the country. The total amount of rain reported on that day was 9.9 mm (W1). It was decided to model/analyse the 'B' and 'C' events using the model parameters calibrated using event 'A', as for this event detail geocomposite response to precipitation was recorded. The aim was to analyse the events, evaluate drainage *in situ* performance and hence to help assess Vadose/W applicability.

During event 'A' geocomposite flow rate in response to a short rainfall event was fully described by thirty measured data points. Only limited data on drainage outflow was recorded during the event 'B' (4 data points), hence defining the correlation or model verification is

limited in this case and conclusions are based on both the actual event 'B' modelling and event 'A' results. Figure 4.22 presents the drainage layer outflow data collected on the 24/09/2012. For modelling purposes adjustments were implemented for the weather boundary conditions: time of the event occurrence selected to reflect accurately the water which entered the system and initiated the recorded flow rates. It was observed that to represent the actual recorded drainage flow rates, careful analysis of rainfall period and amount has to be carried out in order to replicate *in situ* behaviour.

It is notable that selection of accurate weather data boundary conditions influences significantly the final result. Additionally, Vadose/W does not allow modelling of several rainfall events on one day or rainfall events with highly changeable rainfall intensity throughout a single day. Only one event per day is allowed and one out of three scenarios of rainfall distribution during the event: average, sinusoidal or constant grow and dropping can be selected.

Due to insufficient data between the two first monitoring times (Figure 4.22), it is difficult to justify which of the approaches represent the best actual *in situ* drainage response. It is not clear whether the recorded data presented had already peaked, since the highest intensity of the precipitation occurred within the hour before the measurements of flow. These simulations shows how sensitive the modelled parameters are to applied input weather boundary condition, and how selection of various rainfall periods can generate different results. Additionally, during event 'A' it was observed that rapid increase in soil volumetric water content indicates directly the change in drainage and storage action and this could occur in between the readings observed during event 'B'. Hence, the scenario with modelled increased flows 1.2 - 1.6 l/s could occur *in situ* (Figure 4.22 *e.g.* green line, blue line).

Figure 4.22 Attempts to calibrate the model (per meter width) against recorded Event 'B' (note: the legend describes event occurrence and rainfall amount considered in the simulations).

4.3.4.3   Event 'C'

Below the results of the event monitored on the 11th and 12th June 2012 are presented, when the highest flows in the drainage materials were monitored. Precipitation recorded on the 11th of June in the area was 23.6 mm (WG1) / 28.4 mm (WG2), with a relatively constant rainfall rate occurring between 3:00-15:00 h and increased intensity between 5:30 h – 8:00 h and 11:00 h - 15:00 h. On the 12th of June low precipitation with medium intensity occurred in the early morning hours (3:30-7:30 h) with totals of 3.3 mm (WG1) and 2.3 mm (WG2). For these conditions, increased drainage flows and piezometric water levels were observed (Figures 4.23 and 4.24) for all of four monitored sections.

In situ monitored behaviour

The flow and piezometers readings were taken between 10:00 h - 16:00 h every two hours. During this observation period, the highest flow rate was recorded for the band drain reaching approximately 0.16 l/s/m$_{width}$. The peak flow corresponds with the highest recorded piezometric water level on the band drain geocomposite - geotextile section; the peak was reached at 10:00 h. The band drain geocomposite standpipes recorded up to 5 cm of water above the drain elements at the middle section of the slope at 12:00 h (Figure 4.23a), in response to cumulative rainfall reaching a value of 1.5 cm at the recorded peak. In general for the non-woven geotextile and band drain geocomposite the piezometric levels increased up to 10 cm and then decreased. This can be explained by the breakthrough head effect that is a trigger for flow caused by a difference in pressures that allows flow through the material. The piezometric level above the geosynthetic was dropping while the relative high precipitation rate was still occurring.

Figure 4.24 indicates clearly that the band drain geocomposite had the highest flow rate. However in relation to the previous analysis and monitored behaviour, it is expected that the cuspated core drainage material must have responded directly due to the rainfall event beginning (3:00 h), when the flow was not measured, and instantly was draining the excess of the water. The non-woven geotextile and band drain geocomposite responded with maximum flow capacity when their degree of saturation reached an optimal point.

Based on previous analysis it is expected that volumetric water content sensors indicate the time when the cuspate core geocomposite started draining water (*i.e.* Figure 4.21 shows that at 5:30 h a rapid increase in the soil volumetric water content occurred). Piezometric water levels were recorded in standpipes at all three drainage materials and within the soil section (Figure 4.23). In general, piezometric level peaks were recorded at 12:00 h and 14:00 h, with increased values at the slope toe compared to the mid points (Figure

4.23b). Unexpectedly, the highest recorded water level occurred above the cuspated core geocomposite layer reaching 17 cm, which at this location is more than a half of the cover soil layer thickness. Although at the same time no water level was recorded at the standpipe in the mid-section of the panel and lower flow rates (compared to the other geosynthetics) were monitored for this drainage layer during this period. This would indicate that the drain was not full and that water is perched on the top of the geotextile filter. This phenomenon was further investigated on site and confirmed, it appears that the geotextile filter was partially clogged within the toe section (Figure 4.25).

Modelling

The aim of the event 'C' analysis was to model recorded conditions and simulate monitored flow quantities for the three drainage materials, to investigate the process of outflow development and compare drainage material performances. As discussed in section 4.3.1 two functions control material behaviour in Vadose/W: water retention curve and permeability coefficient versus soil suction function. However, the modelling exercise carried out to modify these functions could not reproduce *in situ* drainage behaviour. It was not possible to obtain constant high flow throughout the day for the modelled drainage material, for the recorded rainfall rates, using increased permeability properties. The model could not produce recorded flows for the applied climatic boundary conditions. The approach incorporated in the software does not represent the actual *in situ* behaviour accurately (*e.g.* increased infiltration rate for the dry soils where suction pressures occur, macro pores, cracks, fractures *etc.*). Figure 4.26 shows attempts to replicate cuspated core material behaviour, but these poorly reflect actual behaviour and reveal high model sensitivity to simulation 'rainfall period' selection. This research project highlighted Vadose/W limitations associated with modelling a single rainfall event. Replicating prolonged daily rainfall with various intensity and highly transient materials conditions is challenging and complex.

a)

Stand pipe piezometers level
11-12.06.201
MIDDLE

b)

Stand pipe piezometers level
11-12.06.201
BOTTOM
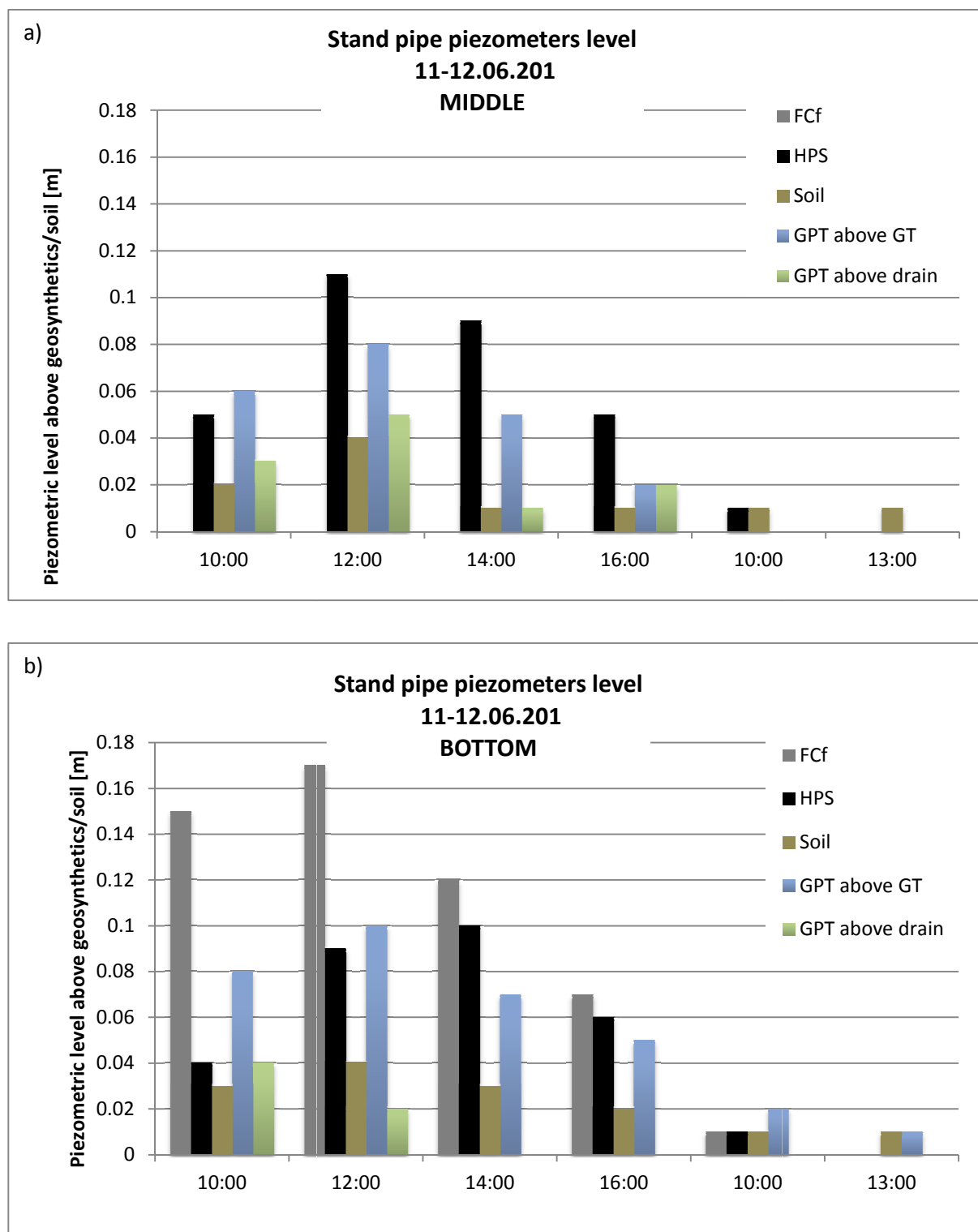
Figure 4.23 Piezometric level monitored on the 11-12.06.2012 on the site for two locations on along the slopes: a)in the middle and b) at the slope bottom.
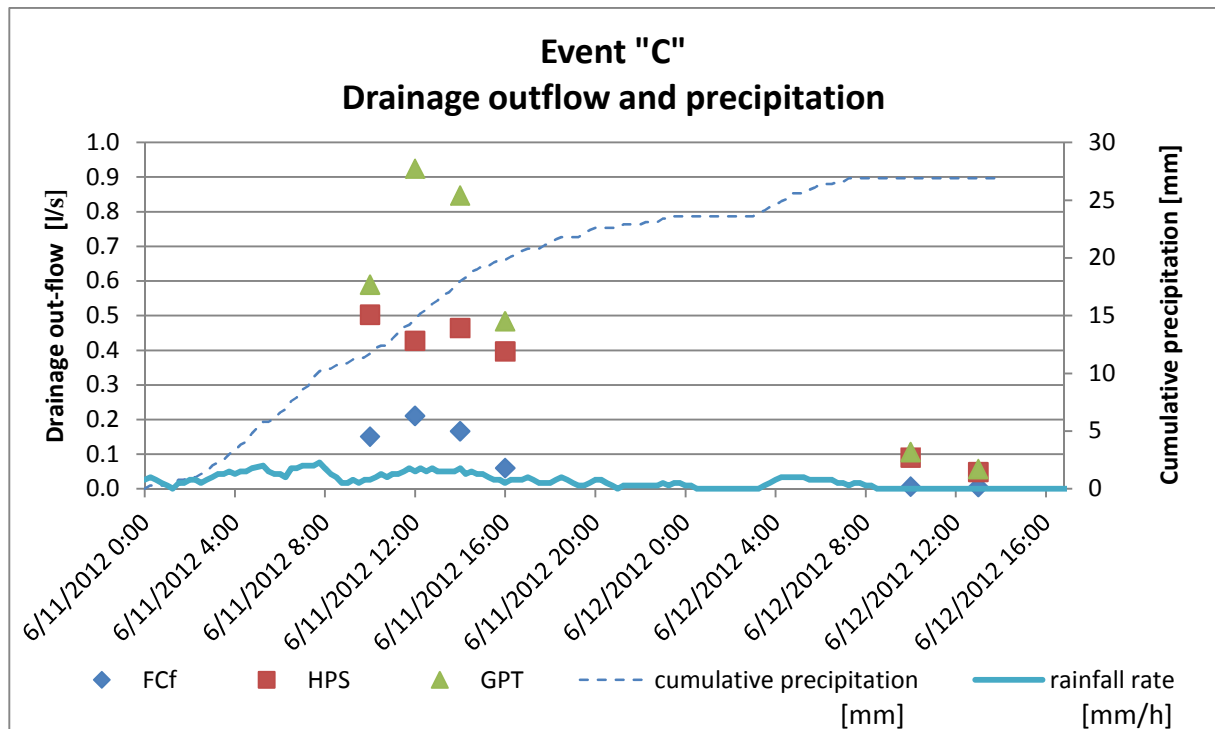
Figure 4.24 Cumulative precipitation, precipitation rate and geosynthetic drainage systems out-flow.

Figure 4.25 Site investigation, identification of filter partial clogging - the drain was operating properly, note the standing water on the filter.
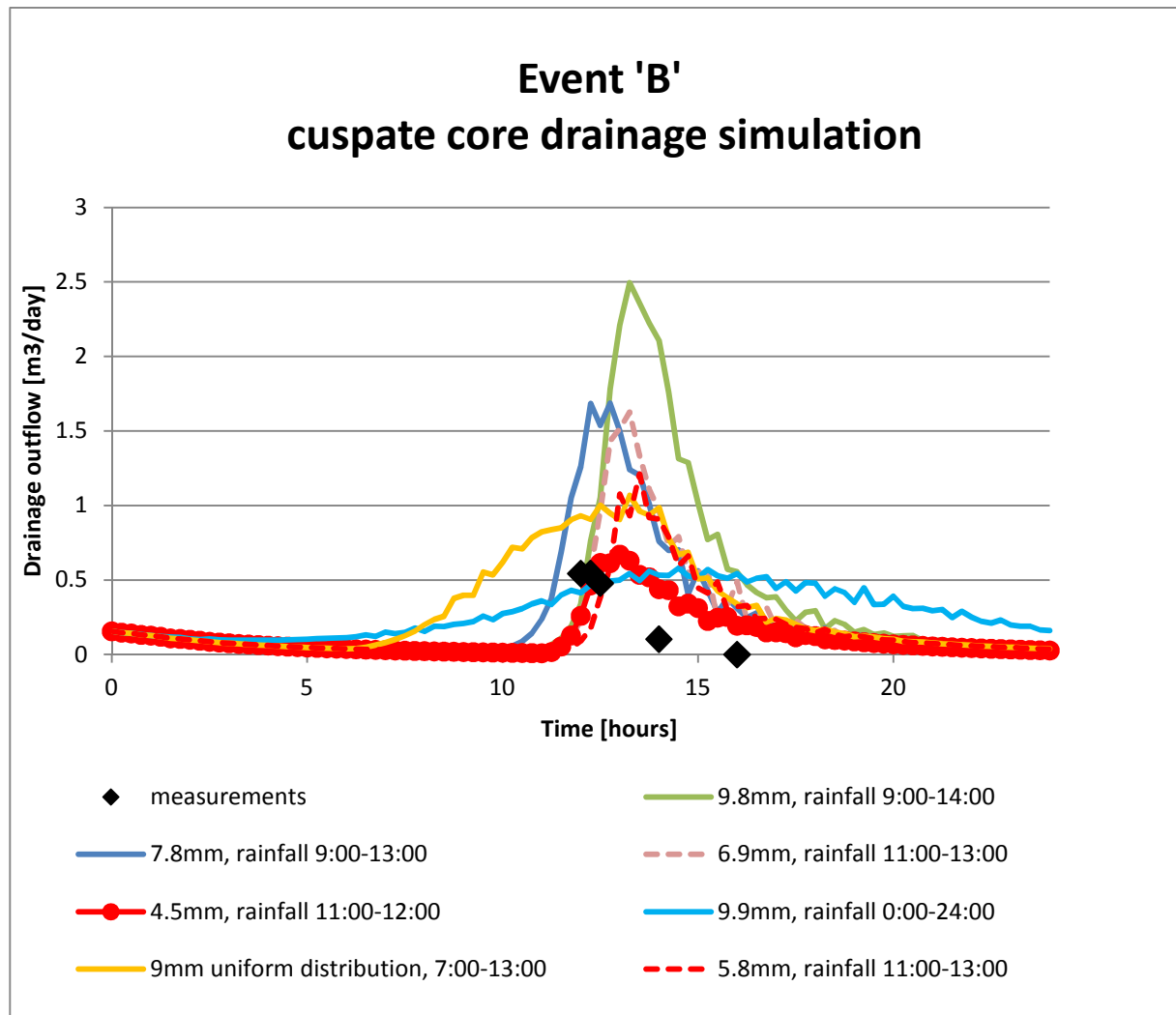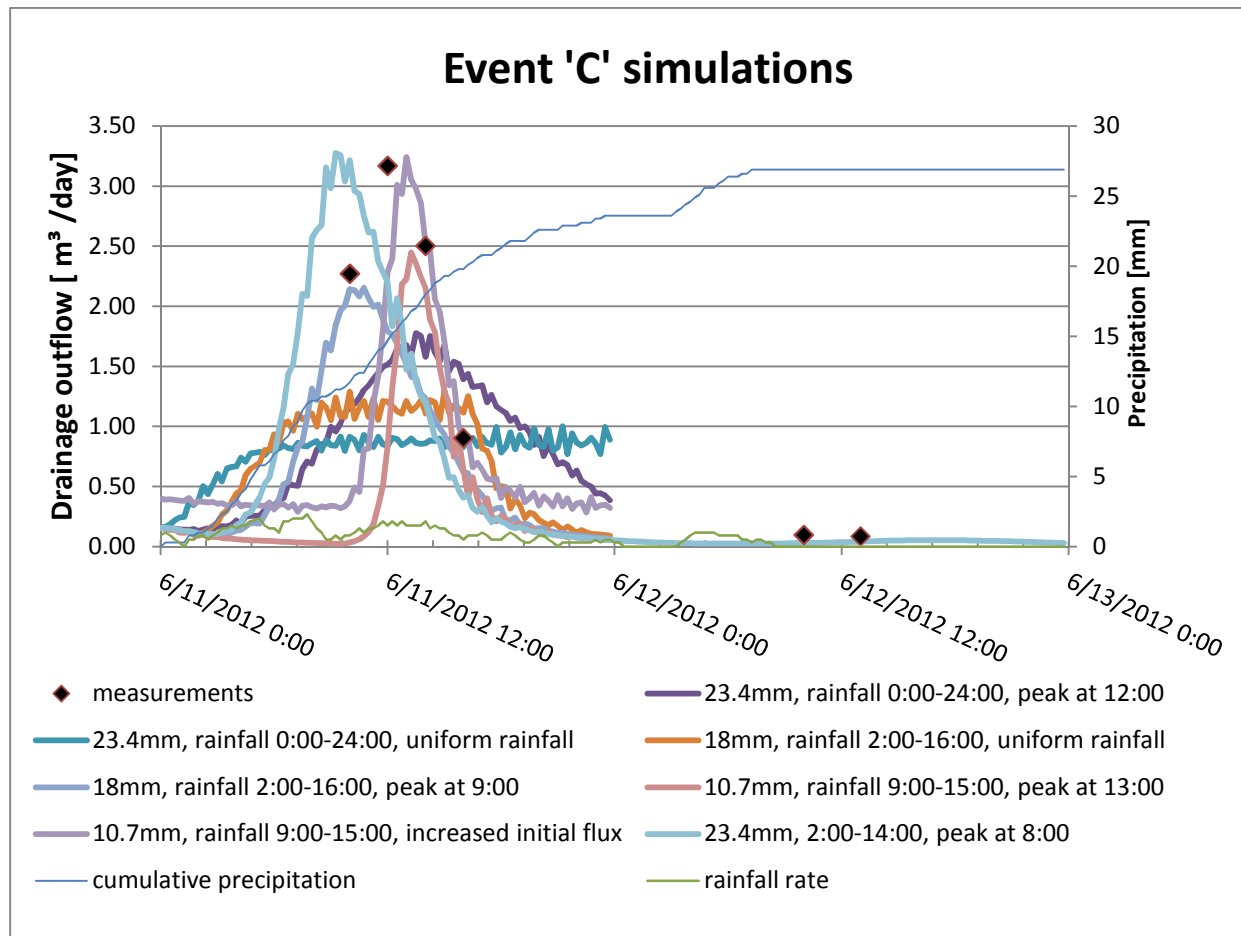
Figure 4.26 Attempts to calibrate the model (per meter width) against recorded Event 'C' (note: the legend describes event occurrence and rainfall amount considered in simulation).

4.4 SUMMARY

The side slope lining system and the capping system projects delivered novel information regarding the systems in service behaviour. This has improved existing knowledge of the mechanisms governing geosynthetic in service performance and revealed additional aspects affecting materials behaviour. The limited agreement between monitored and modelled geosynthetics behaviour, allowed investigation and definition of additional aspects controlling the materials performance:

(1)   Geomembrane thermal behaviour controls geotextile increased displacements; and

(2)   Landfill capping soil cracks which affect permeability properties, control capping hydraulic regimes and drainage layer performance.

The consistent difference between modelled and monitored behaviour allowed identification of mechanisms of lining systems specific behaviour and definition of limitations of the applied models.

5 FINDINGS AND IMPLICATIONS

This chapter presents the main findings drawn from this study. Discussion of the findings, contribution to the existing theory and practice, implication on the sponsor and the wider industry with recommendations for the industry and future research are presented in this chapter.

5.1 LANDFILL SIDE SLOPE LINING SYSTEM - DISCUSSION

The Milegate Extension Landfill monitoring project was conducted for over three and a half years. The first phase of the project has been completed, however continuation of the monitoring will investigate lining system performance during waste degradation and settlement. The aim of the project was to validate numerical modelling design approaches of the landfill lining systems. Data including stresses imposed on the slope, geosynthetics displacements, strains and surface temperature are presented in this work.

Interface shear strength of exposed materials and FLAC modelling

This study was to indicate which interface shear strength parameters (peak/residual/strain softening) represent the in service conditions accurately. However instead of the clear guide, the project revealed a further possible option: increased displacement induced by temperature related stresses. In this case neither peak, residual or strain softening parameters were defined by the *in situ* behaviour. The interface parameters were analysed in this study in a limited extend as one of the major mechanism driving geosynthetic displacement was not represented in the model.

A significant factor related to atmospheric conditions has been identified as the driving force for geotextile displacements in the upper part of the slope (*i.e.* reduced interface shear strength of the lining components due to wrinkling of the exposed materials and ageing of the geosynthetics). In an attempt to replicate this behaviour, reduced values of the lining

system interface shear strength were assigned and modifications in soil and geotextile material properties applied.

It is acknowledged that sections of the Milegate Landfill site liner were exposed to weathering conditions and estimation of its influence on the liner performance is limited, therefore, analysis with reduced residual interface shear strength values were carried out to trigger movement on the clay / geomembrane interface and in an attempt to enhance geotextile movement. Overall peak and residual values were highly similar in terms of the material displacement and maximal axial forces occurring in the geomembrane. The interface strain softening subroutine triggered movement of the geotextile especially in the top section.

It is recognised that FLAC gives a valid approximation of the *in situ* lining system behaviour where the ambient temperature does not change significantly or if the geosynthetic materials are not directly exposed to solar radiation (*e.g.* when they are covered by soil). However, its applicability is limited when environmental factors play a significant role in materials performance.

If it was recognised that geosynthetics might remain exposed for a prolonged time, modified and reduced values for not only interface shear strength should be considered but also modified values for materials properties (stiffness). Extended material exposure (solar radiation and related increased temperature) is a significant factor affecting *in situ* material behaviour. It is recommended to conduct sensitivity analyses of interface shear strengths to account for ageing and solar radiation exposure.

Wrinkle formation

It should be highlighted that although the Milegate landfill slope was fully covered after three years the highest displacements occurred one year from the project start, hence a year of exposure is enough to trigger increased temperature related liner displacements.

It should be emphasised that for the section of slope covered by sand, where geosynthetics are not directly exposed to the environment, no increased deformations occurred within the lining components and values computed in a standard approach are in the range of the monitored ones. This indicates that numerical model applicability is not valid when prolonged exposure and thermal effects become the dominant mechanism of displacement.

5.2 LANDFILL CAPPING SYSTEM - DISCUSSION

Currently, in practice, landfill capping stability risk assessment is based on the limit equilibrium approach derived by Koerner & Daniel (1997). In order to verify actual pore water pressure development and distribution along the upper surface of a low permeability barrier/base of cover soil layer, simulations of the capping water balance were conducted along with site specific rainfall event simulations. Initial geometrical simulations in Vadose/W revealed that for certain soil permeability and geometry conditions, pore water pressures develop on a landfill cap to various degrees (*e.g.* PSR=0.3, 0.5 or fully saturated layer). However, the precision and accuracy of these analyses is not certain. Nevertheless, it is recognised that a drainage material with even very low transmissivity properties can help reduce pore water pressures within a cap. Instances where the top soil layer is left exposed for extended periods of time, desiccation and saturation processes lead to crack formation, which allow unopposed water inflow and soil erosion. This might lead to development of excessive pore water pressure above landfill impermeable barrier and could cause failure.

Vadose/W modelling

Although Iryo and Rowe (2005) proved good correlation of laboratory tests against modelled behaviour, modelling of a full scale experiment with complex variables and with spatial differences in soil properties (*e.g.* highly permeable cracks next to low permeability intact regions) brings limited agreement. Further, representation of the differences between three

drainage materials in Vadose/W, could not be achieved. It is possible to replicate with a certain accuracy drainage behaviour, however using Vadose/W tool for design purposes in order to quantify drainage layer requirements is insufficient. It was not possible to replicate water table development within the soil layer above geocomposite drainage materials, which would correspond to the high outflow – monitored situations.

Bletchley Landfill capping materials and the occurring processes are considered complex and significantly transient. Hence, modelling of drainage layer performance which would provide satisfactory results was limited. Additionally, for modelling cuspated core geocomposite material, the boundary conditions required several adjustments in order to meet monitored values. It is highly challenging to create a model which represents the system's complexity.   Vadose/W at this stage of validation is not considered to be commercially applicable to predict with confidence pore water pressure distribution along landfill capping and assess required geosynthetic drainage layer performance. The specific system configuration – relatively thin soil layer (commonly 1m thick), limited moisture exchange with below layers, possible thermal effects from wastes, additional waste settlements (*i.e.* triggering cracks within the veneer cover) comprises a landfill specific case. Vadose/W requires much site specific and material data which might be very sensitive in terms of the final outcome and bring incorrect results. It is acknowledged that the study does not provide full information on the drainage performance, however it indicates the nature of the problem.

PSR consideration

Considering Bletchley project results, questions arose to the accuracy of simulations that are based on average daily precipitation, as intensity of precipitation changes throughout the day, or occur within certain hours of the day. It is acknowledged that Vadose/W can deliver only limited information about drainage layer performance and its required parameters for these conditions.

Additionally, the project provided information on PSR formation within a capping system with relatively shallow inclination and different geosynthetic drainage layers in place. The study proved a low PSR development above the drainage layers for the site conditions investigated. Although it might be soil specific behaviour, in the circumstances where hydraulic parameters of soil are rarely tested, this implies significant stability risks for geosynthetic capping systems.

Due to the low gradient shallow slopes are expected to develop PSR. Slopes with geosynthetic drainage in place are expected to transport the entire water surplus outside the system, hence no PSR is considered for capping with drainage. However, this research shows that low PSR can develop above the drainage layer even when the drain is working effectively and this can influence significantly a capping systems stability.

5.3 KEY FINDINGS OF THE RESEARCH

It should be noted that conclusions drawn in this section for both projects, in some cases might be limited to the site specific conditions.

Landfill Side Slope Lining System Study – Milegate Extension Landfill Project

The process of validation of the commercially applied advanced design method has been successful. It allowed defining areas of good agreement between monitored and modelled geosynthetics behaviour and indicated areas requiring landfill designer concerns and further research.

The key findings of the project are as follow:

(1)   Integrity of geosynthetic lining system (HDPE geomembrane overlaid by geotextile) was monitored under *in situ* conditions. It was found out that lining system when exposed to atmospheric conditions reveals increased displacements of the geotextile (and geotextile stretching) triggered by expansion and wrinkling of the underlying HDPE.

(2)   The process of cyclic geomembrane expansion / contraction (heating / cooling) overall does not impact geomembrane movement as significantly as the geotextile.

(3)   Once covered by the sand veneer, mobilised displacement on geotextile / geomembrane interface remains within peak values. In overall mobilised displacements on geomembrane / clay interface also oscillate within peak strengths values.

(4)   The monitored performance was further modelled in FLAC. Good agreement was obtained between computed and monitored geomembrane behaviour, even for exposed sections, while geotextile displacements were significantly underestimated.

(5)   Overall for the Milegate Extension Landfill lining components interface shear strength parameters (peak, residual and strain softening), the model produced displacements with similar magnitude, which were underestimated for the exposed geotextile.

(6)   Materials once covered by a sand layer (then waste), behave in a predictable – modelled manner that can be replicated using numerical modelling software and procedures.

Landfill Capping Drainage System Study – Bletchley Landfill Project

The works on the capping project enhanced understanding of the processes occurring within landfill capping hydraulic regime. This contributed considerable implication to a current capping stability approach. The key findings of the project are as follow.

(7)   Site monitoring revealed that a 'transient' piezometric level develops in the cap cover soil along the cap also during the summer season due to short intensive rainfall events. This forms during the rainfall event, responds directly to rainfall intensity and disappears when rainfall intensity decreases. Most of the water is discharged by drainage elements within hours of the rainfall occurrence.

(8)   According to Vadose/W modelling, a water table might develop within the capping soil cover layer (PSR=1) without a drainage system during a wet / winter season. The

distribution and magnitude varies with the cap's geometry and soil hydraulic properties. This requires further site studies in order to confirm the model.

(9)   For certain conditions, nonwoven geotextile and band drain geocomposite develop a higher piezometric water level (PSR>0), than soil layer without drainage. Additionally, the cuspate core measurements show that filter selection is important in order to prevent clogging and resulting increased water heads above the drainage element;

(10)   Even for a relatively low permeability soil layer, geosynthetic drainage fulfils its function, since hydraulic processes within the top cover soil and drainage layer response is driven by permeability of cracks and fissures formed within the soil layer;

(11)   Limited agreement was met between modelled and monitored conditions, Vadose/W does not include sufficient functions to represent capping drainage conditions adequately. Further software development is required to accommodate spatial soil variability (*e.g.* cracks formation, preferential flow paths) and various rainfall rate distributions throughout one day only;

(12)   Additional field data is required to monitor constantly drainage outflow and pore water pressure above the drainage layer in order to evaluate its performance in response to weather conditions.

5.4   IMPLICATIONS AND RECOMMENDATION FOR THE SPONSOR AND THE WIDER INDUSTRY

The study sponsored by Golder Associates (UK) Ltd, allowed the company to remain within a leading position in the landfill design sector in the UK through the direct dialogue with the main company landfill designers – project supervisors. The topics investigated in this study were directly associated with the sponsor's research demands. Additional involvement in conferences and personal communication with G. Fowmes allowed the main findings to be distributed to the wider industry.

The process of numerical modelling validation revealed factors affecting significantly lining system performance. The indicated factors limiting the software and areas of concern for a landfill designer were highlighted:

Landfill Side Slope Lining System Study – Milegate Extension Landfill Project

(1)   In order to reduce displacement between the components and strains within the components of a multilayered lining system it is recommended to cover geosynthetics as soon as possible after installation;

(2)   It is possible to roughly assess displacements of the lining system components exposed for a prolonged time using air temperature records;

(3)   Provided the lining system is not exposed for a prolonged time, peak interface shear strengths are considered acceptable for a design process where configuration of the lining system is similar to the one presented in the thesis (*i.e.* slope length of 30m, inclination 21.8°);

Landfill Capping Drainage System Study – Bletchley Landfill

(1)   It is recommended to consider min 0.1m of water table above drainage layer along the capping in the design process. This is to accommodate break through head effect that might occur above drainage layer.

(2)   It is recommended to apply conservative values of the interface shear strength between veneer cover and underlaying material. This is due to slip surface developing between soil / geosynthetic materials..

5.5 FURTHER RESEARCH

5.5.1 SIDE SLOPE LINING SYSTEM

The full scale experiment revealed additional precautions that should be considered in the design process that encounters factors related to liners exposure time. The applicability of a numerical modelling approach to evaluate landfill lining system performance (integrity

issues) was confirmed except the limiting case of exposed materials. Along the sections of the slope where geosynthetics were not directly exposed to atmospheric conditions, mobilised interface shear strengths oscillate within the peak values.

This research formed further questions that could not be addressed within this study due to time constraints. These concern contaminant barrier interaction especially the conditions of exposed geosynthetic materials. These are as follows:

(1)   How interface shear strength subjected to very low normal stresses (*i.e.* from geotextile self-weight) is influenced by the shearing / displacement process? And how the same interface will later respond to shearing under a high normal stress *i.e.* is the interface strength weaker? / stronger? / no change?. This further work should include weathered materials, with depleted strength properties;

(2)   Quantification of wrinkles in exposed landfill lining system geosynthetic components in terms of their influence on total displacement / relative displacements of interfaces;

(3)   Geomembrane thermal expansion on a steep wall lining system, wrinkles naturally disappear under the geomembrane's self-weight, but are these suspected to reduce membrane tensile properties?

(4)   It would be challenging to model HDPE response to various thermal versus time conditions. FLAC software has a feature allowing incorporating thermal factor, which should be explored in future research;

(5)   Direct monitoring of geomembrane wrinkle formation on inclined planes in order to observe / quantify movement in a down the slope direction due to cyclic processes (heating / cooling), could be conducted in laboratory conditions. Further studies could include overlaying a geotextile in a dry and moist condition;

(6)   Milegate data collected on the lining performance is difficult to interpret due to extended exposure to the atmospheric conditions, therefore to gain further insight of liner

mechanical response, it would be of value to conduct monitoring of a slope where the sand (protection / drainage) layer was placed immediately in one phase along the slope, and where waste placement progresses faster. Analysis of the climate data and its influence on the liner temperature and overall performance are uncommon and much more complicated than the standard design approach. However, attempts should be given to estimate environmental impacts on the geosynthetic performance to better represent actual conditions, to limit possibility of failure occurrence and to assure safe construction.

(7)   *Demec strain gauges*. The method proved applicable in the landfill environment, although the process of reading collection on the site can be demanding. However, they delivered valuable data in a cost effective manner and therefore could be applied in future geomembrane studies.

(8)   *Fibre optic instrumentation*. This is believed to deliver valuable information and further application in this environment is possible, however it is of high importance to ensure robust protection to fibre optic strings, when installed on a site.

5.5.2 CAPPING DRAINAGE SYSTEM

In terms of actual landfill drainage optimised design, Vadose/W has only limited applicability and it is not recommended to apply this model solution in commercial field applications, unless modified and fully justified. This is due to difficulties in reproducing *in situ* soil hydraulic conditions and rainfall patterns. However Vadose/W confirmed that for a permeable material the cap soil performs as a drainage layer itself and for a low permeability capping soil water penetration is delayed. Hence, the medium permeability soil ($\sim 10^{-5}$, 10^{-6} m/s) can require use of a drainage layer. Also, higher saturation of cover soil is expected for shallow slopes (*e.g.* 1v to 8h), than for the corresponding steeper slopes (*e.g.* 1v to 2.5h).

Additionally, attention should be given to filter selection when considering drainage materials, so to avoid / minimise clogging risks or break through effect.

When selecting textile drainage layer, for stability risk assessment it is advisable to still consider low PSR values, corresponding to at least 0.1 m water head level above the drain.

Further, this research confirmed that for interface shear strength properties, in order to reflect increased volumetric water content within a thin soil layer directly above drainage material, tests results for submerged samples should be used.

This research formed further questions that could not be addressed within this study due to time constraints. These are as follows:

(1)   To better understand mechanisms controlling drainage system performance and to conduct full comparison and analysis of different types of drainage materials, instrumentation logging drainage outflow and pore water pressure above the drainage layer constantly is crucial; additional mapping of fissures and further refinement of the model could allow better representation *in situ* conditions in the model.

(2)   Further study of various top soils, in order to quantify and categorise influence of soil type, crack system development, cracks permeability and degree of vegetation on a slope in relation to infiltration and drainage layer performance.

5.6 CRITICAL EVALUATION OF THE RESEARCH

Best practice was followed when installing instrumentation, reading acquisition and validation of representative numerical models. Nevertheless, it is acknowledged that the projects have several limitations related to various aspects of the studies. The projects were conducted in order to ensure the best research practice and to collect accurate and reliable data Although, it is well known that field works often encounter unexpected difficulties or are subjected to *in situ* /environmental conditions, still acquired data is of significant scientific and commercial value, as it enhances understanding of the process contributing to design approaches.

To the Author's knowledge in the current literature there is a lack of practical remarks regarding particular instrumentation installation or performance, hence it was decided to list below remarks / recommendations /'lesson learned' of this study:

Landfill Side Slope Lining System Study – Milegate Extension Landfill Project

(1)   One geomembrane/geotextile panel was deployed and monitored, the project could be improved by conducting parallel monitoring for cross checking, however, this involves additional costs, therefore efforts were made to deliver high quality, reliable data from the available resources;

(2)   All instrumentation is subjected to temperature changes. Thermal corrections are based on the coefficients available from the literature (details in Appendix B), but no direct thermal calibration of the materials and instruments were conducted;

(3)   Extensometer readings represent localised movement at the six points installed on the geomembrane and geotextile, hence these may not adequately reflect the behaviour of the entire material sheet (this refers mostly to geomembrane and wrinkle formation);

(4)   Strains calculated from extensometer records are averaged over 5.4 m gauge length and might not reflect localised behaviour;

(5)   Geomembrane strain information acquired from fibre optics are more comprehensive (the gauge lengths are shorter) than the extensometers. However, sensors were lost through the project and therefore limited information is available from these (last readings were collected prior to the 3rd sand veneer placement);

(6)   Most of the instrumentation was installed along the middle part of the panel and plane strain conditions were assumed. Therefore, it is expected that necking effects are minimal;

(7)   Additionally, it is recognised that numerical modelling design approaches would benefit if more of this type of monitoring data was available, particularly for different lining system configurations, geometries and with consideration given to various rates of waste

placement (not only in terms of geosynthetics loadings but also exposure to surrounding environment);

(8)   It is of importance to record each stage of the site works in pictures, that can significantly help in further analysis and justification of the approaches (*i.e.* wrinkles formation and further development).

Landfill Capping Drainage System Study – Bletchley Landfill Project

(9)   Only one panel of each geosynthetic drainage material was deployed and monitored. The project would gain by conducting parallel monitoring for cross checking and excluding local errors / issues;

(10)   Piezometers and volumetric water content sensors were installed along the middle part of the panel, therefore it is assumed that the edge effect was reduced to a minimum on the panels ~six metres wide;

(11)   Single sets of the instrumentation have been installed (except piezometers on the band drain geocomposite where standpipes were installed over a section of nonwoven geotextile and over a band drain). Examination of results in between two or more geosynthetic panels in the same arrangements would be beneficial for the project, however, due to financial restrictions was not possible;

(12)   Additionally, it is recognised that numerical modelling design approaches would benefit if more of these types of monitoring data could be provided, particularly for different geometries (capping length, angle and cover soil thickness), and cover soil parameters with consideration given to various rate of vegetation covering the surface;

(13)   This project was dependent on weather conditions / weather forecasts, which turned out to be highly inaccurate and unpredictable; the project could have gained significantly if the system logging water discharge from the drainage layers was installed. Initially the dynamic of the system was not known, it was not certain if the flow would have occurred, and

therefore it was decided to consider its installation on future project stages but was not cost effective;

(14)   It is of importance to record each stage of the site works, as it can significantly help in further analysis and justification of the approaches (*i.e.* cracks photographed in the soil, water overflow from surrounding area and development of vegetation);

(15)   If other stakeholders are involved in the works on the site (*i.e.* materials installation - contractors) it is of significant importance to be present on the site, when works are completed, to ensure satisfactory work accomplishment, in case of unpredicted issues support and for best project solutions. Solutions proposed by other parties might not necessarily support project purpose or future works;

(16)   It is advised to make maximum testing, simulations, calibrations or preparations in laboratory conditions, even if the task seems to be simple to carry out on site;

(17)   Although Vadose/W provides relatively complex methods for solutions, still it is challenging to express site conditions in the idealised model. It requires specialised data that are not normally required within standard laboratory soil investigations;

(18)   Vadose/W applies differentiated permeability of the soil depending on negative suction. This retards the quantity of the infiltrated water modelled in unsaturated conditions. Actual infiltration rate for dry soil is relatively high at the beginning of precipitation (fissures / cracks offer unopposed flow until they saturate) and decreases throughout the event down to soil actual permeability rate value.

5.7 NOVELTY OF THE RESEARCH

Application of relatively novel materials, continuous upgrading of technologies and development of new solutions, without verification *in situ* performance, occurs frequently in geosynthetic/landfill sector. Therefore, it has been emphasised that site derived data is

required in order to enhance knowledge and understanding of *in situ* processes. This becomes challenging when the results do not meet the initial expectations / prediction.

A significant proportion of this research project comprised field studies and delivered information from full scale experiments, with some novel application of standard instrumentation *e.g.* Demec strain gauges and fibre optics. The two site investigations contributed to knowledge on *in situ* material performance and defined factors strongly influencing system performance. These helped to determine areas of additional limitations of standard numerical model approaches, and contributed to understanding actual processes taking place throughout a material in service lifetime. Additionally, uncertainties related to novel modelling approaches were highlighted.

5.7.1 SIDE SLOPE PROJECT

A standard method of strain measurement in concrete sciences was used to measure strains in the HDPE geomembrane using Demec strain gauges. This delivered valuable data on material deformation along and across the slope. To the Author's knowledge this instrumentation was used for the first time in this application. This is due to required direct access to the measurement points, which in this case was provided. Additionally, strain measurements across the geomembrane sheet placed on a landfill side slope were carried out. To the Author's knowledge there is no data available about HDPE *in situ* performance in a cross-slope direction.

Fibre optics which are a relatively novel instrumentation were deployed on the site, although most of sensors were lost throughout the filling process, they proved their applicability. Additionally, they are becoming more popular in the wider geosynthetic industry and materials with embedded sensors have been reported to deliver information on geosynthetic performance in various applications (*e.g.* Nancey *et al.* 2007).

5.7.2 **CAPPING PROJECT**

In the available literature there are no studies investigating actual pore water pressure development and distribution along a landfill veneer capping system. Many of the reported numerical modelling studies investigate capillary break effects, or present laboratory scale studies. Therefore, the hydraulic capping analysis incorporating transient, climatic data provides knowledge and is an innovative approach.

Additionally due to lack of initial financial sponsorship for the capping project, the instrumentation applied in this project was limited to low cost equipment (standpipe piezometers – use of waste pipes, drained water collection system – use of guttering). The capping monitoring system was simplistic and cost effective, but well-designed solutions did prove significant in terms of providing useful data. This kind of study has not been reported before.

The *in situ* data was used to verify sophisticated numerical modelling software. In the landfill engineering sector, these usually come from laboratory tests (*e.g.* Iryo & Koerner 2005). There is a significant lack of site derived data of material in service performance. Therefore research aiming to verify those models against *in situ* performance is highly essential, to allow further development in this area. This project indicated areas of further concern for a landfill designer *i.e.* pore water pressure build up over a capping drainage layer and confirmed formation of a slip plane on the drainage layer / soil veneer interface.

During the process of site data analysis, model verification and calibration, unexpected aspects of material specific performance were identified, mechanisms described and topics for further investigation were indicated.

6 **CONCLUSIONS**

Even though regulations exist to provide technical assessments for an adequate landfill design, stability (global instabilities) and integrity (local small scale movements / deformations) failures within landfill construction still occur. While stability can be assessed applying limit equilibrium methods, integrity requires use of numerical models. Both approaches require appropriate selection of interface shear strength properties. It is common to consider peak, residual or factored values to accommodate various uncertainties. To investigate which approach better represents field conditions and to validate numerical modelling Milegate Extension Landfill project was designed and conducted.

Site derived data is valuable for future development of engineered landfill facilities not only for optimised and economic construction but also to increase confidence in design and verification of rapidly developing numerical modelling approaches. Field data including stresses imposed on the slope, geosynthetics displacements, strains and surface temperature are presented in this thesis. The project revealed an additional factor influencing lining component increased displacement along the slope: thermal effect on the geomembrane and geotextile.

Numerical modelling of the *in situ* conditions has been undertaken in FLAC software. Due to limited agreement between modelled and monitored behaviour, an additional study has been undertaken to investigate a mechanism causing increased geotextile movement. This revealed significant geotextile displacements (and stretching) driven by HDPE thermal deformation within exposed sections of lining system. This occurred due to prolonged liner exposure to atmospheric conditions then typically encountered.

FLAC proved its applicability within sections covered by the sand veneer, and HDPE geomembrane displacement modelling produced reasonable agreement.

Furthermore, analyses were undertaken to investigate pore water pressure development and distribution above the low permeability cap barrier, within the top soil of a capping system. The simulation in Vadose/W demonstrated water table development depending on a slope geometry and soil permeability. Intrusion of low transmissivity drainage layer helped to reduce water level within the soil layer. However, this part of the study requires further site investigations with instrumentation monitoring constantly pore water pressures above the low permeability barrier.

Additional site investigation took place to validate the model. Monitoring instrumentation was installed on a landfill site capping. The instrumentation comprised of volumetric water content loggers, piezometers, a runoff plot, drainage layer outflow monitoring systems and a weather station. These allowed recording of data used to describe the water budget of the landfill cap.

Attempts were undertaken to validate Vadose/W model against site derived data. Through a number of simulations and adjustments only limited agreement was observed between the data. Vadose/W was recognised not to model aspects of various geosynthetic drainage materials suitably. A further limiting feature was related to rainfall occurrence and its' daily distribution.

The capping study allowed production of recommendations enhancing future capping designs. These can be used was to considerably enhance capping stability.

Site derived data is crucial for further improvement within the industry. It delivers information on already identified processes but also can reveal further implications and aspects worth considering in design. Numerical modelling software are powerful tools for studying geosynthetic *in situ* behaviour, however, when there is no validation against *in situ* performance limited confidence can be given to their output.

Novel implementation of numerical modelling codes should be accompanied by detailed site studies and monitoring in order to verify and justify these models, investigate simplifications, better understand site specific factors controlling behaviour and add confidence in model predictions and future designs.

7 **REFERENCES**

Albright,W.H., Benson,C.H., Gee,G.W., Abichou,T., Tyler,S.W., Rock,S.A., 2006. Field Performance of Three Compacted Clay Landfill Covers. Vadose Zone Journal 5:1157–1171 (2006).

Anubhav, Basudhar, P.K., 2010. Modeling of soil-woven geotextile interface behavior from direct sheartest results. Geotextiles and Geomembranes, 28(4), 403-408. [doi:10.1016/j.geotexmem.2009.12.005]

Arab, 2011. The Integrity of Geosynthetic Elements of Waste Containment Barrier Systems Subject to Seismic Loading. PhD thesis, Arizona State University. July 2011.

ASTM D4491-99a 2009. Standard Test Methods for Water Permeability of Geotextiles by Permittivity.

Aubertin, M., Cifuentes, E., Apithy, S.A., Bussiere, B., Molson, J., Chapuis, R.P., 2009. Analyses of water diversion along inclined covers with capillary barrier effects. Canadian Geotechnical Journal, 2009, 46(10): 1146-1164, 10.1139/T09-050.

Bacas, B.M., Konietzky, H., Berini, J.C., Sagaseta, C., 2011. A new constitutive model for textured geomembrane/geotextile interfaces. Geotextiles and Geomembranes Volume 29, Issue 2, April 2011, pp.137-148.

Bamforth, A., 2008. Role of Platens Hardness on Interpretation and Use of In-Plane Flow Capacity Test results for Geocomposites. 4th European Geosynthetic Conference, Edinburgh 2008.

Bohnhoff, G.L., Ogorzalek,A.S., Benson,C.H., Shackelford,C.D., Apiwantragoon,P., 2009. Field Data and Water-Balance Predictions for a Monolithic Cover in a Semiarid Climate Journal of Geotechnical and Geoenvironmental Engineering, Vol. 135, No. 3, March 1, 2009. ©ASCE, ISSN 1090-0241/2009/3-333–348/$.

Bouazza, A., Gassner, F., 2005. Geosynthetics reinforcement in landfills design: Australasian perspectives. GeoFrontiers 2005 ASCE proceedings.

Bouazza, A., Zornberg, J.G., McCartney, J.S., Nahlawi, H., 2006. Significance of unsaturated behaviour of geotextiles in earthen structures. Australian Geomechanics Vol 41 No 3 September 2006.

Bouthot, M., Blond, E., Fortin, A., Vermeersch, O.G., Quesnel, P., Davidson, S., 2003. Landfill extension using geogrids as reinforcement. Discussion and case study in Quebec http://www.infogeos.com/view.php?ID=569.

Brink, D., Day, P.W. and du Preez, L. 1999. Failure and remediation of Bulbul Drive landfill: Kwazulu-Natal, South Africa. Proc. Waste Management Conference, Sardinia 1999, Cagliari, Italy, 555-562.

Campbell Scientific 2011 User Guide. CS616 & CS625 Water Content Reflectometers.

Cazaux, D. & Didier, G., 2000. Field evaluation of hydraulic performances of geosynthetic clay liners by small and large-scale tests. Geomembrane and Geotextiles Volume 18, Issues 2-4, Pages 163-178.

CDM, 2007. Construction (Design and Management) Regulations 2007. HMSO Statutory Instrument 2007, No. 320.

Department of Environment, Food & Rural Affairs, 2011. UK waste data and waste management. Waste and recycling statistics. ENV23 – UK waste data and management. 4 August 2011.

Council of the European Union (1999). Directive 1999/31/EC on the landfilling of waste. Official Journal of the European Union, L 182, 1-19.

DETR, 2000. Department of the Environment, Transport and the Regions 2000. Environment Agency and Institute for Environment and Health, 2000.

Dixon, N., Jones, D. R., 2003. Stability of Landfill Lining Systems: Report no.2 Guidance. Environment Agency Research and Development Technical Report P1-385/TR2.

Dixon, N., Jones, D.R., 2005. Engineering properties of municipal solid waste. Geotextile and Geomembranes 23, 205-233.

Dixon, N., Jones, D. R. V. & Fowmes, G. J., 2006. Interface shear strength variability and its use in reliability-based landfill stability analysis. Geosynthetics International, 13, No. 1, 1–14.

Eid, H.T., Stark, T.D., Evans, W.D., Sherry, P.E., 2000. Municipal Solid Waste Slope Failure. I: Waste and Foundation Soil Properties. J. of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 126, No. 5, May, pp. 397-407.

Eijkelkamp, 2013. http://en.eijkelkamp.com/ Site accessed on 06/07/2013.

Ellard,H., Ingrams,J., Hewitt,P., Arber,N. 2005. The use of glass cullet as leachate drainage in landfill engineering. Sardinia 2005, Tenth International Waste Management and Landfill Symposium.

Environmental Permitting (England and Wales) Regulations 2010 no 675.

EN ISO 11058:1999 – Geotextiles and geotextile-related products – Determination of water permeability characteristics normal to the plane, without load.

EN ISO 12958:1999 – Geotextiles and geotextile-related products – Determination of water flow capacity in their plane.

Eurocode7: Geotechnical design. Part 1: General rules. BS EN 1997-1:2004

Fassett, J.B., Leonardo, G.A., Repetto, P.C., 1994. Geotechnical properties of municipal solid waste and their use in landfill design. Waste Tech'94, Landfill Technology Technical Proceedings, Charlesto, SC (USA), January 13-14.

Fenn, D.G., Hanley, K.I. & Degeare, T.V. (1975) Use of the Water Balance Method for Predicting Leachate Generation from Solid Waste Disposal Sites. U.S. Environmental Protection Agency, EPA 530/SW-168, pp. 1–10.

Filz, G.M., Esterhuizen, J.J.B., Duncan, J.M., 2001. Progressive failure of lined waste impoundments. Journal of Geotechnical and Geoenvironmental Engineering. October 2001, pp. 841-848

Fowmes, G.J., Dixon, N. and Jones, D.R.V., 2007. Landfill stability and integrity: the UK design approach. ICE J. of Waste and Resource Management. May 2007, WR2, 51–61.

Fowmes, G.J, 2007. Analysis of Steep Sided Landfill Lining Systems. Published EngD thesis, Loughborough University. November 2007.

Fowmes, G.J., Dixon, N., Jones, D.R.V., 2008. Validation of a numerical modelling technique for multilayered geosynthetic landfill lining systems. Geotextiles and Geomembranes 26 (2008) 109–121.

Fredlund, D. G., and Anqing Xing. 1994. Equations for the soil-water characteristic curve. Canadian Geotechnical Journal. Vol. 31, pp: 521-532.

GEOSLOPE International 2007. Vadose/W User's Guide.

Genuchten van, M.Th., 1980. A closed-form equation for predicting the hydraulic conductivity of unsaturated soils. Soil Science Society of America Journal, Vol. 44, pp:892-898.

Ghanbarian-Alavijeh, B., Liaghat, A., Huang, G.H. and van Genuchten, M. Th. 2010. Estimation of the van Genuchten soil water retention properties from soil textural data. *Pedosphere*. **20**(4): 456–465.

Giroud, J. P., Gross, B. A., Darasse, J., 1992. Flow in leachate collection layers, steady-state condition. GeoSyntec Consultants Rep.

Gotteland, P., Gourc, J.P., Alboura, A., Thomas, S., 2002. On site determination of geomechanical characteristic of waste. Proceedings GeoEng 2000, The Institute of Engineers, Australia.

Gourc, J.P., Berroir G., Stock R., Begassa P., 1997. Assessment of lining systems behaviour on slopes, Proceedings of Sardinia 1997, Sixth International Landfill Symposium , Cagliari, Italy.

Gourc, J.P., Camp, S., Viswanadham, B.V.S., Rajesh, S., 2010. Deformation behaviour of clay cap barriers of hazardous waste containment systems: Full-scale and centrifuge test. Geotextiles and Geomembranes Volume 28, Issue 3, pp. 281-291 IS Kyushu 2007 Special Issue on New Horizons in Earth Reinforcement, 5th International Symposium on Earth Reinforcement.

Head, K. H., 1982. Manual of soil laboratory testing, Vol 2, Pentech Press, ISBN 0-7273-1305-3

Henken-Mellies, U. & Gartung, E., 2004. Long-term observation of alternative landfill capping systems – field tests on a landfill in Bavaria. Land Contamination and Reclamation, 12 (1), 2004.

Iryo, T., Rowe, R.K., 2005. Hydraulic behaviour of soil-geocomposite layers in slopes. Geosynthetics International, 12, No. 3, 145-155.

Itasca Consulting Group Inc., 2002. FLAC version 4.00 user's guide, Second revision.

Jones, D R V, Shercliff, D A, & Dixon, N, 2000. Difficulties associated with the specification of protection geotextiles using only unit weight, Proc 2nd European Geosynthetics Conf , October 2000, Bologna.

Jones, D.R.V., Dixon, N., 2003. Dixon, N., Jones, D. R., 2003. Stability of Landfill Lining Systems: Report no.1 Guidance. Environment Agency Research and Development Technical Report P1-385/TR2.

Jones, D.R.V., Dixon, N., 2005. Landfill lining stability and integrity: the role of waste settlement. Geotextiles and Geomembranes 23, pp. 27-53.

Koerner, R. M., 2005. Designing with Geosynthetics, 5th Edition. Prentice Hall, Englewood Cliffs, NJ, USA, 816 pp.

Koerner, R., M., Daniel D., E., 1997. Final Covers for Solid Waste Landfills and Abandoned Dumps. American Society of Civil Engineers 1997.

Koerner, R.M., Soong, T-Y., 2000. Stability Assessment of 10 Large Landfill Failures Geodenver 2000 ASCE SPECIA vol 103, pp. 1-38.

Khoury, C.N., Miller, G.A., Hatami, K., 2011. Unsaturated soil–geotextile interface behaviour. Geotextiles and Geomembranes, Volume 29, Issue 1, February 2011, Pages 17–28.

Langer, U., 2006. Shear and Compression Behaviour of Undegraded Municipal Solid Waste. Unpublished PhD thesis, Loughborough University. July 2006.

Look, B.G., 2007. Handbook of Geotechnical Investigation and Design Tables. Taylor & Francis 2007. Print ISBN: 978-0-415-43038-8, eBook ISBN: 978-0-203-94660-2

Machado, S.L., Carvalho, M.F., and Vilaro, M., 2002. Constitutive model for municipal solid waste. Journal of Geotechnical and Geoenvironmental Engineering, 129. (4), pp. 372-378.

McCartney, J.S., Kuhn, J.A., and Zornberg J.G., 2005. Geosynthetic drainage layers in contact with unsaturated soils. 16th ISSMGE Conference. 12-16 September 2005. Osaka, Japan.

McEnroe, B. M., 1993. Maximum saturated depth over landfill liner. J. Environ. Eng., 1993, 119(2), 262–270.

MetOffice, 2012. http://www.metoffice.gov.uk/climate/uk/2012/winter.html Accessed on 10/11/2012.

Mining Department NCB, 1982. Technical Management of Water in the Coal Mining Industry. Published by the National Coal Board and printed by The Southern Publishing Co. (Westminster Press Limited).

Muhsiung, C., 2005. Three-dimensional stability analysis of the Kettleman Hills landfill slope failure based on observed sliding-block mechanism. Computers and Geotechnics 32, pp. 587–599.

Najser, J., Masin, D., Bohac, J., 2010. Numerical modelling of lumpy clay landfill. International Journal for Numerical and Analytical Methods in Geomechanics. Published online in Wiley Online Library (wileyonlinelibrary.com). DOI: 10.1002/nag.990

Nahlawi, H., A. Bouazza and J. Kodikara., 2007. Characterisation of geotextiles water retention using a modified capillary pressure cell. Geotextiles Geomembranes 25:186–193

Nakamura, J., Sakanishi, M., Kitazawa, A., Kumagai, K., Fujihashi, K., Ogasawara, T., 2006. Deformation measurement technology of geomembrane at controlled landfills using optical fiber sensors. Geosynthetics, Volumes 1-4. Proceedings of the 8th International Conference on Geosynthetics (8ICG), Yokohama, Japan, 18-22 September, 2006, pp. 203-206.

Nancey, A., Lacina, B., Henderson, J., 2007. Geotextile and optic fibres: feedback after four years of use in soil. Geosynthetics 2007 GRI-20 proceedings, January 16-17, Washington D.C.

Palmeira, E.M. & Silva, A.R.L., 2007. A study in the behaviour of alternative drainage systems in landfills. Eleventh International Waste Management and Landfill Symposium, 1 - 5 October 2007, S. Margherita di Pula, Cagliari, Italy.

Rowe, R.K., Hoor, A. 2009. Predicted temperatures and service lives of secondary geomembrane landfill liners. *Geosynthetics International,* 16, No. 2, 71-82.

Rowe, R.K., Islam, M.Z., Hsuan, Y.G., 2008. Leachate chemical composition effects on OIT depletion in an HDPE geomembrane. *Geosynthetics International,* 15 No. 2, 136-151.

Rowe, R.K., Sangam, H.P., 2002. Effects of exposure conditions on the depletion of antioxidants from high-density polyethylene (HDPE) geomembranes. *Canadian Geotechnical Journal* 39, 1221-1230.

R&D Technical Reports P1-385/TR1&TR2, see Jones and Dixon (2003) and Dixon and Jones (2003).

Richardson, G. N., Giroud, J.P., Zhao, A., 2002. Lateral drainage design update. Geotechnical Fabrics Report 01/02.2002.

Qian, X., Gray, D. H.; & Koerner R. M., 2004, Estimation of Maximum Liquid Head over Landfill Barriers. Journal of Geotechnical and Geoenvironmental Engineering. May 2004.

Sia, A.H.I., 2007. Landfill Lining Engineering Designs: A probabilistic approach. Unpublished PhD thesis, Loughborough University. November 2007.

Seed, R.B., Mitchell, J.K., Seed H.B., 1990. Kettleman Hills Waste Landfill Slope Failure. II: Stability analysis, Journal of Geotechnical Engineering, ASCE 116 (4), pp. 669–691.

Siemens, G, Bathurst, R.J., 2010. Numerical parametric investigation of infiltration in one-dimensional sand–geotextile columns. Geotextiles and Geomembranes, Volume 28, pp. 460-474.

Stark, T. D. & Newman, E. J., 2010. Design of a landfill final cover system. Geosynthetics International 17, No. 3, 124–131. [doi: 10.1680/gein.2010.17.3.124]

Strunk, R.L, Dobchuk, B.S., Nieminen, G., Barbour S.L., O'Kane, M., 2009. Performance Monitoring of Soil Cover Field Trials at the Regina Municipal Landfill. Web

site accessed on the 15.02.2012: http://www.okaneconsultants.com/pdf_publications/gmm/ Strunk%20 _Dobchuk_%20Regina_landfill_paper.pdf

Stoltz, G, Gourc, J.P., Oxarango, L, 2010. Characterisation of the physico-mechanical parameters of MSW. Waste Management 30 (2010), pp. 1439–1449.

Teragéos, 2012. Teragéos la solution géosynthétique. Web site accessed on the 15.11.2012: http://www.terageos.com/edito.php

Tresselt, K., Miehlich, G., Groengroeft, A., Melchior, S., Berger,K., Harms,C., 1998. Harbour sludge as barrier material in landfill cover systems. Water Science and Technology Vol 37 No 6-7 pp 307–313 © IWA Publishing 1998.

U.S. Environmental Protection Agency (U.S. EPA), 1980. Landfill and surface impoundment performance evaluation. Rep. No. EPA/536/SW-869-C, Cincinnati.

U.S. Environmental Protection Agency (U.S. EPA), 1983. Landfill and surface impoundment performance evaluation. Rep. No. SW-869, Revised Ed., Office of Solid Waste and Emergency Response, Washington, D.C.

U.S. Environmental Protection Agency (U.S. EPA), 1989. Requirements for hazardous waste landfill design, construction, and closure.Rep. No. EPA/625/4-89/022, Center for Environmental Research Information, Office of Research and Development, Cincinnati.

USGS 2012, U.S. Geological Survey. http://ga.water.usgs.gov/edu/watercycleprecipitation.html Accessed on 11/10/2012

Villard, P., Gourc, J.P., Feki, N., 1999. Analysis of geosynthetic lining system (GLS) undergoing large deformations. Geotextiles and Geomembranes, Volume 17, Issue 1 February 1999, pp. 17-32.

Wilson, G.W., Williams, D.J., Rykaart, E.M., 2003. The integrity of cover systems – an update. Cairns, QLD, 12-18 July 2003. 6th ICARD – International Conference on Acid Rock Drainage.

WunderGround 2012a. Woughton Park, Milton Keynes, BUCKINGHAMSHIRE http://www.wunderground.com/weatherstation/WXDailyHistory.asp?ID=IBUCKING38&month=6&day=11&year=2012 Accessed on 01/10/2012.

WunderGround 2012b. Far Bletchley, Bletchley, England

http://www.wunderground.com/weatherstation/WXDailyHistory.asp?ID=IMK37RQM1&month=6&day=11&year=2012 Accessed on 01/10/2012.

Zamara, K.A., Dixon, N., Jones D.R.V., 2011. Design of drainage systems in geosynthetic landfill caps. Sardinia 2011, Thirteenth International Waste Management and Landfill Symposium.

Zamara, K.A., Dixon, N., Jones, D.R.V., Fowmes, G., 2012. Monitoring of a landfill side slope lining system: Instrumentation selection, installation and performance. Geotextiles and Geomembranes. IN PRESS.

Zamara, K.A., Dixon, N., Fowmes, G., Jones, D.R.V., Zhang, B., Sharley, M., 2012b. Instrumentation measuring hydraulic performance of a landfill cap: Case study. The 5th European Geosynthetics Congress.

Zhang, B., 2007. Constitutive modelling of municipal solid waste. Ph.D. thesis, Civil and Building Engineering, Loughborough University, United Kingdom.

Zhang P, Wu Z.C., 2005**.** Municipal sludge as landfill barrier material. J Environ Sci (China). 2005; 17(3):474-7.

APPENDIX A   PAPER 1

Full Reference

Dixon, N., Zamara, K.A., Jones, D.R.V., Fowmes, G., 2012. **Waste/Lining System Interaction: Implications for Landfill Design and Long-term Performance**. Geotechnical Engineering Journal of the SEAGS & AGSSEA Vol. 43 No.3 September 2012 ISSN 0046-5828.

PAPER TYPE – JOURNAL PAPER

Waste/Lining System Interaction: Implications for

Landfill Design and Performance

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ABSTRACT: Despite the relative maturity of landfill design practice, world-wide there are still significant numbers of large scale failures of waste bodies, often incorporating the lining system. In addition, there is growing evidence that post waste placement deformations in the lining system are leading to loss of function (i.e. discontinuous drainage layers, loss on protection and leaking liners). Best practice has established that both stability and integrity of the lining system must be assessed during the design process, and specifically that interaction between the waste body and lining system should be considered both in the short-term (i.e. during construction) and long-term (i.e. following waste degradation). The paper introduces available analysis approaches, reviews knowledge of waste behaviour required for such analyses and provides guidance on the mechanisms to consider. The need for field monitoring to validate numerical models is established as is the need for extensive measurements of waste mechanics properties linked to a standard classification system to aid comparison and use. The benefits of using probability of failure analysis to incorporate material and test variability in design are highlighted.

1. INTRODUCTION

Engineering design of containment systems for municipal solid waste (MSW) is now a mature discipline. The large majority of landfill facilities world-wide use a combination of natural and geosynthetic materials to form lining systems that minimise leakage of contaminants into the environment. Lining systems have two primary functions, to act as a barrier to liquid and gas and to facilitate their collection and removal. A typical lining system is shown in Figure 1. It has been demonstrated that combining natural and geosynthetic materials results in improved performance of the composite system (e.g. Rowe 2005). However, construction of these planar systems introduces the potential for uncontrolled deformation through slippage at material interfaces, especially on cell lining side slopes and capping slopes. These stability failures can be defined as large uncontrolled deformations of the system and are ultimate limit states.
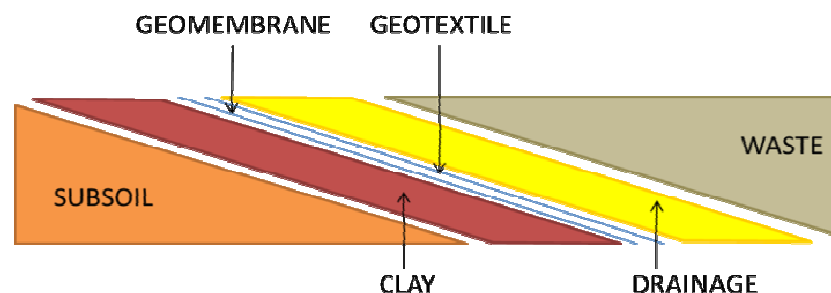
Figure 1Typical lining system components and configuration

Well established analysis methods are available for use by designers to assess stability (e.g. Koerner & Song 1998) and it would appear that designing lining systems to ensure stability throughout the life of the facility is an established and routine process. Unfortunately this statement is proven to be false by the relatively large number of stability failures that have taken place since design procedures were established (e.g. Seed *et al.* 1988, Brink *et al.* 1999, Koerner & Song 2000, Eid *et al.* 2000, Jones & Dixon 2003). Failures have involved a wide range of lining system configurations and they have occurred world-wide including in countries with well-established design procedures. While some of the failures can be

attributed to the use of inadequate materials and/or inappropriate construction, in a significant number of cases the cause of failure can be attributed to the design not considering the critical failure mechanism. A key limitation of standard design methods used for assessing lining stability is that they do not consider the influence of the waste body. The placement of waste against side slope lining systems and the subsequent waste deformations as it degrades can stress the lining elements and lead to failure. Many interfaces between geosynthetics/geosynthetics and soil/geosynthetics are strain softening (i.e. the shear strength of the interface reduces with displacement after a peak value is achieved). A key concern is that post peak shear strengths (i.e. reduced strengths) will be mobilised at interfaces between lining elements in response to waste settlement and this can result in uncontrolled deformation of the lining system leading to pollution of the environment.

A number of analytical and numerical modelling techniques have been proposed by practitioners and researchers to assess this influence of waste deformations on lining system stability (Section 3) and several studies have attempted to answer the questions as to whether peak, residual, or somewhere in between interface shear strengths should be used in side slope stability assessment. Analyses considering the waste body are challenging to carry out because they require information on the engineering properties of the waste and the sequence of waste placement. The high variability of waste mechanical properties both between landfills and within each facility makes the selection of appropriate material parameters for use in design a very difficult task. However, if interaction between the waste and lining is not considered in design then often adequate performance of the containment facility cannot be assured.

In addition to concerns regarding stability of the landfill construction, it is also essential that the integrity and hence long-term performance of the lining system is assured. Integrity failure can be defined as small scale deformations of elements of the lining system that lead to loss of function (i.e. increased permeability of a barrier element or discontinuity of

a protection layer) and are serviceability limit states. Integrity failure mechanisms are linked to waste deformations and hence they are difficult to detect and repair as defects are buried beneath the waste. However there is substantial evidence that these types of failure occur in many landfills, for example gas leaks and cases of rapid increases in leachate levels in sub water table facilities. A design framework for lining systems that considers both stability and integrity has been developed by Dixon & Jones (2003) and extended by Fowmes *et al.* (2007). This framework was developed for the Environment Agency (England and Wales) and is incorporated in their permitting process for landfill facilities. Designers are required to demonstrate that they have considered all potential failure mechanisms for stability and integrity failure both prior to and following waste placement. Figure 2 summarises the lining system design considerations for stability and integrity of the six landfill elements: subgrade, basal lining system, shallow side slope lining system, steep side slope lining system, waste slopes and capping lining system.
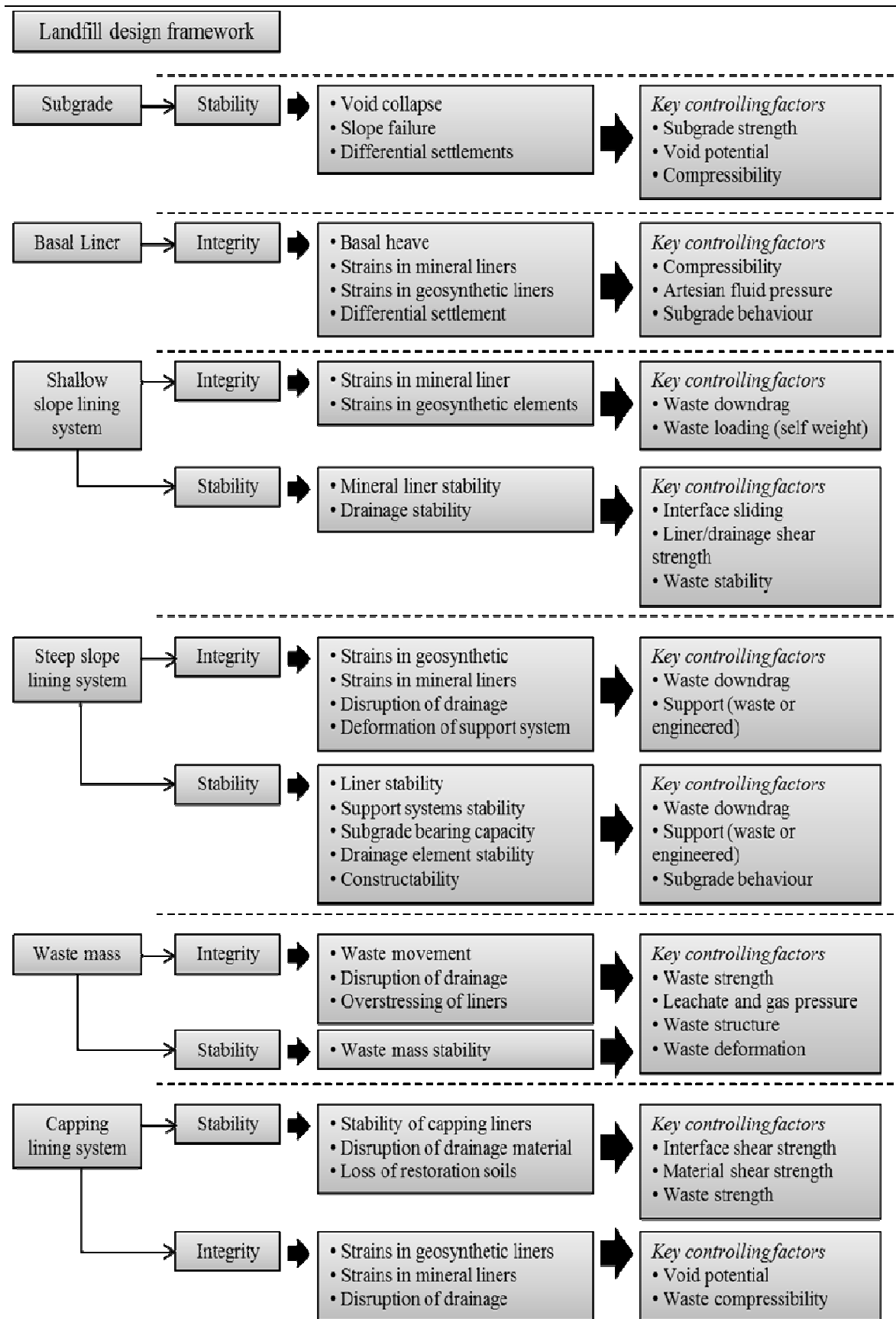
Figure 2 Landfill stability and integrity design framework (after Fowmes *et al.* 2007)

As an example, Figure 3 illustrates stability and integrity failure mechanisms for a shallow side slope lining system before and after waste placement.

There is often poor use of terminology surrounding steep sided lining systems, and there is often an assumption that steep sided landfill lining systems are near vertical. Jones & Dixon (2003) suggest slope angles in excess of 30° are "steep". An alternative approach is to consider the stability of internal components of the lining system and the following definition can be used: A steep slope lining system is a side slope lining system placed at an angle, at, or greater than the limiting value at which the geological barrier, drainage layer, or artificial sealing liners are naturally stable without application of additional loads from the waste mass, anchorage or engineered support structures.

This paper summarises the current state of understanding of waste/lining system interaction, it considers waste types and behaviour, describes the use of numerical modelling methods to assess interaction between the waste and lining elements and highlights the implications of uncertainty and variability for design.

b)
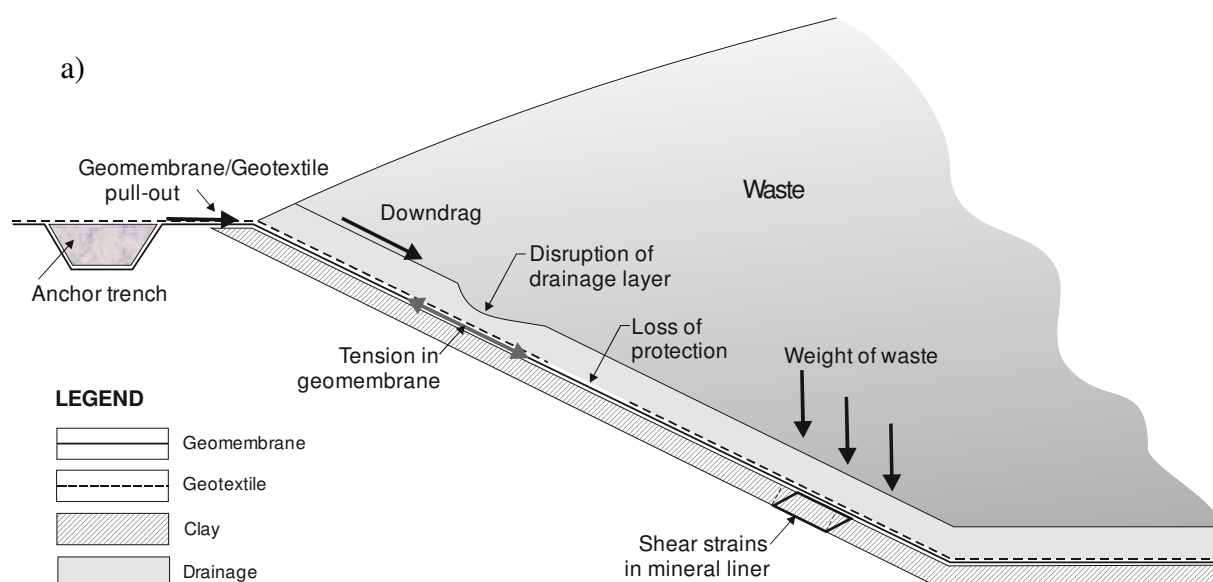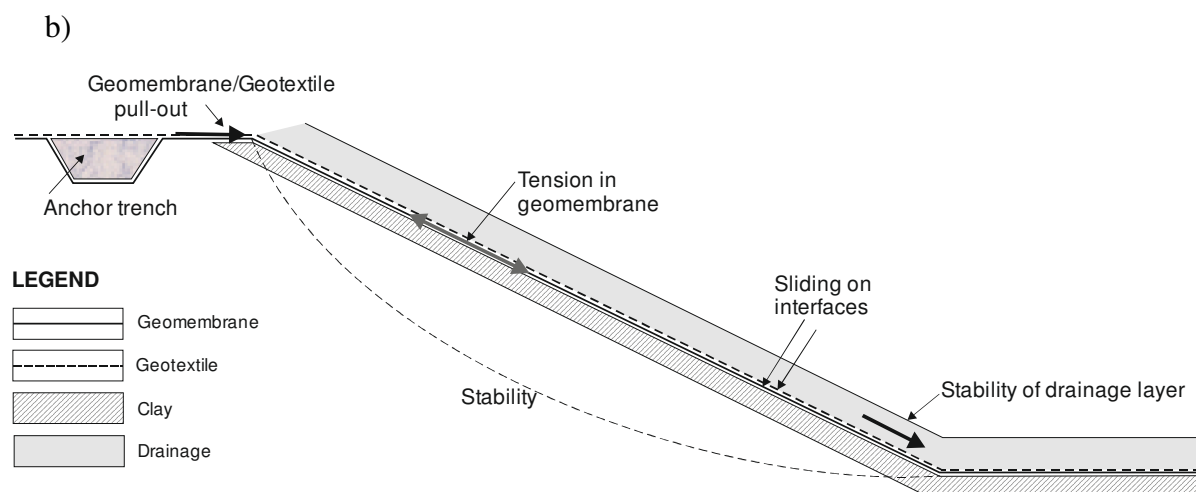
Figure 3 Potential failure mechanisms for a shallow side slope lining system a) unconfined, b) confined by waste (after Fowmes *et al.* 2007)

2. WASTE TYPES AND ENGINEERING BEHAVIOUR

2.1 Classification and engineering properties

Waste is the largest structural element of any landfill. It loads the basal and side slope lining systems and supports the capping system and gas/leachate collection infrastructure. Mechanical behaviour of the waste body controls many aspects of the lining system design and performance including stability and integrity of the geosynthetics and mineral lining components. Waste is a highly heterogeneous material and its mechanical properties change with time due to physical changes in components that result from degradation processes. The significant challenges posed by this spatial and temporal variability mean that it is not possible to fully characterise the engineering properties of a waste body. However, it is important that the basic behaviour of broad categories of waste is understood and that likely ranges of the controlling engineering properties are known for a specific site. Table 1 lists properties required to perform analyses of the stability and integrity failure modes summarised in Figure 2.

Design case	Unit weight	Vertical compressibility	Shear strength	Lateral stiffness	Horizontal in-situ stress	Hydraulic conductivity
Subgrade stability	X		X		X	
Subgrade integrity	X		X	X	X	
Waste slope stability	X	X	X			X
Shallow slope liner stability	X		X		X	X
Shallow slope liner integrity	X	X	X	X	X	
Steep slope liner stability	X		X		X	X
Steep slope liner integrity	X	X	X	X	X	
Cover system integrity	X	X	X			
Drainage system integrity	X				X	
Leachate/gas well integrity	X	X	X	X	X	X

Table 1 Municipal solid waste engineering properties required for stability and integrity lining design

(after Dixon & Jones 2005)

MSW is a mixture of wastes that are primarily of residential and commercial origin. Typically, MSW consists of food and garden wastes, paper products, plastics, rubber, textiles, wood, ashes, and soils (both waste products and material used as cover material). A wide range of particle sizes is encountered ranging from soil particles to large objects such as demolition waste (reinforced concrete and masonry). The proportion of these materials will vary from one site to another and also within a site. Life style changes, legislation, seasonal factors, pre-treatment and recycling activities result in a changing waste stream over time. Examples are increasing plastic and decreasing ash content over the past few decades in developed countries. In addition, over the past decade member states of the European Union have effected a reduction in biodegradable waste in landfills through the introduction of Biodegradable Municipal Waste diversification targets defined in the Council of European

Community (1999). It should be noted that the composition of MSW varies from region to region and country to country. For example, developing countries often have waste streams that contain more biodegradable material and fewer plastics, and countries such as Germany with a well-developed re-cycling and pre-treatment policy (e.g. the use of mechanical and biological pre-treated waste), have wastes with less biodegradable content and a more uniform and consistent grading. These variations produce fundamental and significant differences in waste engineering behaviour and they must be taken into consideration when using results from tests on waste reported in the literature.

There is a growing body of literature on the measurement of engineering properties of MSW and numerous summaries of the state-of-the-art have been produced over the past 20+ years (e.g. Landva & Clark 1990, Fasset *et al.* 1994, Manassero *et al.* 1996, Eid *et al.* 2000, Kavazanjian 2001, Qian *et al.* 2002, Dixon & Jones 2005, Reddy *et al.* 2010 and Stoltz *et al.* 2010). These are a valuable resource but have limitations as they do not use an agreed waste classification system or test standards to present and group information on mechanical properties. This makes it difficult to compare and interpret results from the different studies and to apply findings to other sites. Dixon & Langer (2006) attempt to address this issue by proposing a MSW classification system for the evaluation of mechanical properties, however, such a system can only be effective if it is used by multiple researchers and practitioners so as to build a data base of measured waste behaviour linked to the classification. A milestone event in the development of a unified framework for waste mechanics is the International Symposium on Waste Mechanics held in New Orleans, March 2008. The objectives of this symposium were to: develop consensus on procedures and guidelines for waste characterisation, field testing and laboratory testing of MSW; summarize the state of knowledge on waste properties for use in research and engineering practice; and identify research needs in waste mechanics. The proceedings of the symposium have been published

as an ASCE Geotechnical Special Publication (Zekkos 2011). This includes a chapter on waste characterisation (Dixon *et al.* 2011) which presents an agreed system, based on Dixon & Langer (2006), with a recommendation that it be used for future studies of waste mechanics thus fulfilling the requirement for a universal classification framework. Zekkos (2011) includes specific sections covering the key MSW engineering properties listed in Table 1 and their measurement. It also considers dynamic properties of MSW that are of relevance to landfill design in seismically active regions of the world. It is important to note that studies of MSW mechanics have demonstrated that although waste is heterogeneous it has properties that vary in a consistent and predictable way (e.g. with respect to stress state and method of placement).

2.2 Waste body deformations

Stability and integrity of side slope lining systems are primarily controlled by the magnitude and distribution of adjacent waste deformations during the life of the structure. Waste deformation can be separated into two components: primary compression which is a short-term response to load from the overlying waste, and secondary longer-term response to degradation and creep. Historically, studies of waste deformation have been restricted to measurement of surface settlements that are used to improve efficiency of site management and to predict final landfill capping profiles. These measurements are of limited use for assessing waste deformations adjacent to side slope lining systems as they rarely provide information on primary compression that occurs during the filling process. This is because filing of waste often takes place to a predetermined reduced level and therefore compression of the underlying waste during placement is masked by overfilling to meet the fill level requirement. This means that use of surface settlement measurements grossly underestimates the actual waste deformation depth profile. However, surface settlement measurements

provide useful information on secondary deformation from degradation and creep, although again they do not provide information on depth distributions.

For these reasons, in order to obtain information required to assess waste/lining system interaction it is necessary to install instruments within the waste body to measure settlement at depth intervals during waste filling and degradation. The few studies reported in the literature that have presented such measurements confirm that large waste deformations occur. For example, Dixon *et al.* (2004) report measured immediate settlements of 300 to 700 mm of a 3 metre thick layer of MSW when a further 3 metre layer of waste was placed above. These, when related to measurements of the stress changes that caused the deformations, can be used to derive stiffness values for use in numerical models (Dixon *et al.* 2004). On-going, long-term projects to measure landfill post-closure surface settlements are providing valuable information on degradation induced waste deformations, but longer time-series of measurements are required before conclusions can be drawn on the magnitude and distribution of final waste deformations related to waste classification and site specific construction and operation approaches.

A further complication is that waste adjacent to side slopes may not behave the same as the maximum thickness of waste above the base where to date the majority of monitoring has focussed. This is a results of the slope geometry, which could include changes of slope, and differences in waste materials selected and method of placement used (i.e. in attempts to protect the lining system from damage during waste placement). Gourc *et al.* (1998) report a study to measure the sub-surface waste deformations adjacent to a side slope and this has produced useful information on the magnitude and distribution of waste deformations for the specific waste, filling sequence and slope geometry studied. A comparable study is currently underway at Bletchley Landfill Site near Milton Keynes, UK, where a series of landfill settlement monitoring instruments has been installed. The instrumentation is installed at 6m

vertical increments in two locations, one at the toe of the side slope and the other near to the middle of the landfill cell. The waste depth is approximately 40m. At each location a vibrating wire pressure cell, thermistor and hydrostatic settlement cell are installed and connected via HDPE pipes to a monitoring cabin at the crest of the side slope. The instrumentation has allowed analysis of the settlement profile within the waste mass during the filling process which was completed in late 2011. Monitoring will continue during the site aftercare period. The measurements and interpretation will be reported following completion of the detailed analysis, including influence of waste type and filling sequence.

2.3 Modelling waste body deformations

It is well established that landfill deformation is the product of a combination of phenomena, including load and biodegradation-related processes, and there are numerous useful reviews of models that can be used to calculate surface waste settlement (e.g. McDougall 2011). However, to date, these models are not able to routinely consider waste deformations in two dimensions, and this function is required to investigate waste deformations adjacent to side slopes (i.e. to analyse a cross section through the waste and lining system). A promising approach is the HBM model that combines three models each describing the hydraulic, biodegradation and mechanical behaviour of landfilled waste. A summary of the HBM model is provided by McDougall (2011) and further details of the constitutive formulation are given by McDougall (2007). This type of model gives the promise of being able to investigate time dependant waste deformations in two dimensions from initial filling to completion of degradation, and hence to directly link waste behaviour with lining system performance. However, to date there is still a need for further model development and validation, and specifically to measure the many MSW material parameters that are needed to run the model. Currently, waste models used to investigate lining system performance have concentrated on

short-term construction and waste placement activities by using standard soil mechanics constitutive models to represent the waste (Section 3), but it is only a matter of time before a fully coupled waste (i.e. such as HBM) and lining system model is available.

3. NUMERICAL ANALYSIS OF WASTE/LINING SYSTEM INTERACTION

3.1 Limit equilibrium approach

Waste/lining system interaction cannot be fully considered using limit equilibrium analysis as it is unable to provide information on whether strain softening will occur on a specific interface. Limit equilibrium can be used to analyse overall stability of a system but the designer must select interface shear strength parameters (i.e. whether peak, residual or a factored strength should be used). Selection of the shear strength parameters requires assessment of whether waste settlement (or any other mechanism, see Section 5.1) will generate post peak strengths. There is evidence that generation of post peak strengths by a progressive failure mechanism can lead to catastrophic large scale movements of a waste mass against the lining system (e.g. the Kettleman Hills failure – Seed *et al.* 1988). However, in the majority of instances it is the liner integrity that could be compromised if displacements occur along an interface following waste placement. It is difficult to monitor lining systems once waste has been placed, and this is seldom done, but there is evidence of integrity type failures caused by waste settlement next to the lining system (e.g. Fowmes *et al.* 2006). For landfill side slopes the shear strength mobilised at a liner interface through waste deformation will vary along the interface and this cannot be modelled by a simple limiting equilibrium approach (Long *et al.* 1995).

3.2 Numerical modelling approach

Numerical modelling techniques such as finite element and finite difference formulations can be used to assess the shear stresses mobilised at strain softening geosynthetic interfaces for a range of waste properties and landfill geometries. A summary of literature on modelling waste/barrier interaction was provided by Jones & Dixon (2005) and this is extended below to demonstrate the advances in analysis methods that have been made in recent years and the guidance developed from the studies. Filz *et al.* (2001) demonstrated that numerical modelling could be used to assess progressive failure of a geomembrane/clay interface in response to staged placement of MSW against a landfill side slope. They showed that average mobilised shear strengths along the interface were close to residual for the configurations assessed, and that limit equilibrium analyses carried out using peak strength values significantly overestimated the degree of stability. They concluded that strain softening interfaces must be considered in the design of landfill lining systems. In addition they provided guidance on the selection of appropriate shear strength parameters for use in limit equilibrium analysis of stability. The work by Filz *et al.* (2001) is important because it demonstrates the appropriateness of the numerical approach. However, the guidance they produced is restricted to a geomembrane/clay interface and for relatively short-term construction related behaviour. A further limitation of this work is that deformations on the interface are not reported and therefore issues of lining system integrity are not addressed.

Long-term degradation controlled waste settlements play an important role in lining system behaviour. Meissner and Abel (2000) present results for numerical modelling of tensile stresses in a geomembrane basal and side slope liner resulting from waste degradation. A numerical model was presented that allows time dependent waste settlements to be considered. Displacements at a geotextile/geomembrane interface are predicted. This type of information could allow issues of lining integrity to be considered. Unfortunately, the model

employed to represent the interface was not strain softening and this invalidates many of the results obtained, although the approach of using numerical modelling techniques to assess long-term waste degradation effects on the lining system is valid.

Jones and Dixon (2005) used both limit equilibrium and numerical analysis techniques for assessing stability and integrity of a lining system containing a strain softening interface. The condition considered is for waste/liner interaction resulting from long-term, degradation controlled, waste settlement adjacent to a landfill side slope. A strain softening geotextile/geomembrane interface is introduced as the controlling interface in the lining system. The influence of waste properties, slope angle and waste height are assessed. Comparison of the results from the limit equilibrium and numerical analyses has shown that simple limit equilibrium analysis does not satisfactorily assess the local stability of geosynthetics on a landfill side slope. Even with the use of mobilised shear strengths for the geosynthetic interfaces obtained from the numerical analyses, the limit equilibrium analysis does not give a reliable indication of the stability of the slope. Limit equilibrium methods can only be used to predict instability along a continuous failure plane, and therefore cannot be used to assess integrity failure of a geosynthetic lining systems caused by localized displacement along interfaces. Interface shear displacements in the order of metres were obtained in numerical analyses while the limit equilibrium assessment showed the lining system to be stable. Figure 4 shows displacement distributions along the base and side slope for a range of angles in response to placement of 30 metres of waste. For the steeper slopes it can be seen that displacements of metres take place on the side slope even though the waste mass is still globally stable.
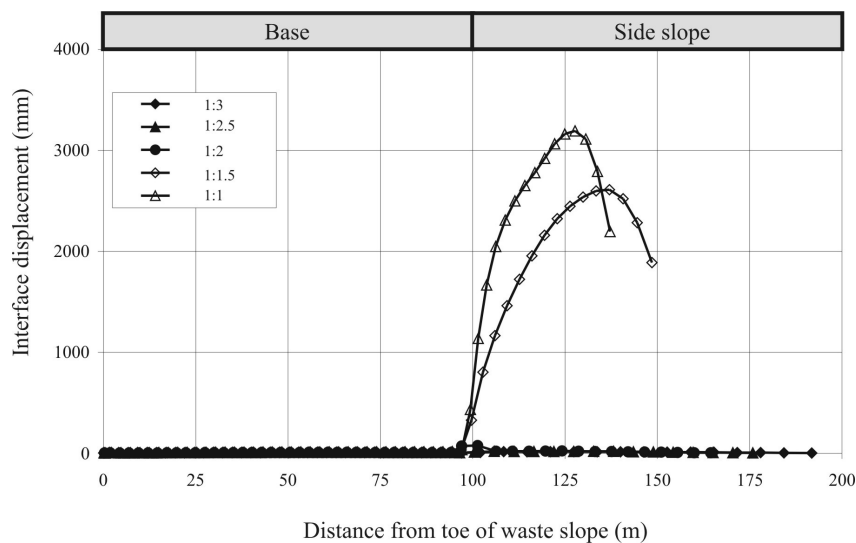
Figure 4 Results from numerical modelling of a range of side slope angles each 30 metre high, which show large relative displacements at a geotextile vs. geomembrane interface caused by long-term waste settlement (after Jones & Dixon 2005).

Villard *et al.* (1999) presented results from finite element modelling of a side slope lining system comprising geosynthetic and mineral components in response to placement of a gravel drainage layer. Their model included strain softening interface behaviour and material stiffness. This enabled consideration of tensile stresses in geosynthetic components and hence assessment of the integrity of lining components. They concluded that the comparison between the numerical model results and full-scale experimental observations was generally satisfactory. However, difficulties were encountered in reproducing measured mobilised tensile forces in geosynthetics. This was believed to be due to the formation of wrinkles at the base of the slope. Unfortunately this work was not extended to investigate the influence of waste placement and subsequent waste settlement. It is not clear whether the approach developed by Villard *et al.* (1999) could be used to assess the integrity of all lining components (i.e. relative displacement and tensile stresses) in response to waste settlement. Fowmes *et al.* (2006) extended the work of Villard *et al.* (1999) and Jones & Dixon (2005) by modelling the waste body and individual lining components with strain softening interfaces

between them. This allows transfer of stresses through the lining components, calculation of tensile stresses in lining components and hence an assessment of system integrity (e.g. likely continuity of the geotextile protection layer) as reported by Fowmes *et al.* (2006).

Long *et al.* (1995) presented an approach to assess the integrity of all components in a lining system. This uses a finite difference formulation that includes non-linear mechanisms to model the shear stress/displacement behaviour of each interface and the axial load/displacement behaviour within each component. The waste body is not modelled directly but effects of waste load and settlement are considered by imposing the combination of displacement and load boundary conditions on the top layer of the lining system. While this approach provides information on the integrity of elements it is based on an assumed waste settlement profile along the side slope. The Long *et al.* (1995) approach could be coupled with those that model waste settlement behaviour in order to obtain more rigorous assessments of lining component integrity (i.e. in respect to over stressing).

4. VALIDATION OF NUMERICAL MODELS

4.1 Requirement for validation

The above numerical methods are capable of providing useful information and insights that can be used to assess the stability and/or integrity of side slope lining system components during construction and subsequent degradation, and hence settlement, of the waste body. Their use has become commonplace in the UK as part of the Stability Risk Assessment required for the permitting process, although analyses incorporating multiple strain softening interfaces are not routine as they require specialist software and a high level of expertise to run the models. However, despite their regular use there is still only limited information available that can be used to validate the numerical models. In traditional geotechnical

engineering it would be inconceivable to design and construct a geotechnical system where there are serious implications if it fails and not to monitor the structure and surround to ensure adequate performance. However, in landfill engineering this is the norm. Landfill containment systems are significant and important geotechnical structures the failure of which can have major consequences for the environment. Current designs are complex soil, geosynthetic, structural systems and novel designs are regularly used particularly for lining steep slopes. However, despite the importance and complexity of these lining systems they are seldom directly monitored during operation to prove adequate structural performance, and hence to confirm the validity of the design assumptions and methodology.

4.2 Validation of numerical models

Fowmes *et al.* (2008) describe a series of large-scale laboratory tests containing geosynthetic elements of a multi-layered lining system exposed to down-drag forces from a compressible synthetic waste material, which was designed to produce data to validate numerical analysis of the same problem. It is recorded that this approach was taken because of difficulties in gaining access to instrument and monitor an in-service lining system. The numerical results presented by Fowmes *et al.* (2008) are from initial best estimate analyses, with interface and synthetic waste properties derived from a laboratory testing programme and geosynthetic material properties supplied by manufacturers. It is reported that the observed trends of tensile stresses in the geosynthetics and relative displacements at interfaces in the laboratory model test are reproduced by the numerical models to an acceptable degree of accuracy that would be appropriate, using site specific input data, for use in commercial design. However, it is noted that although the use of numerical modelling techniques allows prediction of displacements, stresses and strains in multilayer geosynthetic lining systems with non-linear interface behaviour, the outputs are always limited by the accuracy of the input parameters,

the constitutive equations and the application of the numerical calculation technique and this must be considered by the design engineer. It is also stated that whilst it is believed that the laboratory study represented a significant step in the validation of the numerical model behaviour, full scale field instrumentation of a landfill site is still required to allow for assessment of the numerical model accuracy under in-service conditions.

Using this validated numerical model detailed in Fowmes *et al.* (2008), Fowmes *et al.* (2006) report analysis of the interaction between waste and a geomembrane based lining system for a benched steep slope lining system in a hard rock quarry. The analysis was able to replicate the observed tensile failure of the geomembrane, which occurred at the corner of the benches as a result of down-drag forces from the settling waste being transferred into the geomembrane liner element.

In response to the identified need for validation using site performance measurements, Zamara *et al.* (2010) detail on-going research to instrument a side slope lining system with the aim of monitoring structural performance of the components during and post waste placement, and using the measurements to validate a numerical model of the waste/lining system interaction. The lining system comprises a compacted clay layer overlain by a geomembrane, which is in turn overlain by a geotextile protection layer. A sand drainage layer is present above the geotextile. Figure 5 shows the lining system components and instrument layout. Instruments installed during construction of the lining system include pressure cells to measure the stress on the lining components from placement of waste, extensometers to measure strains in the geomembrane and geotextile and the relative displacement at the interfaces between clay and geomembrane, and geomembrane and geotextile. Fibre optic bragg strain gauges and demic gauges have also been used to measure strains in the geomembrane. Monitoring has been carried out during staged construction of the sand veneer drainage layer and waste placement on the 31 metre long and 11.6 metre high

slope (i.e. 21.8° slope angle). Monitoring commenced in July 2009 and is continuing. Waste filling will be completed in summer 2012. Results from the monitoring programme are presented by Zamara *et al.* (2012) for the sand veneer construction. Monitoring will be continued after cell closure during waste degradation. The monitoring to date has shown relative displacements between the geomembrane and geotextile that would mobilise post peak interface shear strengths, which is consistent with the results from numerical models of similar systems. Multiple direct shear laboratory tests have been conducted on the clay/geomembrane, geomembrane/geotextile and geotextile/sand interfaces to provide data for use in a multi-layered strain softening interface numerical model of the construction sequence. It is planned to use the detailed field measurements to fulfil the requirement to assess performance of the numerical modelling approach.
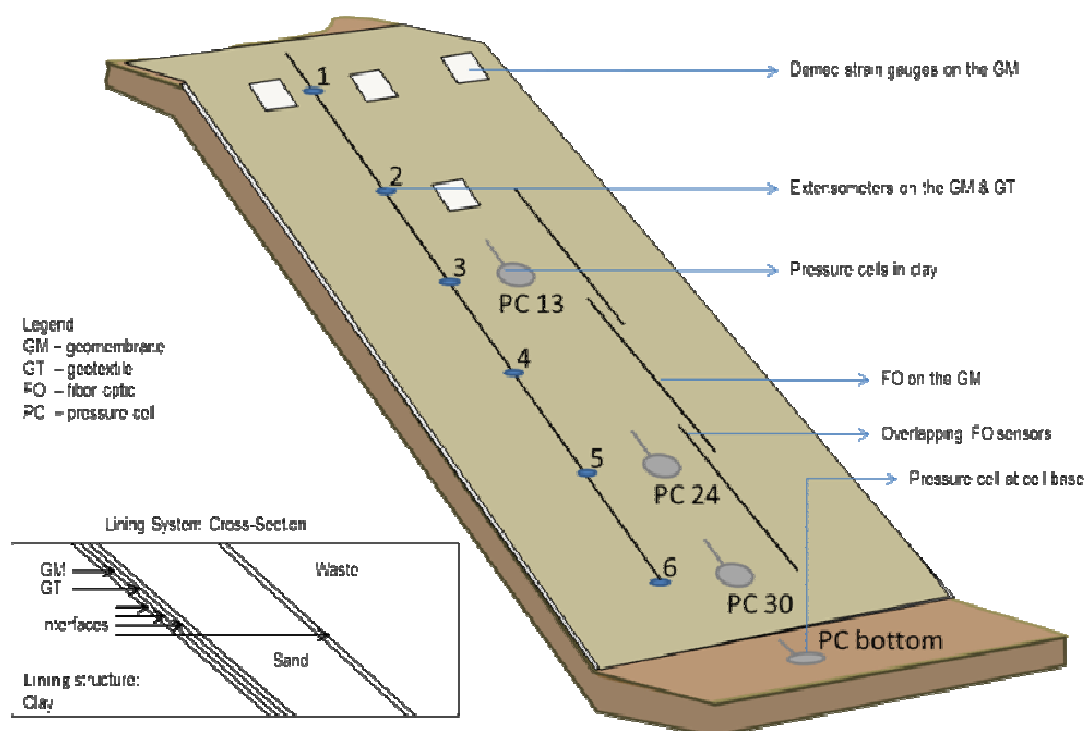

Figure 5 Details of the lining system, instrument types and locations for the field trial at Milegate Landfill, UK, which is being used to validate numerical models of waste/barrier interaction (after Zamara *et al.* 2012).

5. DESIGN GUIDANCE

5.1 Mechanisms producing interface post peak shear strengths

Field observations and the output of numerical models of waste/lining system interaction are consistent in showing that post peak interface shear strengths can be mobilised. Therefore, this produces a requirement for the designer to select shear strength parameters and factors of safety to ensure adequate performance of the landfill facility. Since the magnitude and distribution of shear strength mobilised between lining components is dependent upon magnitude of displacements at the interfaces, the design approach used must include assessment of likely relative displacements and the implications of these for both stability and integrity of the system. This paper has focused on the role of waste/lining system interaction, however there are a number of additional mechanisms associated with the construction and operation of landfill facilities that can result in relative displacements occurring at geosynthetic/ geosynthetic and geosynthetic/soil interfaces and hence in mobilization of post-peak shear strengths. Those related to construction activities are:

- Dragging geosynthetic materials over one another to position correctly.

- Construction plant loads (including acceleration and braking forces) from trafficking interfaces with inadequate cover. Particular attention should be given to placement of veneer soil layers on slopes (Koerner and Daniel 1997, Jones *et al.* 2000).

- Compaction of fine grained soils above geosynthetic layers. This should be particularly discouraged on slopes.

- Improper storage and handling of geosynthetics leading to loss of internal strength (e.g. breaking of glued connections in geocomposites).

Activities associated with landfill operations are:

- Compaction of waste against side slopes (i.e. similar issues as placement of veneer soil layers)

- Differential settlement of the sub-grade beneath a basal liner or of waste beneath a cap.

The designer must consider all possible mechanism that could potentially result in the mobilisation of post-peak, and even residual, strength conditions for the interfaces under consideration. This assessment should be used to justify the selection of strength parameters and factors of safety used in the design.

5.2 Design assessment and parameter selection

If a mechanism exists to generate post-peak shear strengths then it is recommended that residual shear strengths are used in the limit equilibrium stability analysis. A factor of safety that would be deemed acceptable in this instance would be less than the factor of safety against failure using peak shear strengths. If the design allows for the development of post-peak shear stresses, then it follows that displacements will occur at the interfaces and the lining system must be designed to ensure that these do not cause integrity failure. If such displacements are not desired or integrity cannot be assured through design, then it is suggested that peak shear strengths are used in the analysis with a suitable factor of safety that reflects the consequence of failure. The mobilisation of post-peak shear strengths could result in a loss of integrity as large displacements along a side slope interface could lead to the loss of liner protection (e.g. Figure 4). A common approach in landfill design is to ensure that the weakest interface is above the primary liner so that any deformations do not result in movement within the liner itself with loss of integrity leading to leakage of gas and leachate (e.g. Gallagher *et al.* 2003). However, adequate protection must be ensured when large displacements are anticipated above the primary liner and this is particularly an issue when

considering steep wall lining systems since the relative displacement at the interfaces can be in the order of several metres.

The consequence of failure must be reflected in the selection of both the interface strength parameters and factors of safety. For example, failure of a basal lining system would be costly and difficult to repair, whereas veneer stability failure, although highly undesirable, could be repaired with lower disruption and cost. For high risk design cases such as failure of a basal liner, both Gilbert (2001) and Thiel (2001) proposed an approach based on ensuring that the factor of safety using the residual strength controlling lining system stability is > 1.0 (if only just so), with a higher factor of safety obtained using the peak strength (e.g. 1.5). Essentially in this approach the consequence of failure is being taken into consideration, although only in a simplistic way.

An important consideration when selecting whether to use peak or residual shear strengths in design is to understand that the residual strength controlling stability of the whole lining system is not the interface with the lowest residual strength, but the residual strength for the interface with the lowest peak strength (Gilbert 2001). This approach confirms the importance of carrying out site specific interface tests for all combinations of materials to be used in the lining system, and the need to obtain the full shear strength/displacement relationship for each interface. It is only when armed with this information that the designer can identify the interface(s) controlling stability and then apply appropriate factors of safety. It should be noted that at locations along a lining system it is possible for the controlling interface to be different. This can occur due to the normal stress dependency of interface shear strength.

As discussed in Section 5.1, many of the mechanism that can lead to the mobilization of post-peak shear strengths are construction related. The construction quality assurance process (CQA) should control:

- Method of material placement to minimise any dragging

- Specify minimum soil cover over geosynthetic before being trafficked, and limit the type of plant and its operation

- Specify methods of soil placement on slopes (i.e. spread up slope not down) and minimise vehicle operations (e.g. braking);

- Discourage construction of haul roads on side slopes

- Control handling and storage of geocomposite materials so that internal strength is not compromised.

The following design issues should also be considered:

- If interfaces with low strength are used to isolate the geomembrane from shear stresses then the constructability of the lining system must be checked (e.g. check veneer stability and any temporary waste slopes)

- Check the tensile stress in each layer in addition to assessing stability. This assessment also requires peak and residual strength information for each interface.

- Designs should limit the use of compacted soils over geosynthetics, especially on slopes.

6.   UNCERTAINTY AND VARIABILITY

6.1 Limit equilibrium

Stability of landfills is controlled by slippage at interfaces between the lining components. Designers must ensure that all potential failure mechanisms are considered, with appropriate strength parameters selected (i.e. peak, residual or somewhere in-between) and the interface that controls stability identified. Information on the variability of interface shear strength is required both to carry out limit equilibrium stability analysis using characteristic shear strengths and to analyse the probability of failure. Current practice is still to carry out a

limited number of site-specific tests, and this provides insufficient information on the variability of interface strength for design. Measured shear strength variability of commonly used interfaces has been reported by Criley & Saint John (1997), Koerner & Koerner (2001), Stoewahse *et al.* (2002), McCartney *et al.* (2004), Dixon *et al.* (2006) and Sia & Dixon (2007). Sia & Dixon (2007) demonstrate that interface shear strengths and derived strength parameters can be represented by normal distributions. They also demonstrate that variability of interface strengths computed using published global data sets are 3 to 5 times, and reach up to 8 times, higher for the derived parameters compared to repeatability datasets (i.e. data obtained for material from single sources, tested by one operative using one shear device). It is concluded that variability and uncertainty computed using global and inter-laboratory datasets yield unreliable designs and this leads to a strong recommendation that published data should not replace measurements from site specific testing.

With the availability of interface variability data it is possible to undertake risk assessment of landfill stability using probability of failure. Common stability mechanisms (i.e. veneer and waste slope stability) have been considered by Koerner & Koerner (2001), Sabatini *et al.* (2002), McCartney *et al.* (2004), Dixon *et al.* (2006) and Sia & Dixon (2007). All employ the first-order, second moment reliability-based methodology proposed by Duncan (2000). These studies show that designs based on published global datasets result in unacceptably high probabilities of failure for controlling stability mechanisms, and they highlight the need for landfill designers to give greater consideration to variability of interface shear strength and to the consequences of failure. Design based on combined criteria for factor of safety and probability of failure would allow uncertainty in measured interface strength to be considered fully. However, appropriate and attainable target factors of safety and probability of failure values need to be selected if this methodology is to be implemented in general practice. A key requirement is that regulators, operators and designers need to

agree acceptable design requirements in relation to probability of failure. This will support justification of the cost of obtaining the required quality of input parameters in relation to the cost and consequences of failure. Although reliability based assessment using the approach proposed by Duncan (2000) is relatively straightforward, it is perceived by the Authors that there is reluctance for engineers to incorporate it into the design process. In an attempt to overcome this inertia, Sia & Dixon (2008) have developed a reliability based design chart for veneer cover soil stability. The chart can be used to enhance decision-making by taking into account uncertainties in the design parameters, such as the variability of interface shear strength parameters. Additionally, the chart can also be used to determine the optimum slope angle for a containment facility that will satisfy both the target factor of safety and acceptable failure probability. It is believed that currently reliability based approaches are seldom used by designers.

6.2 Numerical analysis

The influence of variability and uncertainty on integrity of lining system components cannot be assessed using limit equilibrium and the reliability based techniques outlined in Section 6.1. Specifically, tensile stresses generated in geomembrane and geotextile layers by waste settlement requires analyses that can represent material variability, geometry variability and the construction process, including waste placement. Sia (2007) presents numerical analyses to examine the integrity of a constructed shallow sloped landfill lining system (i.e. including staged waste placement), in which the uncertainty of significant input parameters are treated probabilistically using Monte Carlo simulation. Long-term waste degradation effects are not considered. Statistical information required to derive distributions of input parameters were obtained from literature, a laboratory interface repeatability testing programme and an expert elicitation process. Strain softening interfaces were incorporated between the lining elements.

Outputs from the analyses include the relative shear displacements within the lining system, informing the likelihood of generating post peak strengths and discontinuity of elements, and the tensile strains in the geosynthetic components, both generated by downdrag settlement during waste placement. It was concluded that discounting the variability of significant input parameters, such as interface strength, can lead to unsafe design as a result of not considering potential failure scenarios. The analyses presented by Sia (2007) are complex and time consuming to undertake and it is unlikely that this approach would be used for routine landfill design. However, such studies can guide the engineer regarding combinations of landfill geometry and lining materials that could produce unsafe designs and/or loss of integrity.

7. SUMMARY

This paper reviews current understanding and practice for design of landfill lining systems. Both stability (large scale movements leading to collapse) and integrity (small scale deformations leading to loss of function) failure mechanisms are identified. There is evidence in the literature that significant waste slips involving the lining system are still occurring, despite what appears to be established design guidance. In addition, there is growing evidence that integrity failures are occurring post waste placement. Many of the failure mechanisms are linked to the adjacent waste body behaviour and the requirement to consider interaction between the waste body and lining components in all designs is established. In design it is no longer acceptable to ignore interaction between waste and lining system and/or to wish the waste body in place thus ignoring the influence of waste placement process and sequence on lining system performance.

Despite the heterogeneous nature of MSW, the extreme range of materials around the world and the spatial and temporal variations within a given landfill, research has demonstrated that mechanical properties of MSW often vary in a logical and consistent

relationship (e.g. depth/stress dependency of stiffness). While there is a growing literature on measured properties that can be used to support selection of parameters for design, lack of a standardised classification system and test procedures has meant that in many cases it is difficult to translate tests on waste from one site and application to another. This situation is improving with the publication and acceptance of a classification framework. Given the importance of waste mechanics information there is still significant further work required to produce relevant information to enable routine design of robust lining systems. In addition, further development and validation of waste constitutive models is required if combined short-term compression and long-term creep and degradation behaviour is to be incorporated in two dimensional models of waste deformation.

Traditional limit equilibrium analysis methods can be used to assess stability of the lining elements and waste body (i.e. ultimate limit states), however the designer must select appropriate shear strength parameters (i.e. peak, residual or somewhere in between) by taking into consideration all possible mechanism that could cause relative displacement at interfaces, including waste settlement. There is now a significant body of information in the literature based on experience, numerical modelling and a limited number of field measurements to guide the designer on which strength parameters to select. This information includes aspects of interface type, slope angle, waste stiffness and waste thickness.

Limit equilibrium techniques cannot be used to assess integrity of the lining components. Use of numerical analysis to investigate integrity of the lining system has become common in the UK as part of the design process used to obtain a permit. However, it should be noted that this level of analysis is not required if the design engineer can demonstrate that the site specific landfill geometry (i.e. shallow side slope), waste type (i.e. high stiffness, low organic content) and mode of operation (i.e. waste filling sequence) are unlikely to produce integrity failure mechanisms. If numerical modelling is considered relevant the designer must establish

shear behaviour for the interfaces, including any post peak reduction in strength, typical waste properties such as unit weight, stiffness and compression due to degradation, and the lining system and waste construction sequence. Such analyses are capable of identifying conditions that could lead to relative deformations in the order of 100s to 1000s mm at side slope lining component interfaces even though global stability is acceptable. If deformations are indicated below the primary liner (e.g. geomembrane) then the design should be revised to provide a weaker interface above the geomembrane so that the integrity of the liner is not compromised. If significant deformations are indicated above the liner then the design must ensure that materials employed to protect the liner and form a drainage layer remain continuous. Although use of numerical models of waste/lining system interaction is becoming common it should be noted that there is still a dearth of data available to validate the models. A research project is currently in progress to provide field measurements of in-service liner performance for use in a validation exercise.

Despite recent advances in numerical modelling, measurement of waste mechanical properties and design practice, there is still significant uncertainty regarding many of the material parameters and processes required to analyse and hence design landfill lining systems interacting with waste. Probability of failure analysis can be used to better understand the significance of poor or limited input data for limit equilibrium analyses of stability. A growing number of published databases from repeatability testing programmes can be used to establish variability of interface shear strength data. In order for probability of failure analyses to be used more widely, designers, operators and regulators must agree threshold levels for acceptable performance. Numerical analysis using Monte Carlo simulation to investigate the influence of parameter variability on integrity mechanism has produced interesting results that could be used to guide engineers. However, the complexity and extended time required to

conduct such analyses mean that they are unlikely to be used for standard design situations in the foreseeable future.

8.   ACKNOWLEDGEMENT

9.   REFERENCES

Brink, D., Day, P.W. and du Preez, L. (1999) "Failure and remediation of Bulbul Drive landfill: Kwazulu-Natal, South Africa". Proc. Waste Management Conference, Sardinia 1999, Cagliari, Italy, 555-562.

Council of European Community, (1999) Council Directive of 26 April 1999 on the landfill of waste (1999/31/EC). Official Journal of the European Communities, L182/1.

Criley, K.R. and Saint John, D. (1997) "Variability analysis of soil vs. geosynthetic interface friction characteristics by multiple direct shear testing". Proc. of Geosynthetics '97, Long Beach, CA, 885-897.

Dixon, N. and Jones, D.R.V. (2003) Stability of landfill lining systems: Report No. 2 guidance, R&D Technical Report P1-385, Environment Agency.

Dixon, N. and Jones, D.R. (2005) "Engineering properties of municipal solid waste". Geotextile and Geomembranes, 23, 205-233.

Dixon, N., Jones, D.R.V. and Fowmes, G. (2006) "Interface shear strength variability its use in reliability-based landfill stability analysis". Geosynthetics International, 13, No. 1, 1-14.

Dixon, N. and Langer, U. (2006) "Development of a MSW classification system for the evaluation of mechanical properties". International Journal of Waste Management Science & Technology, 26, 3, 207-318.

Dixon, N., Ng'ambi, S. and Jones, R., (2004) "Structural performance of a steep slope landfill lining system". Proc of the ICE, Geotechnical Engineering. Vol. 157, 115-125.

Dixon, N., Langer, L., Reddy, K., Maugeri, M., Tinjum, J., Mahler, C., Cho, Y. (2011) "Waste Characterization". Geotechnical Characterization, Field Measurement and laboratory Testing of Municipal Solid Waste (GSP209). Proceedings of the 2008 International Symposium on Waste Mechanics. March 13, 2008 New Orleans, Louisiana, 135-152.

Duncan, J.M. (2000) "Factors of safety and reliability in geotechnical engineering". Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 126, No. 4, 307-316.

Eid, H.T., Stark, T.D., Evans, W.D. and Sherry, P.E. (2000) "Municipal Solid Waste Slope Failure. I: Waste and Foundation Soil Properties". J. of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 126, No. 5, May, pp. 397-407.

Fassett, J.B., Leonards, G.A. and Repetto, P.C. (1994) "Geotechnical Properties of Municipal Solid Wastes and Their Use in Landfill Design". Earth Tech. 1994.

Filz, G.M., Esterhuizen, J.J.B. and Duncan, J.M. (2001) "Progressive failure of lined waste impoundments". Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 841–848.

Fowmes, G.J., Dixon, N., Jones, D.R.V. and Cowland, J. (2006) "Modelling of Lining System Integrity". Proceedings 8th International Conference on Geosynthetics, Yokohama, Japan.

Fowmes, G., Dixon, N. and Jones, D.R. (2007) "Landfill Stability and integrity: the UK design approach". Waste and Resource Management, 160 WR2, 51-61.

Fowmes, G.J., Dixon, N. and Jones, D.R.V. (2008) "Validation of a numerical modelling technique for multilayered geosynthetic landfill lining systems". Geotextiles and Geomembranes, 26, 109–121.

Gallagher, E.M.G., Needham, A.D. and Smith, D.M. (2003) "Use of Geosynthetics in landfill steepwall lining systems". Proc. of the East midlands Geotechnical Group symposium, Nottingham, Sept 2003. 71-91.

Gilbert, R.B. (2001) "Peak versus residual strength for waste containment systems". Proc. GRI 15, Hot Topics in Geosynthetics II. 29-39.

Gourc, J.P., Thomas, S. and Vuillemin, M. (1998) "Proposal of a waste settlement survey method". Proceedings of Environmental Geotechnics, Editor Seco e Pinto, 195-200.

Jones, D. R. and Dixon, N. (2003) Stability of Landfill Lining Systems: Report no.1 Literature Review. Environment Agency Research and Development Technical Report P1-385/TR1.

Jones, D.R.V. and Dixon, N. (2005) "Landfill lining stability and integrity: the role of waste settlement". Geotextiles and Geomembranes, 23, 1, 27-53.

Jones, D.R.V., Dixon, N. and Connell, A. (2000) "Effect of landfill construction activities on mobilised interface shear strength". Proceedings of 2nd European Geosynthetics Conference, Bologna, Italy, October, Eds. Cancelli, A., Cazzuffi, D. & Soccodato, C., 2, 581-586.

Kavazanjian, E. (2001) "Mechanical properties of municipal solid waste". Proceedings Sardinia '01, 8th International Waste Management and Landfill Symposium, Cagliari, Italy, 415-424.

Koerner, R. M. and Daniel D. E. (1997) Final Covers for Solid Waste Landfills and Abandoned Dumps. American Society of Civil Engineers.

Koerner, R.M. and Soong T-Y. (1998) "Analysis and design of veneer cover soils". Sixth International Conference on Geosynthetics, Atlanta, 1, 1-23.

Koerner, R.M. and Soong, T-Y. (2000) "Stability Assessment of 10 Large Landfill Failures". Geodenver, ASCE SPECIAL, Vol. 103, 1-38.

Koerner, R.M. and Koerner, G.R. (2001) "Geosynthetics design beyond factor of safety; risk assessment using probability of failure analysis". Proc. GRI-15, Hot Topics in Geosynthetics-II, December 2001, Houston, 235-253.

Landva, A. O. and Clark, J. I. (1990) "Geotechnics of Waste Fill. Geotechnics of Waste Fill - Theory and Practice", ASTM STP 1070. ASTM. Philadelphia, Pa., pp. 86-106.

Langer, U. and Dixon, N. (2006) "Potential of a waste classification for the assessment of waste geotechnical properties", Proceedings of Abfallforschungstage, Goettingen, 2006, pp. 173-185.

Long, J.H., Gilbert, R.B. and Daly, J.J. (1995) "Effect of waste settlement on sloped lining systems". Proceedings of Geosynthetics '95, Nashville, Tennessee, USA, February, Vol. 2, 729–744.

Manassero, M., Van Impe, W.F. and Bouazza, A. (1996) "Waste disposal and containment". Proceedings Second International Congress on Environmental Geotechnics, Osaka, 3, 193-242.

McCartney, J.S., Zornberg, J.G., Swan, R.,H. Jr. and Gilbert, R.B. (2004) "Reliability-based stability analysis considering GCL shear strength variability". Geosynthetics International, 11, No. 4, 241-262.

McDougall, J. (2007) "A hydro–bio–mechanical model for settlement and other behaviour in landfilled waste". Computers and Geotechnics, 34, 229–246.

McDougall, J. (2011) "The long and the short of it". Geotechnical Characterization, Field Measurement and laboratory Testing of Municipal Solid Waste (GSP209). Proceedings

of the 2008 International Symposium on Waste Mechanics. March 13, 2008 New Orleans, Louisiana, 75-111.

Meissner, H. and Abel, K. (2000) "Numerical investigations of inclined landfill liner systems". Proceedings of the GeoEng2000, Melbourne, Australia, published on CD.

Reddy, K.R., Hettiarachchi, H., Parakalla, N.S., Gangathulasi, J. and Bogner, J.E. (2009) "Geotechnical properties of fresh municipal solid waste at Orchard Hills Landfill, USA". Waste Management, 29(2), 952-9.

Rowe, R. K. (2005) "Long-term performance of contaminant barrier systems". Geotechnique, 55, 631–678.

Sabatini, P.J., Griffin, M.L., Bonaparte, R., Espinoza, R.D. and Giroud, J.P. (2002) "Reliability of state of practice for selection of shear strength parameters for waste containment system stability analyses". Geotextiles and Geomembranes. Vol. 20, 241-262.

Seed, R.B., Mitchell, J.K. and Seed, H.B. (1988) Slope stability failure investigation: Landfill Unit B-19, Phase 1-A, Kettleman Hills, California. Report No. UCB/GT/88-01, Department of Civil Engineering, University of California, Berkeley, July.

Stoewahse, C., Dixon, N., Jones, D.R.V., Blümel, W. and Kamugisha, P. (2002) "Geosynthetic interface shear behaviour: Part 1 test methods", Ground Engineering, Vol.35, 2, 35-41.

Sia, A.H.I. (2007) Landfill Lining Engineering Designs: A Probabilistic Approach, Ph.D. thesis, Loughborough University, United Kingdom.

Sia, A.H.I. and Dixon, N. (2007) "Distribution and variability of interface shear strength and derived parameters". Geotextiles and Geomembranes. Vol. 25, 139-154.

Sia, A.H.I. and Dixon, N. (2008) "Reliability-based design: Veneer cover soil stability". Geosynthetics International, 15, 1, 1-13.

Stoltz, G., Gourc, J.P. and Oxarango, L. (2010) "Characterisation of the physic-mechanical parameters of MWS". Waste Management, Vol. 30, Issues 8-9, pp. 1439-1449.

Thiel, R. (2001) "Peak vs residual shear strength for landfill bottom liner stability analyses". Proceedings of the 15th Annual GRI Conference Hot Topics in Geosynthetics – II, presented in Houston, TX, Dec. 13, 2001, Geosynthetics Institute, Folsom, PA, pp. 40-70.

Qian, X., Koerner, R. M. and Gray, D. H. (2002) Geotechnical aspects of landfill design and construction. New Jersey: Prentice Hall.

Villard, P., Gourc, J.P. and Feki, N. (1999) "Analysis of geosynthetic lining system (GLS) undergoing large deformations". Geotextiles and Geomembranes, Vol. 17, 1, 17-32.

Zamara, K.A., Fowmes, G. and Dixon, N. (2010) "In-service structural performance of landfill lining components: instrumentation of a side slope lining system". Proceedings 9th International Conference on Geosynthetics, Editors: Palmeira, E.M., Vidal, D.M., Sayao, A.S.J.F. & Ehrlich, Brazil, May, 1061-1066.

Zamara, K.A., Dixon, N., Jones, D.R.V. and Fowmes, G. (2012) "Monitoring of a landfill side slope lining system: Instrumentation selection, installation and performance". Geotextiles and Geomembranes (In press)).

Zekkos, D. (Editor) (2011) Geotechnical Characterization, Field Measurement and laboratory Testing of Municipal Solid Waste (GSP209). Proceedings of the 2008 International Symposium on Waste Mechanics. March 13, 2008 New Orleans, Louisiana.

APPENDIX B   PAPER 2

Full Reference

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PAPER TYPE – JOURNAL PAPER

Monitoring of a landfill side slope lining system: Instrument selection, installation and performance.

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ABSTRACT: Municipal solid waste landfill barrier systems often comprise a combination of geosynthetics and mineral layers. Throughout the last twenty years there has been extensive research on the interactions between the materials and on performance of the geosynthetics including aspects of durability. This research has resulted in significant advances in the design and specification of landfill lining systems. However, to date there has been limited research carried out on *in situ* landfill lining system behaviour. Measured behaviour from field scale trials and of in service operation can provide valuable information on landfill lining system performance and allow a better understanding of composite material behaviour. Although many numerical modelling programs are applied to evaluate lining system stability and integrity, data to validate these models is currently limited. This paper highlights the data required to validate numerical models and instrumentation techniques that may be used to acquire this information. The paper focuses on geotechnical instrumentation deployed on the side slope lining system at the Milegate Extension Landfill, UK. The instrumented lining system comprises 1.0 m of compacted clay, a 2 mm double textured high density polyethylene

geomembrane, a nonwoven geotextile and a sand cover soil layer. Instrument selection and problems associated with acquiring consistent, reliable and valuable data in a field environment are discussed, as are the challenges and problems that occur when preparing a full scale experiment. Sources of uncertainties within readings are highlighted. Additionally, initial results collected during sand veneer layer placement on the slope are presented. These demonstrate acceptable instrument performance over a 2 year period. Measured behaviour highlights the significance of geomembrane strains driven by temperature changes, generation of post peak strengths at interfaces during fill placement on the side slope due to relative displacement at interfaces between components, and mechanisms of stress redistribution in the geomembrane that result in time dependent changes in strain under constant load and temperature conditions.

1   Introduction

Modern landfill engineering in the UK involves detailed analysis of construction and environmental matters in order to meet requirements of the Environmental Agency, European Union regulations and UK legislation. This is to minimise impact on human health and ensure environmental safety. According to the Council Directive 1999/31/EC (1999) official required measures are associated with landfill emissions: leachate volume and composition, surface water composition, gas emission and atmospheric pressures, these are related to waste classification. While advanced systems of design and construction are mandatory according to Landfill (England and Wales) Regulations (Environment Agency 2002), there is no formal requirement to monitor directly the mechanical performance of the lining system. Monitoring parameters focus on the environmental impact of the liner's performance (*eg.* groundwater contamination) and reported data in the case of exceeded values may relate to an already damaged liner. Even when monitoring is required as a condition of the license for a particular landfill, data is often not published. Information about lining system performance can be derived from back analysis of large scale landfill failures (Koerner and Soong 2000, Dixon and Jones 2003, Muhsiung 2005) but these cannot provide insight into in-service performance of nominally stable facilities. Current practice in landfill design involves applying numerical modelling software and employing complex methods for predicting landfill lining system behaviour to evaluate displacements, strains, and tensile stresses resulting from waste body lining system interaction (Dixon and Jones 2003). However, there is still little attention given to in-service performance of landfill lining systems and interaction of the materials within a barrier system. Although in-service failure can lead to environmental damage, commonly used design approaches have not been verified through monitoring of liner behaviour during construction, filling and after closure.

To meet requirements for an environmentally safe landfill it is important to maintain the lining system stability (ultimate limit state) and integrity (serviceability limit state) throughout the landfill lifetime. While stability of the landfill involves large scale movements (*e.g.* slope failure), integrity is related to overstressing of liner elements with consequent loss of original functions, according to which the liner was designed (*e.g.* low permeability barrier, protection layer). The key areas for a landfill design engineer are: side slope (steep/shallow) stability/integrity, basal lining system stability/integrity, subgrade behaviour, ground water behaviour, appropriate material selection for the barrier components and waste parameters. It is of high importance to build environmentally safe landfill constructions and to assess adequately the performance of landfill lining systems and in particular, to predict stresses and strains in lining elements resulting from waste placement and settlement. To the Authors' knowledge only a limited number of full scale geotechnical landfill monitoring research projects have been conducted to investigate landfill liner behaviour (Gourc *et al.* 1997, Bouthot *et al.* 2003, Nakamura *et al.* 2006) but a limited number of long term monitoring experiments with back analysis have been reported (Najser *et al.* 2010, Villard *et al.* 1999). Although there is an increasing number of laboratory projects investigating lining system behaviour (*e.g.* Gourc *et al.* 2010, Koerner *et al.* 1997) and attempts to develop software calibration have been undertaken (Vilard *et al.* 1999, Fowmes *et al.* 2008), there is still an urgent need for information on in-service physical performance of barriers. Additionally, much attention has been given to landfill temperature monitoring and assessment of landfill lining system durability depending on liner temperature (*i.e.* Rowe and Sangham 2002, Rowe *et al.* 2008, Rowe and Hoor 2009) and also studies of waste body parameters and mechanical properties have been reported (Fassett *et al.* 1994, Gotteland *et al.* 2002, Dixon and Jones 2005, Stoltz *et al.* 2010). Nevertheless, limited information exists about geosynthetics liner performance throughout cell operation and after closure.

The aim of an ongoing study conducted by the Authors is to validate design approaches. This paper reports an investigation of the mechanical performance of a multilayered landfill lining system (Figure 1) at various stages of landfill development (*i.e.* barrier response to applied pressures in relation to waste placement during subsequent stages of cell filling, waste compaction and liner performance after cell closure due to waste settlement). This paper describes the challenges associated with design of a landfill lining monitoring system. It reports the research conducted at Milegate Extension Landfill in East Yorkshire, UK (Figure 2), where a section of landfill slope has been monitored using geotechnical instrumentation to measure stresses imposed on the liner, displacements within and between the lining elements, strains within the liner elements, and also temperature of the clay surface and geosynthetics. This provides an opportunity to obtain valuable information to aid the design of future landfill lining systems and to assess performance of existing systems. Problems associated with waste barrier interactions, interface properties and mechanisms involved in certain material/interface behaviour are also investigated. The project started in June 2009. Instrumentation consists of pressure cells (PC), extensometers, fibre optics (FO) and Demec strain gauges. In addition, site slope surveys were conducted using laser scanning. To date, response of the lining system to placement of three soil veneer layers and 9 m thickness of waste body has been recorded.
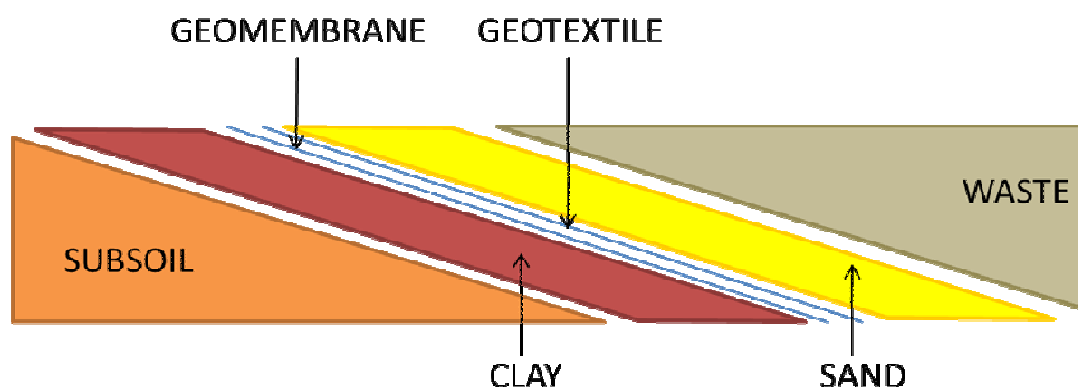

Figure 1. Schematic view – Milegate landfill multilayered side slope lining system.

1.1   Context of the research

Construction of landfill barriers typically involves placement of geosynthetic materials over a mineral layer, which is often compacted clay. Further, landfill construction often comprises a mineral drainage layer and subsequent placement of waste layers. In the last 15 years there has been significant improvement in understanding of landfill lining systems. Numerical modelling codes have been developed with specific functions for analysing landfill site geotechnical problems (Fowmes *et al.* 2008, Villard *et al.* 1999) including staged placement of municipal solid waste , mobilized shear strength of the geosynthetic lining system interfaces (strain softening and progressive failure), tensile stresses in geosynthetics elements and representation of waste behaviour (Zhang 2007, Machado *et al.* 2002). Construction stages result in deformation of components and hence shear stresses developed between and within materials and consequently formation of tensile stresses in the geosynthetic elements. The importance of several factors has been established and these should be considered when designing landfill lining system. They include consideration of progressive failure through strain softening of interfaces between geosynthetics and geosynthetics/soil materials, staged placement of the waste body, consideration of tensile stresses in geosynthetics, and assessment of waste properties such as unit weight, stiffness and strength, and their change due to ageing, deformation and settlement. An important challenge is the selection of peak or residual interface shear strength parameters for use in limit equilibrium stability analyses of multi-layered lining systems. The importance of accurate prediction/design of engineering aspects of landfill behaviour is self evident.

Previous studies undertaken to investigate landfill liner behaviour have been based on back analysis of monitored slopes (Villard *et al.* 1999), laboratory results (Fowmes *et al.* 2008), landfills failures (e.g. extensive numerical and laboratory analysis of Kettlemen Hill failure - Seed *et al.* 1990, Byrne *et al.*1992, Mitchell *et al.* 1990*,* Chang *et al.* 1999) and

parametric studies partially based on global databases (Kodikara 2000, Sia 2007). However, a validated approach taking into consideration the full complex nature of the landfill lining systems is still unattainable, and research is required to achieve a better understanding of the material behaviour and interaction mechanisms incorporated in models, to remove conservatism in the design process and to assure confidence and optimal construction geometry.

Recently, a number of laboratory investigations have been carried out using centrifuges (*i.e.* size reduced models with increased model weight, which simulates *in situ* stress conditions). Thusyanthan *et al.* (2007) conducted tests to investigate tension within geomembranes (GMB) occurring due to static and dynamic loadings and compared these with results of limit equilibrium methods. Gourc *et al.* (2010) reported a test carried out on landfill clay capping in a centrifuge in order to investigate deformation within cap barrier and Viswanadham and Rajesh (2009) investigated behaviour of landfill basal clay barrier and the effect of differential settlement.

2 Details of the Field Trial

Milegate Extension Landfill is located in East Yorkshire, UK, at National Grid Reference TA 131 472 (Figure 2). The void used for the landfill cell was formed as a result of sand and gravel extraction. The landfill accepts inert and non-hazardous, building, agricultural, commercial and industrial waste. However mostly the waste body comprises of household, campsite wastes and limited construction site waste.

Figure 2. Milegate Extension Landfill, monitored section (black strip in centre of the slope) in cell 3 and site general location.

The monitored slope has a length of 31.2 m and is 12 m high with an inclination angle of 1v:2.5h (~21.8°). Figure 3 presents the general geometry of the slope. The geosynthetic lining system deployed during the experiment was placed in addition to the existing-clay liner, and therefore is an additional and hence sacrificial layer that does not form part of the approved containment system. The combination of materials within the lining system: clay, GMB, geotextile (GTX) was chosen to represent common practice in the United Kingdom. One panel of the geocomposite drainage material that covers the clay liner over the whole cell was replaced by one 5m wide panel of the high density polyethylene (HDPE) GMB and GTX trial lining system.

The instrumented lining system comprised a 2 mm double textured HDPE GMB 5 m wide, density 0.949 g/cm^3 (GMB TMT from Atarfil S.L.). Information on the tensile stress/strain behaviour of the GMB is available from the manufacturer and will be used as input data for the numerical modelling study. The GMB is overlain by a non-woven needle punched GTX 5m wide panel. This protection layer has a static puncture strength [CBR] of 14 kN, thickness of 7.8 mm and weight of 1400 g/m^2 (HPS14 from GeoFabrics Ltd). The manufacturer's data on tensile stress/strain behaviour will also be used as input data for the numerical model. The multi-layered landfill system is schematically presented in the

Figure 1. The GMB and GTX were placed along the entire length of the side slope and fixed at the top of the slope in a "U" shaped 600 mm x 600 mm anchor trench.
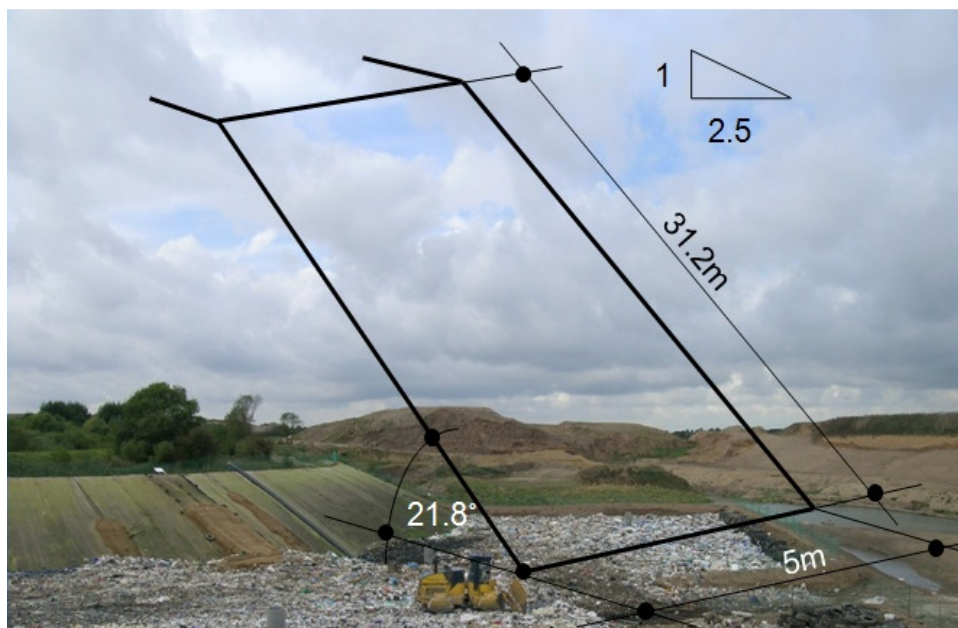

Figure 3. Milegate Extension Landfill - slope geometry.

A nominally 0.5 m thick sand veneer was placed in stages on the GTX ahead of waste placement. This represents the common practice of providing a mineral drainage layer on side slopes. This would typically be a gravel drainage layer but as this material was not locally available, sand was used as a replacement as it produces loading equivalent to the medium to coarse gravel typically used for mineral drainage layers. Although use of sand does not represent current construction practice it is not significant because the numerical modelling phase of the study will model the constructed configuration, with results compared to the observed site behaviour. Prior to waste filling the sand layer was placed parallel to the slope along 10 m of slope length. When the waste body reached the top of the first veneer a second 10 m long sand layer measured parallel to the slope was placed along the slope. When the waste reached the top of the second veneer layer a third and last veneer trial was constructed such that the whole length of the slope was covered with a 0.5m thick sand layer.

Placement of the sand veneers provides opportunity to measure response of the underlying geosynthetic components to the applied load. The sand veneer was placed by a digger arm. It was compacted using the bucket and not traversed by the digger (the arm was sufficient length to place the full length of veneer without tracking on placed sand). This paper presents results from the second veneer trial (Figure 4). History of undertaken and planned works is presented chronologically in Table 1.
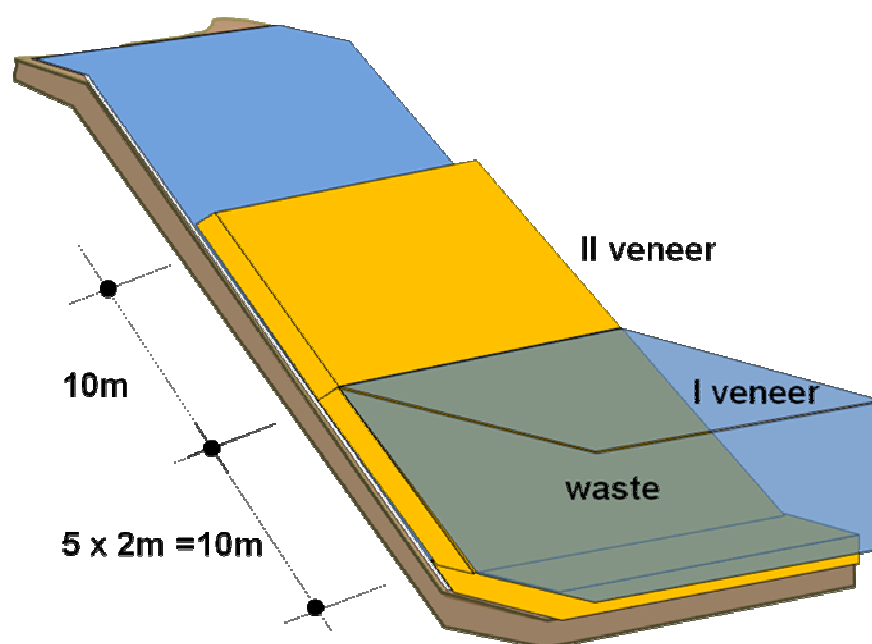

Figure 4. Stages of sand veneer construction and waste filling

Description of the event	Date
Installation of the instrumentation/materials;	07/2009
1st sand veneer; 10 m of the lower part of the slope under the veneer;	11/2009
1st scanning;	11/2010
2nd sand veneer; 20m of the lower part of the slope under the veneer;	11/2010
2nd scanning;	11/2011
3rd sand veneer; Demec gauges covered by the veneer; All the slope covered by the veneer;	11/2011
Planned: whole length of the slope covered with waste;	Summer 2012

Table 1. History of the works undertaken on the monitored slope.

3   Liner Instrumentation

Suitable selection of the instruments for a multilayered lining system (clay/GMB/GTX/sand) was of significant importance for the study. Instruments are expected to deliver data for several years. More importantly, as the main aim of the study is numerical model validation, it was important to measure parameters related to design criteria (*e.g.* displacement, strain, tension, pressure). High confidence in adequate instrument performance was of importance as damaged sensors are difficult to repair once waste has been placed.

The key aspects of instrumentation selection for this project can be listed as follows:

- To minimize instrumentation impact on the barrier *in situ* performance (*i.e.* so the instrumentation placement does not modify the measured values);

- To ensure instrument/material durability during interaction with the landfill environment;

- Provide desired accuracy;

- Design instruments for relatively simple and easy instalment in the landfill environment (*i.e.* maximum laboratory preparations to minimize and accelerate site works);

- Reasonable cost of the instrument;

- Minimise required maintenance works and hence implication for costs; and

- Provide the possibility of comparing results between different methods of measurement.

Instrumentation selected for Milegate Extension Landfill monitoring system, the numbers of sensors/instruments and measured parameters are listed in Table 2. Ongoing analysis of instrumentation readings resulted in purchase of the temperature logger to record GMB temperature in order to aid identification of relationships between measured behaviour of liner components in response to temperature changes. It is important to rigorously assess all the

collected data in order to identify inconsistency or unexpected behaviour of the instruments. This ensures the recording of valuable, reliable and comprehensive data. Early verification of the results and determination of errors/inconsistency allows for instrument adjustments or replacement.

Instrument	Number of instruments/ sensors	Measured parameter
Vibrating Wire Pressure Cells	4	Normal Stress
Extensometers	12	Displacement (at 6 points on the GMB, 6 on the GTX)
Demec strain gauges	16 steel disks	GMB strains across the slope
		GMB strains along the slope
Fibre Optics	15	GMB strains along the slope
	7	GMB temperature
Thermistors	2	Clay surface temperature
Additional records		Waste height
Temperature logger	1	GMB temperature

Table 2. Instruments and parameters measured at Milegate Extension Landfill.

4   Measured parameters and methodology

The proposed instrumentation at Milegate Extension Landfill provides the possibility of conducting a full-scale experiment with known dimensions, loadings and waste placement conditions. The experiment is designed to obtain information on relative displacements of lining elements, tensile behaviour of geosynthetic elements, loads applied to the lining system during and post waste placement, and temperature of lining components. Figure 5 presents schematically the location of instruments along the slope. It was considered of particular importance to monitor strains within the GMB with overlapping instruments to allow comparison and verification of the different approaches.
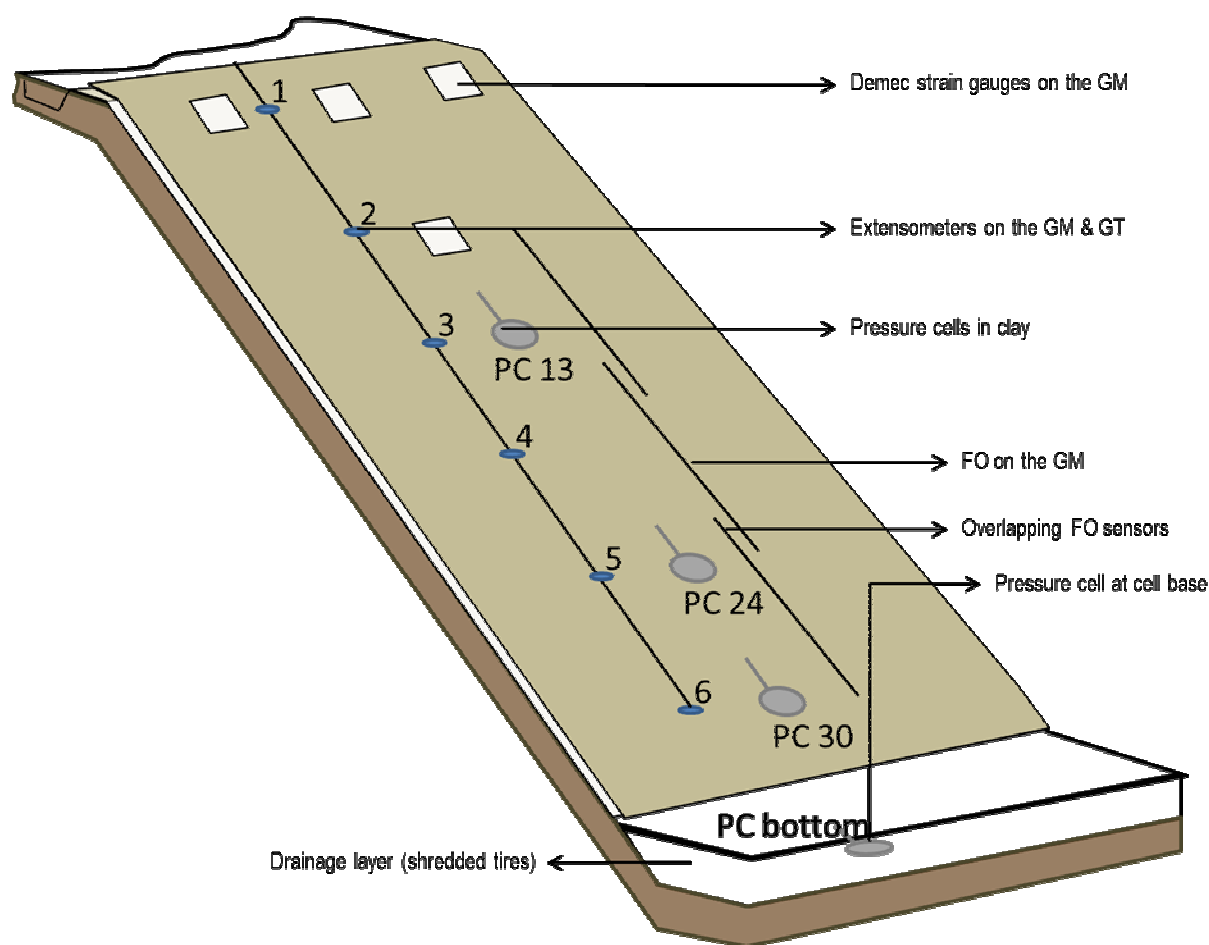
Figure 5. Schematic view of instrument locations along the slope.

4.1 Pressures imposed on the liner – pressure cells

Vibrating Wire Pressure Cells (PC) were chosen for installation in the top of the clay liner to measure total stresses imposed on the lining system during and after waste placement. They consist of two stainless steel plates welded together along their periphery. Space between the plates is filled with oil. Changes in the pressure on the plate surfaces corresponds to change of oil pressure within the cell, this is converted by a vibrating wire pressure transducer into an electrical signal, which is transmitted to the measuring table at the top of the slope where all cables terminate. Two of the PC are equipped with thermistors which allow the influence of temperature on measured pressure readings to be considered.

In total 4 vibrating wire PC were installed, 3 along the slope in shallow excavations and one in the cell base, placed in a plastic bag filled with sand under the one metre thick shredded tyre drainage layer. The cells were installed beneath the GMB, at distances 12.3m, 23.3m and 29.3m from the slope crest and one at the toe beneath the drainage layer. The second and third PC contained thermistors. Armoured cables attached to the instruments run along the GMB and GTX panel edge to the top of the slope, where readings are taken during site visits.

4.2 Displacement and relative displacement of the geosynthetic layers

Relatively simple wire extensometers (Dunnicliff 1993) were used as the method has already been proven to deliver valuable information about liner performance (*e.g.* Gourc *et al.* 1997). Six extensometers were installed on the GMB and six on the GTX. Each extensometer consisted of a wire (high tensile strength), with one end attached to the geosynthetics at locations along the slope measured from the crest of 3m, 8.4m, 13.8m, 19.2m, 24.6m and 30m. Figure 6 presents a schematic view of extensometer operation. The values of relative displacement between the geosynthetic lining elements are calculated from the GMB and GTX displacements. This follows the monitoring techniques used previously on landfill sites by Gourc *et al.* (1997), Koerner *et al.* (1997), Bouthot *et al.* (2003) and in laboratory research by Fowmes *et al.* (2008). This technique also allows calculation of strain values occurring within the GMB and GTX, though with relatively low accuracy and over a long gauge length. In terms of materials used to construct the extensometers it was essential to use durable high quality wire that can survive the robust landfill environment and ensure satisfactory performance during the years of monitoring. It is anticipated that the instruments would be exposed to elevated temperatures and mechanical impact due to waste placement and compaction.
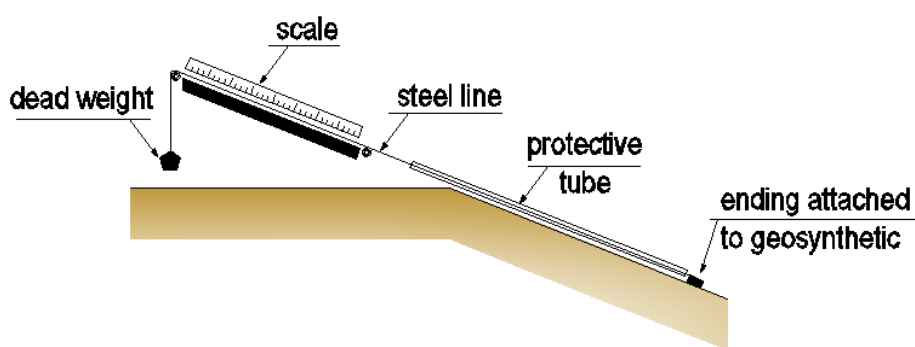
Figure 6. Extensometers operation.

Wire attachments to the GMB and GTX were prepared by drilling (GMB) or cutting (GTX) two holes measuring approximately 8 mm in diameter in the material allowing the wire to be pulled through and fastened in a loop. Holes in the GMB liner were acceptable in this case because it does not form part of the lining system required under the site hydrogeological risk assessment. The wires run up the slope in tubing to the measuring station. Free and smooth movement of the wire is ensured by the protective tubing which isolates the wire from forces imposed by the GMB, GTX and overburden (*i.e.* sand and waste). The wires pass through a system of pulleys across the measuring board (see Figure 7). Each wire is tensioned by an individual weight. To ensure sufficient durability of the instrument, wire rope 1.5mm diameter made of AISI 31G steel was selected. The material and diameter of the protective tubing were determined by the ease of pulling wire through the tube in laboratory trials. Consequently 1.5 mm nylon tube was selected. Temperature influence on wire extension or contraction is estimated and a correction factor applied.

Figure 7. Extensometers measuring table

4.3 Geomembrane strains and tensile behaviour

Strains within the GMB are measured by three independent methods: Fibre Optic (FO) Bragg Gratings, Demec strain gauges and extensometers. Each of the methods measures average strains over a different gauge length (Table 3). GMB strain measurement allows calculation of tension forces developed within materials if the stress/strain behaviour is known. This is an important aspect of the research as generally it is a common practice when designing landfill lining systems to set failure criteria on allowable stress or strain values in the primary GMB liner (typically HDPE) so as to minimise the possibility of environmental stress cracking. This study provides information on in-service GMB strains in response to a range of known loadings conditions.

Instrument	Gauge length
Extensometers	540 cm
Demec strain gauges	20 cm
Fibre optics	50 cm

Table 3. Distance of average strain measurement for different methods (*i.e.* gauge lengths).

4.3.1 Extensometers

This method allows approximate calculation of average strains between any two adjacent anchor points on the geosynthetic (*i.e.* over a length of 5.4 m). The measuring mechanism is robust with a relatively low resolution compared to the two other methods used on the site.

4.3.2 Fibre optics

FO were selected to measure both GMB strain and temperature. They are known to be relatively fragile but it was decided to deploy sensors along the GMB sheet in an attempt to obtain high resolution measurements. Figure 8 presents details of the FO measurement method. The system and method of deployment was developed by the Cranfield University Photonics Group. FO technology has been used in the study of geosynthetic for over 15 years. Geosynthetic sheets with embedded sensors are widely available but the technology is typically expensive. FO deformation monitoring systems have already been used to monitor geosynthetics including in landfill environments. Yashima *et al.* (2009) report strain distributions along a geogrid measured by FO installed in a longitudinal member of the geogrid. This was used to assess the stability of a geogrid reinforced soil wall during and after construction. Nakamura e*t al.* (2006) report deformation within a landfill cap monitored using FO technology, and it is believed to have delivered information over a period of several years. Additionally various geotechnical applications of geosynthetic with embedded FO sensors were investigated and developed in France (*i.e.* Nancey *et al.* 2007, Loke *et al.* 2006, Artieres *et al.* 2010).

In this research, FO with Bragg Gratings were deployed on the GMB. Bragg Gratings are structures within the FO that reflect particular wavelengths (see Figure 8b). When light travels through the fibre core, a particular wavelength is reflected by the Bragg grating. Deformation induced within the GMB is characterised by the strain within fibre Bragg

Grating causing a shift of wavelength, which is directly represented by the shifts in peaks of a spectrum graph (see Figure 8c).

FO installed at Milegate landfill were draw tower grating chains from FOS&S (Fibre Optic Sensors & Sensing Systems) with five strain sensors in each chain. The technology used by the company to write gratings in a fibre allows production of arrays without splicing. Six arrays of FO sensors were deployed at various positions along the slope. Three strings with 5 strain sensors each and 6 temperature sensors each were installed. Each array was originally 5m long, but it was decided to extend each by adding additional FO strings in between the Bragg gratings to increase sensor coverage along the GMB sheet. In addition, the FO arrays were located on the slope to ensure that the last sensor on a string overlapped the first sensor of the next array (Figure 5). It was possible to cover approximately 25 meters of GMB slope length with FO strain sensors. Bragg Gratings produced by Cranfield University were also deployed on the GMB to measure temperature changes, as this strongly influences strain sensor measurements.
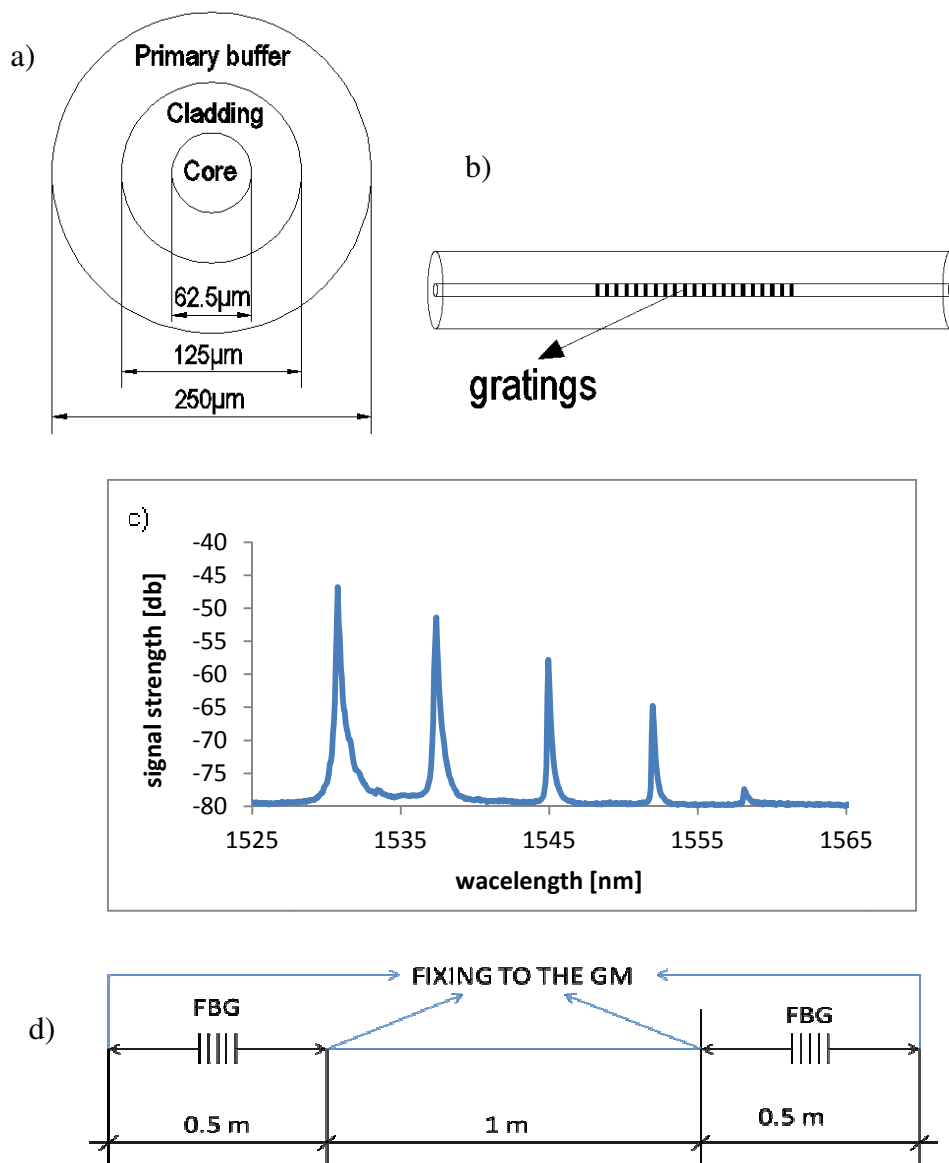
Figure 8. a) A typical size of the fibre (after Crisp and Elliot 2005); b) Uniform Fibre Bragg Grating (FBG); c) Typical wavelength spectrum; d) Schematic method of fibre Bragg grating installation on the GMB.

Strain gauges were located every second metre of the slope from the bottom. Fibre Bragg grating sensors are attached to the GMB to allow the strain to be measured over 0.5 m long gauge lengths (Figure 8d). To allow correct measurements of strains, the GMB surface had to be thoroughly polished to remove texture, dust and dirt and to obtain a smooth surface for FO sensor attachment. Both the GMB and FO coatings are low energy surfaces; therefore a structural plastic adhesive was used to adhere fibres to the GMB. In total, 15 strain and 7

temperature FO sensors were installed along the GMB. Silicon sealant was used to cover and protect the whole length of each fibre string.

4.3.3 Demec strain gauges

Demec strain gauges were selected for deployment on the GMB because of the low cost, simplicity and robustness of the method. The change in distance between pairs of measurement points is used to measure strains using a standard measurement instrument. Demec gauges are widely applied in concrete science for strain measurement on concrete beams. This is typically accomplished in laboratory conditions and is used to define tensile forces occurring within concrete structures under load. To the Authors' knowledge the instrument has not previously been used in the landfill environment.

The reading instrument consists of an invar steel bar nominally 200mm long with two conical-shaped points, one at each end, that are located in the receiving targets attached to the material. One of the conical points is able to rotate thus allowing the distance between them to vary. A dial gauge is used to measure the distance between the two measurement points to accuracy of 0.01mm. Stainless steel discs with a central depression to locate the conical point of the invar bar are attached to the GMB (Figure 9). Movement between the steel discs is obtained by a change in dial gauge reading. Measurements are taken manually during site visits. The purpose of these was to obtain tensile stresses near the anchorage point at the top of the slope by measuring strains and converting to stress. Four small U-shape incisions were made in the GTX to form flaps that gave access, with Demec steel discs located on the GMB beneath the flaps of GTX. Three measurement locations were installed along the crest of the slope and a fourth down the slope at the level of the first FO temperature sensor. Under each GTX flap four steel discs were glued to the GMB to form a square with approximately 200 mm side lengths. Use of pairs of discs in vertical and horizontal orientations allows strain readings down and across the slope respectively.
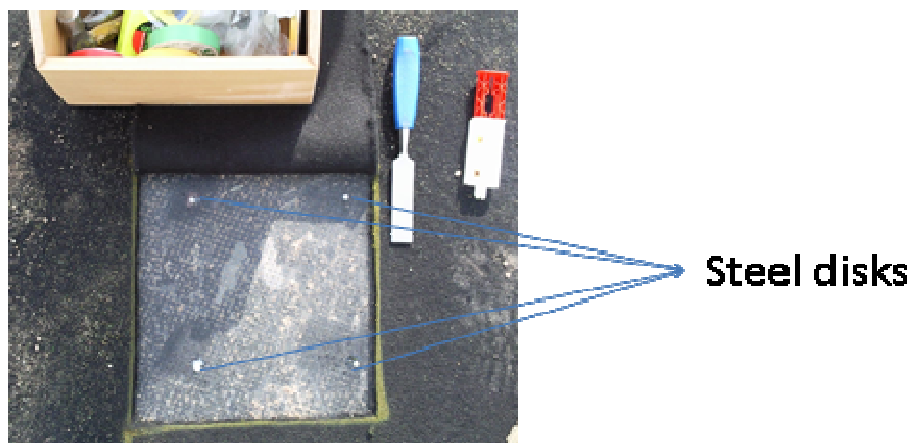
Figure 9. Demec strain gauges with steel disks placed approximately 200mm apart in a square on the GMB.

The accuracy of Demec strain measurements in laboratory are at a micro scale (0.001 mm), however the accuracy and precision of the GMB readings under site conditions is greatly reduced. Also, it was observed that the readout device can detect strains imposed on the GMB by the person obtaining the readings and hence care had to be taken to ensure that the person taking the readings did not influence the measurements. By taking this precaution the Demec strain gauges still provided consistent results (Section 7.2.2).

4.4 Temperature of the Liner

4.4.1 Pressure Cell Thermistors

Two of the four PC include an additional thermistor to measure temperature in the upper layer of the clay liner. The temperature is measured in the lower part of the slope: 24 m and 29.3 m below the slope crest. It was anticipated that the thermistors would allow application of temperature corrections for the PC readings but it was discovered that they were not suitable for the purpose (see Section 5).

4.4.2 Fibre Optics

Initially, temperatures were measured by FO in an attempt to provide temperature corrections for FO strain sensors, however limited reliability was achieved.

4.4.3 Temperature logger

During the second year of monitoring a thermometer logger was installed on the site so the changes in temperature on the GMB surface could be recorded directly. The device was located underneath the GTX approximately 9m below the crest and temperature between the GMB and GTX liner was recorded every half hour. Temperature is an important driving force generating strains within the GMB in the early stage of the project (*i.e.* expansion during increasing temperature forming wrinkles that disappear during reducing temperature which caused contraction). This process has been observed even when the GMB is constrained under low overburden pressures. A key challenge is related to interpretation of temperature sensitive measurements when instruments are partially uncovered, *i.e.* covered by the sand veneer or partially covered by sand and waste (*e.g.* the extensometers), as the temperature varies along the length of the instrument.

4.5 Waste height

4.5.1 Manual measurements

Waste height above the slope toe is evaluated from ordinates which were marked at one metre increments along the slope after instrument installation. From this, waste height is estimated during each visit. When estimation of waste level is not clear, additional measurements with measuring tape are carried out. At the time of preparing this paper the waste level had increased relatively slow, reaching a height of 9m measured from the cell toe.

4.5.2 Laser scanning

In order to collect detailed information about the slope, size of applied loadings (*i.e.* the veneer cover thickness), and waste body build up, 3D slope laser scanning (Figure 10) has been carried out. The slope was scanned on 28th September 2009 for the first time, when the length of uncovered slope was 29m. The resolution of the scanning is 1cm. The result of the scanning was compared with the design data of the slope, revealing good correlation (+/-

10cm) of the slope geometry. This was followed by scans conducted after the first veneer experiment in order to record full information about the veneer cover (Figure 11) and also before and after the second veneer trial. Figure 11 presents a profile showing a comparison of the original slope scanned data with the sand veneer upper surface. It is recognised that despite efforts taken to maintain an even thickness, the sand layer thickness varies between ~0.4m and ~0.6m.

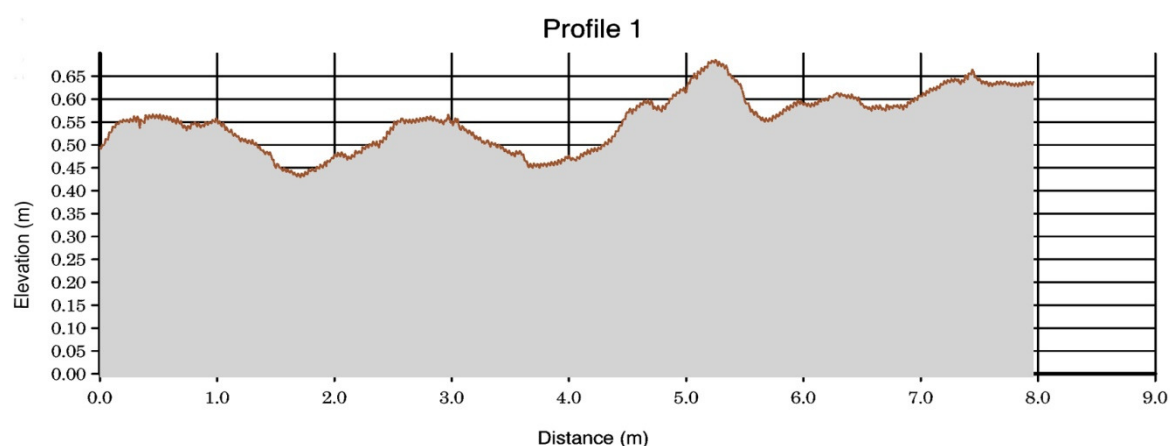**Figure 10. Laser scanning equipment positioned in front of the slope, after the first veneer experiment.**

Figure 11. Sand cover profile for the 1st veneer calculated as the difference between laser scans conducted before and after veneer construction.

5 Instruments resolution and factors affecting measurements accuracy, precision and reading variability

It is common for the terms accuracy, precision and resolution to be mixed up and misused. According to Dunncliff (1993) "**accuracy** is the closeness of approach of a measurement to the true value of the quantity measured. Accuracy is synonymous with degree of correctness. Accuracy of the instrument is evaluated during calibration", while "**precision** is the closeness of approach of each of a number of similar measurements to the arithmetic mean. Precision is synonymous with reproducibility and repeatability" and **resolution** is the smallest unit the measuring device is able to record. Table 4 presents parameters characterising measured values for particular instruments. In terms of strain measurement, as this was one of the most crucial parameters, using three independent methods of measurement adds confidence in the collected information.

Table 4 contains data regarding PC performance. Although the instrument resolution is reported by the manufacturer to be relatively high, it was observed that temperature readings are underestimated or not fully incorporated by the manufacturer within the corrected measurement. According to the instrument manual, correction was not supposed to be applied for PC with thermistors (*i.e.* they were self-calibrating), however after a period of time when the PC were exposed to atmospheric conditions, the readings clearly responded to seasonal temperature changes. Therefore additional analysis was undertaken in order to define a correction factor for temperature. Figure 12 presents temperature recorded by the PC located at the toe of the slope.

Instrument	Accuracy	Resolution	Increment size	Instrument range
Extensometers Displacement	+/-0.5mm	0.5 mm (for displacement)	1mm	1m
Demec strain gauges		0.001 mm	4 micro strains	
Fibre optics[1]	<= 4nm wavelength		1nm	1% long term 5% short term
Pressure cell no thermistor	0.1% of full scale = 1kPa [2]	0.025% of full scale = 0.25kPa [3]		0 to 1000kPa
Pressure cell with thermistor	0.1% of full scale = 0.5kPa[2]	0.025% of full scale = 0.05kPa [3]		0 to 500kPa

[1] According to Fibre Optic Sensing & Sensing Systems – FOS&S datasheet.
[2] Depends on a read out.
[3] Refers to the pressure transducer.

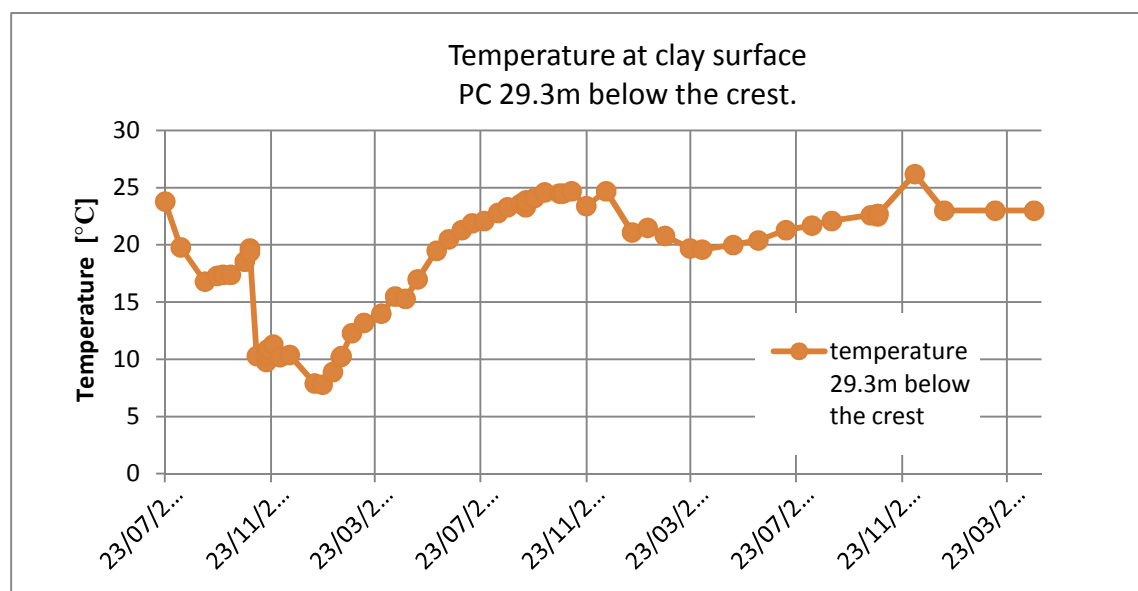**Table 4. Characterisation of instrument readings.**

Figure 12. Temperature by the PC 29.3m below the crest.

6   Factors affecting readings and interpretation

A well-equipped and performed full scale experiment potentially delivers a large amount of data and information. However it requires significant effort to obtain results and it is a complicated and challenging task to analyse and interpret the readings. Consideration has to be given to several aspects of instrument performance and the site environment. The following summarises factors taken into consideration while interpreting results:

- Temperature and weather conditions - during the early stages of waste placement on the slope seasonal changes of temperature have more influence on material behaviour than loadings imposed on the materials. Although the readings from instruments are corrected for temperature changes, the GMB exhibits significant expansion/contraction in response to seasonal temperatures differences and also daily temperature changes;

- Veneer sand layer overburden pressure - controlled by variability of sand layer thickness and sand compaction density;

- Waste body overburden pressure - controlled by waste height, waste type and degree of waste compaction;

- Position of the instruments along the slope in relation to the loaded areas and response of instruments in relation to atmospheric factors;

- Possibility of erroneous readings due to operating error or faulty instrumentation;

- The cell stage of filling, the magnitude of imposed load from the waste compared with later stages of filling when the slope will be fully covered with waste and after site closure;

- FO strain and temperature sensors are very fragile, and although much attention was given to sensor installation to maximise durability, many of the sensors were damaged and became inoperable during the first year of the project. More robust protection could have been provided to the FO cables but a key aim was to minimise the instrument's influence on the lining system performance.

6.1 Instrument and material sensitivity to temperature changes

Temperature was defined as a main factor strongly affecting all of the readings and therefore most of the data is processed to include temperature effects on instrument performance. Table 5 presents values of temperature coefficients applied for particular instrument readings.

Instrument	Temperature sensitivity
Extensometers (wire)	16 µm/m/°C
Demec strain gauges	(readings require HDPE contraction/expansion analysis)
Fibre optics	0.01 nm/°C
Pressure cells	Calculated linear dependency

Table 5. Instruments temperature sensitivity.

6.2 Extensometers temperature sensitivity

Extensometer temperature dependence is relatively complicated to analyse, as the longest wires were partially covered by the sand layer and waste, partially by sand alone and partially just by GTX, therefore most of the time they were exposed to three different temperatures along their length. However, due to a lack of detailed spatial and temporal temperature information for these instruments the correction equation is based on the liner temperature obtained from the thermometer located under the GTX liner and which is not covered by the veneer sand layer. The coefficient of thermal expansion for steel used for the extensometer wires $C_t = 16$ µm/m/°C (Koerner *et.al.* 1997 at Cinnciati landfill applied a factor 17 µm/m/°C for the wire used). The longest wire rope on the slope is 32m (30m along the slope + 2m measuring table). In this case for a large temperature fluctuation, change in wire length can reach values of over 10mm. Table 6 shows the magnitude of contraction/expansion that can occur due to temperature change. The correction equation applied is as follows:

$$C_r = R_s + L_w \times \Delta t \times C_t, \text{ where}$$

C_r - corrected reading;

R_s - original reading;

L_w - wire length;

Δt - liner temperature difference (between first and current reading);

C_t - coefficient of thermal expansion for stainless steel AISI 316G;

Note that when the temperature is below the initial commissioning reading, the correction value is subtracted; however when the current temperature is higher than the initial reading the temperature correction value is added.

point along the slope	wire length [m] l= l_0+2m	Δl [mm] for Δt=1˚C	Δl [mm] for Δt=5˚C	Δl [mm] for Δt=10˚C	Δl [mm] for Δt=20˚C
1	5	0.08	0.4	0.8	1.6
2	10.4	0.1664	0.832	1.664	3.328
3	15.8	0.2528	1.264	2.528	5.056
4	21.2	0.3392	1.696	3.392	6.784
5	26.6	0.4256	2.128	4.256	8.512
6	32	0.512	2.56	5.12	10.24

Table 6. Extensometers wire response to temperature changes.

7 Results from placement of second sand veneer

Monitoring has been carried out at regular intervals since instrument installation, a period of 35 months, three sand veneers have been constructed and the cell is nearly full with waste. This monitoring to date includes evidence of significant influence of temperature on instrument and material performance. The more data that is available the easier it is to observe trends and identify inconsistent behaviour and errors. Therefore in order to establish long-term trends of behaviour significant effort has been given to achieve a full understanding of temperature influences, especially in cases where some of the instruments/materials are exposed to a daily temperature change, while others to only seasonal changes. This work is ongoing.

To demonstrate the type and quality of measured liner behaviour that has been achieved from the field trial, the results from the 2nd veneer sand placement experiment are presented. Only measurements made immediately before, during and on the day following the veneer construction are included. The short time period (two days) and the fact that there was no direct exposure to sun during this period (*i.e.* they were cloudy days), means that the

temperature influence is minimal and can be neglected, as the measured temperature changes during the monitoring period were small. The location of the 2nd sand veneer is shown on Figure 4 and it was constructed in November 2010. During the one day construction period a 10m sand layer, measured parallel to the slope, was placed in five 2m long stages. The thickness of the sand veneer is approximately 0.5m. Instrument readings were taken, before construction started, after each 2m fill stage and the day after.

7.1 Displacements

7.1.1 Geotextile and geomembrane displacements

Measurements from the robust extensometers reveal relatively large movements of both geosynthetic components. Loadings on the GTX are transferred to the GMB hence the displacement detected on the GTX are higher than on the GMB (Figures 13 and 14). Rectangles presented in the charts represent progression of the veneer placement stages, while sensor positions on the slope are schematically presented in the diagrams on a right hand side (Figures 13 – 19). First stage of the veneer placement caused up to 16mm movement within the GTX. Initially the characteristic behaviour involves higher movements within the middle/lower parts of the slope (10-28mm on GTX and 4-18mm on the GMB), and lower within the upper parts of the slope (0-6mm on the GTX, 0-1mm on the GMB). Moreover, some uphill movements are observed within the toe section of the GTX. It is suspected that excess of material might be folding at the slope bottom and this results in up slope displacements of the anchor point. The most significant downhill displacement within both materials is observed the day after construction. Within the middle part of the slope GTX displacements are in the range 8 to 28 mm, while top and bottom are in the order of 9 to 16mm. The next day response of the GMB, have a smaller range with the middle section moving up to 18mm while the crest section stays unchanged and the toe section displaces 4 to 8mm downhill. The lowest GMB extensometer does not change its location.

7.1.2 Geotextile and geomembrane relative displacement

Relative displacement occurring between the GMB and GTX is a calculated value obtained from recorded displacement of the two materials at the same slope position. This is presented in Figure 15. Positive values represent GTX movement exceeding GMB displacement down slope. Negative values indicate higher GMB movement. It is noticeable that the most significant slippage between materials occurs after the first 2m stage of sand placement. In the middle part of the slope relative displacement reaches 20mm. The upper part of the slope follows the same trend but with values reaching 5 mm, while toe section initially reaches values of 11 to 18mm and is followed by decreasing relative displacement to end values of 5 to 8mm of relative displacement. The measured relative displacements are of sufficient magnitude to mobilise post peak values of the GMB/GTX interface strength for the low normal stress conditions (*e.g.* Jones and Dixon 1998). It should be noted that these are incremental relative displacements in response to construction of the 2nd sand veneer and that the measured total relative displacements are larger.
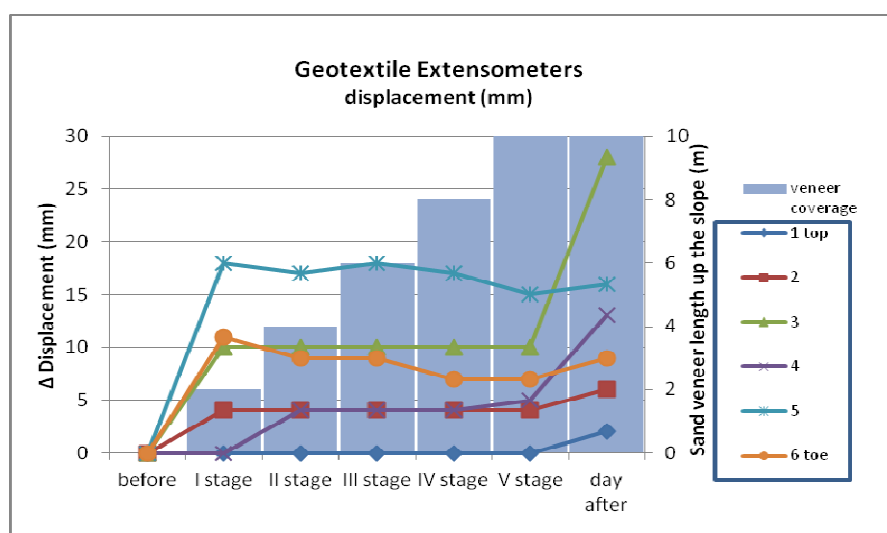

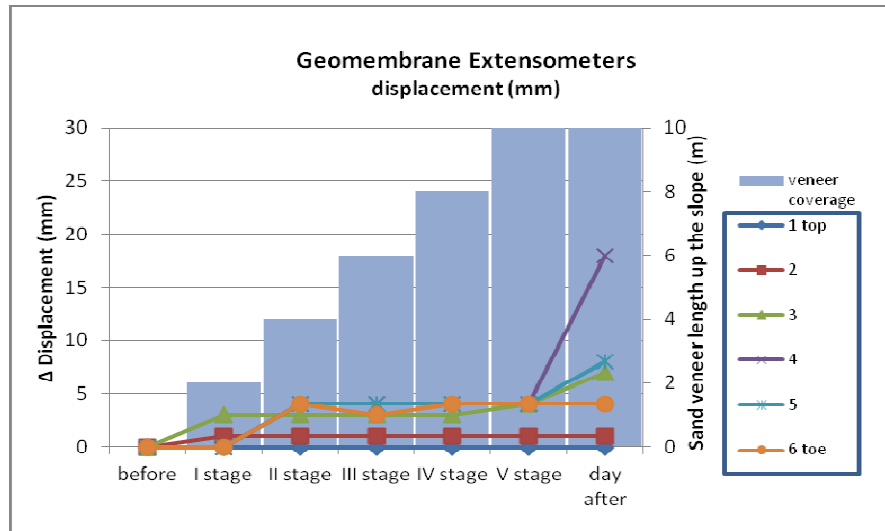**Figure 13. GTX extensometers readings before, during and after the 2nd veneer trial.**

Figure 14. GMB extensometers displacement readings before, during and after the 2nd veneer trial.
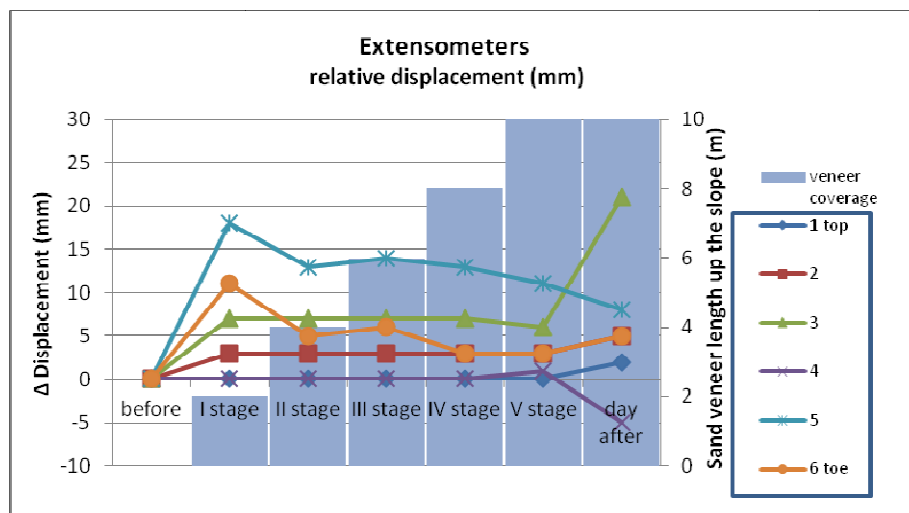

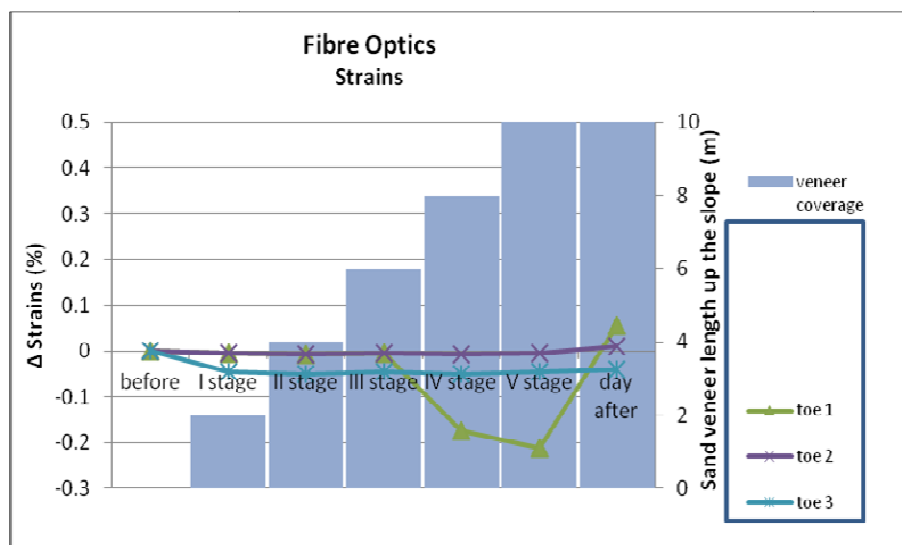**Figure 15. GMB/GTX relative displacement measured before, during and after the 2nd veneer trial.**

Figure 16. GMB FO strains readings before, during and after the 2nd veneer trial
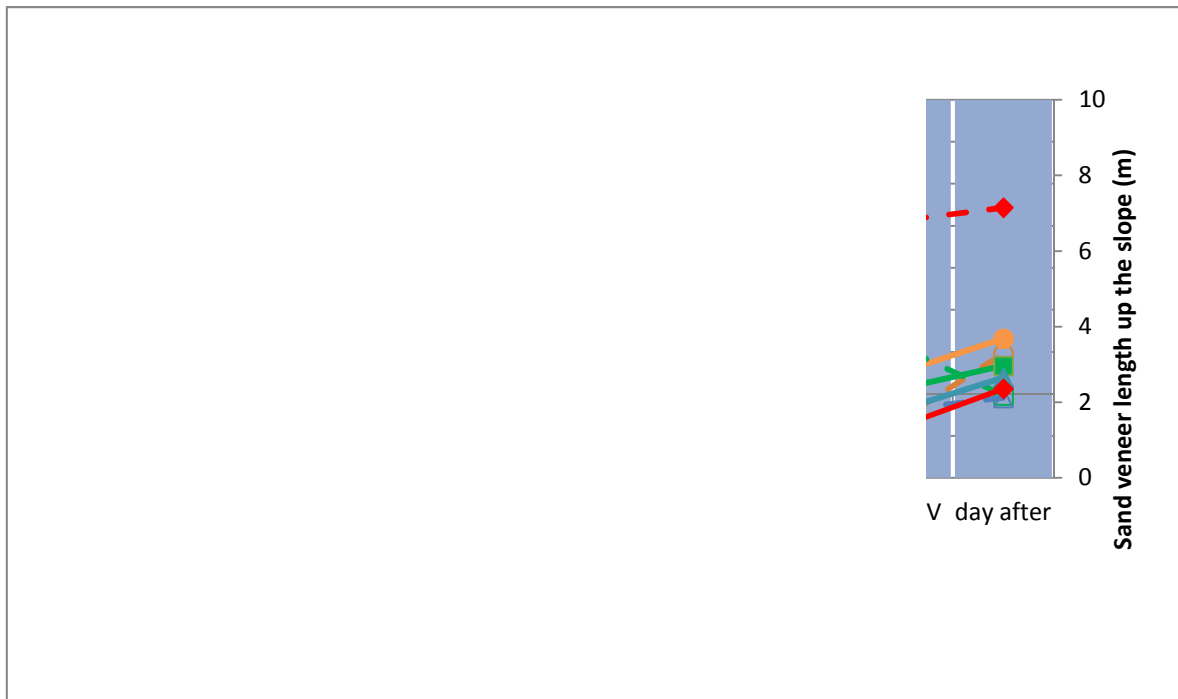
Figure 17. GMB strains – Demec strain gauges readings before, during and after the 2nd veneer trial.
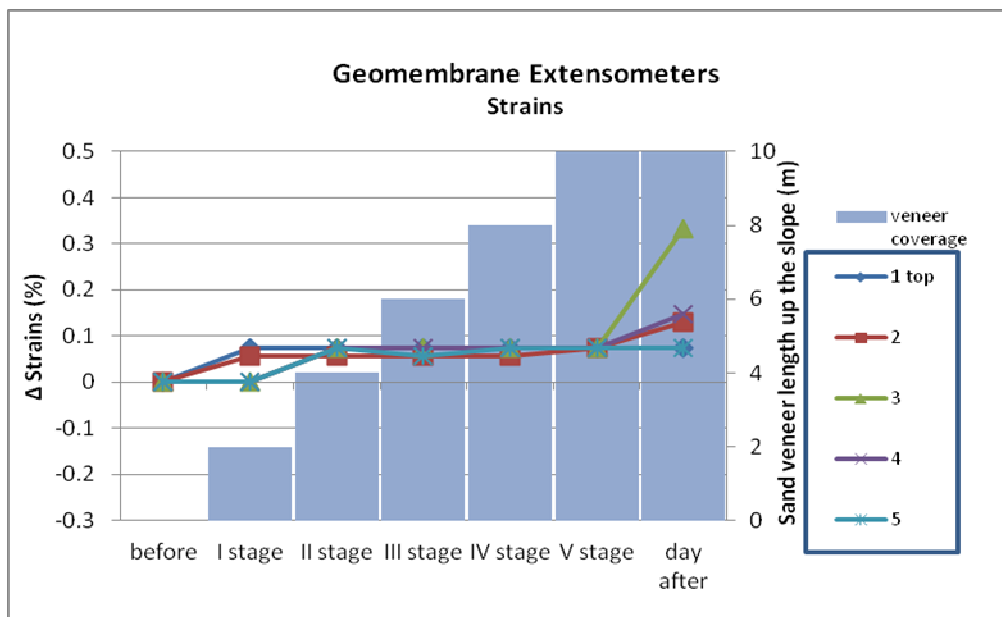

Figure 18. GMB strains calculated from extensometers displacements before, during and after the 2nd veneer trial.
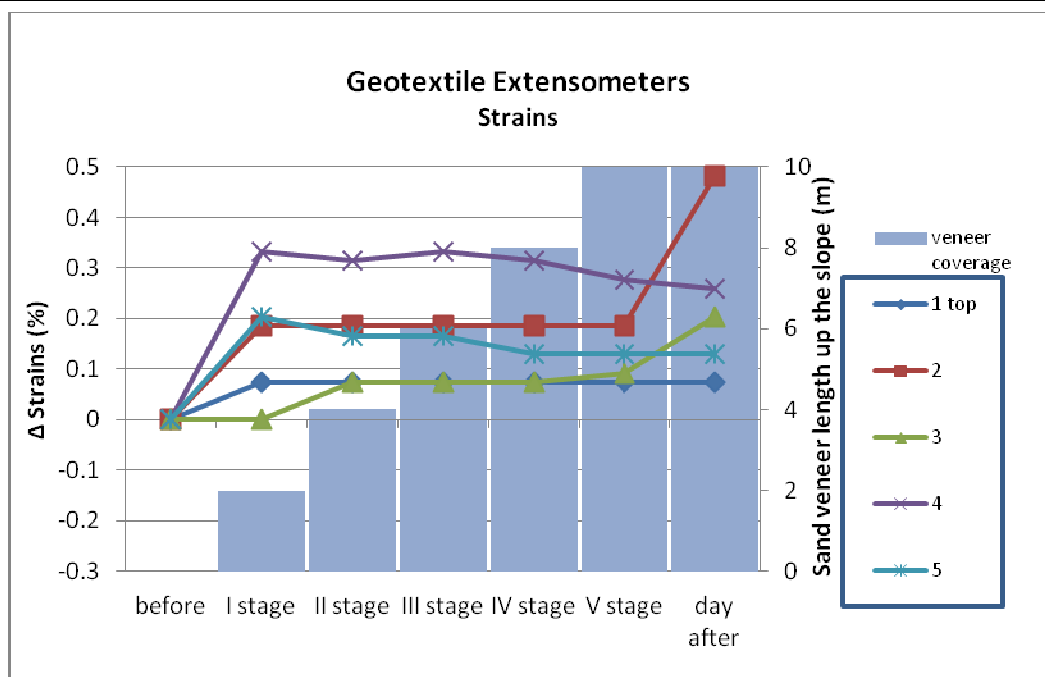
Figure 19. GTX strains calculated from extensometers displacements before, during and after the 2nd veneer trial

7.2 Strains

7.2.1 Fibre optics

FO have the potential to provide very accurate strain measurements. Presented data (Figure 16) include readings recorded within the lower parts of the slope, for sensors installed close to the right hand side edge of the GMB. This part of the slope was already covered by the first sand veneer. In general, low contraction strains are detected near the toe. The 4th stage of veneer placement caused compression measurements for the GMB within the parts of the slope where extensometers detected uphill movements. While the two lower sensors stay within the same range, the top one (25m below the crest) is subjected firstly to relatively high compressive strains ~0.2%, which changes by the next day to extension exceeding 0.25%. The changes in measured strains the day after the veneer construction are considered to occur due to stress relaxation in the geosynthetic and are recorded by all instrumentation.

Particular problems related to the FO strain measurement technology were:

- Periods with lack of response from sensors, although the same procedure was always followed when taking readings (*e.g.* maintaining clean connections and ensuring good attachment to reading device), periods of no readings were observed after which sensors responded again;

- Most of the sensors stopped responding after a year of monitoring. Although at first no particular reason was identified. It was thought that mechanical damage would be limited as the sensors had been protected under the sand layer for many weeks. However, site investigation revealed that the silicon sealant protection installed for the cables was insufficient and the sensors were broken.

7.2.2 Demec strain gauges

The four Demec strain gauges provide interesting data on strains within the GMB, because not only do they allow measurements of strains along the slope but also strains across the slope (Figure 17). Initially it was thought to use horizontal measurements to derive temperature corrections for strains along the GMB. However as the monitoring progressed it was observed that GMB strains across the slope are significant and even exceeding values of strains along the slope in some cases. However for the 2nd veneer experiment strains presented in Figure 17 are not subjected to major temperature changes as noted above. In general, strains for the three upper locations remain throughout the trial in the very low range of +/- 0.07%, similar to the results obtained from FO. Interesting behaviour is observed within the middle section of the slope from the lower measurement position with an immediate high response of horizontal strains reaching 0.2/0.25%. However, placement of the veneer has not resulted in any immediate response in terms of vertical GMB strains at this location, although by the next day tensile strains are again observed. The increases in tensile strains observed

during the 2nd veneer placement are later compensated by material contraction when the temperature dropped during the winter period.

7.2.3 Extensometers strain measurement

An additional source of information on strains is provided by the extensometer measurements. Unlike the FO, extensometers are less delicate instruments, however also less precise and partially subjected to temperature changes as mentioned above, but they cover nearly all the length of the slope and measure strains within the GTX as well as GMB.

(1) Geomembrane strains

The GMB calculated strain values are in the range of values recorded by the FO and Demec strain gauges. Due to precision of the instrument, values are relatively constant and less oscillation within the plot is observed (Figure 18). It is encouraging that similar trends are observed, and recorded values are within the same range as for other strain measurement techniques.

(2) Geotextile strains

The GTX extensometers reveal larger tensile strains comparing to GMB (Figure 19). This is expected as GTX is directly subjected to imposed loadings from the sand veneer and the recorded displacements for GTX were mostly higher, and in addition the material has a lower tensile stiffness. The most noticeable response within the GTX is observed after the 1st stage of sand layer placement. Generally, strains along the GTX exhibit variable values from 0.07 to almost 0.5%. Initially higher values are monitored within the lower sections of the slope, but on the next day the upper section of GTX had also stretched significantly, with an incremental strain of 0.3%.

8 Further Work

The main aim of the ongoing project is to validate numerical models used in landfill design approaches. Future work will comprise modelling the Milegate Landfill slope section in Fast Langrangian Analysis of Continua (FLAC) software, which is a finite difference numerical modelling software and can be used to model multiple strain softening interfaces between components (Fowmes *et al.* 2008) and staged construction. Computed results will be compared with the *in situ* measured behaviour and the design approach evaluated.

To support this stage of the investigation large direct shear box tests have been conducted on multiple samples of materials from Milegate Landfill to establish the interface shear strength properties between the lining system components, including statistical characterisation and input data for the numerical modelling validation. It is planned to continue monitoring until waste placement is complete and, if possible, for an extended period after closure of the landfill. As data becomes available it will be possible to continue interpretation of information from the site and to more clearly define factors influencing behaviour.

9 Conclusions

In the last 20 years much progress has been made in understanding geosynthetic behaviour within landfill lining systems and mechanisms of composite liner performance. In recent years significant attention has been given to developing appropriate modelling of the landfill lining system including characteristic behaviour of materials, stages of construction and advanced waste models, in order to demonstrate acceptable design.

Design process is usually a complicated and time consuming procedure, often supported by many simplifications and assumptions. When numerical modelling software is applied within the design process, it is important to acquire confidence in the computed results. Therefore,

full scale experiments are of crucial importance to establish confidence in design, ensure safety and to allow future design development. The experiment being carried out at Milegate Extension Landfill has provided an opportunity to conduct a full-scale trial with known dimensions, loadings and waste placement conditions. This gives an opportunity to obtain valuable information to aid the design of future landfill lining systems and to assess performance of existing systems. This study aims to obtain information on relative displacements of lining elements, tensile behaviour of geosynthetic elements, loads applied to the liner system and on temperature influences on GMB behaviour during construction stages and after landfill closure.

Data delivered from the three strain measurement instruments gives an opportunity to compare results from independent methods: Demec strain gauges, FO and extensometers. It has to be emphasised that cell filling has progressed slowly, currently the cell is nearly full. Consequently measured pressures imposed on the GMB within lower parts of the slope only reach values of 50kPa. However this has provided an opportunity to establish instrument behaviour and temperature influences on their performance. Many challenges associated with site experiment and data processing have been addressed, including a lack of information about instruments response to temperature changes. As the trial continues, more data that is available that makes it easier to identify and to understand temperature controlled material behaviour and instrumentation response. Considerable effort has been focused on establishing consistent readings and rigorous interpretation criteria.

To date all strain gauge types have provided useful results. Although a few responses are incoherent, readings from all three sources show the same trends and ranges. This provides confidence in collected readings. An important finding is that in the early stages of the cell

filling, the most significant influence on behaviour of the lining system comes from temperature changes (*i.e.* greater than from loading). Significant strains and displacements of the GTX and GMB lining elements have occurred in response to construction of the sand veneers and placement of waste. This loading has also generated relative displacements between the GTX and GMB that are of sufficient magnitude to generate post peak interface shear strengths. This has important implications for the selection of parameters for use in limit equilibrium analyses of waste mass stability against the slope both during construction and in the long term. Measured horizontal strains across the slope provide information on GMB response to both changes in temperature and loading from the soil veneer and waste. It is believed that confidence can be attributed to the results obtained from the Milegate Extension Landfill trial due to the effort paid to instrumentation selection, performance and comparison between methods of measurements.

References

Artieres, O., Beck, Y.L., Fry, J.J., Guidoux, C., Pinettes, P., 2010. Monitoring of earthdams leaks and stability with fibre-optics based monitoring system. 8th ICOLD European Symposium.

Bouthot, M., Blond, E., Fortin, A., Vermeersch, O.G., Quesnel, P., Davidson, S., 2003. Landfill extension using geogrids as reinforcement. Discussion and case study in Quebec *http://www.infogeos.com/view.php?ID=569*

Byrne, R.J., Kendall, J., Brown, S., 1992. Cause and mechanism of failure of Kettlemans Hills Landfill. In: Proceedings of ASCE Conference on Stability and Performance of Slopes and Embankment II, pp. 1–23.

Chang, M.-H., Mitchell, J.K., Seed, Raymond, B., 1999. Model Studies of the 1989 Kettleman Hills Landfil Failure. Geotechnical Testing Journal, Volume: 22, Issue Number: 1.

Council of the European Union (1999). Directive. 1999/31/EC on the landfilling of waste. Official Journal of the European Union, L 182, 1-19.

Crisp,J., Elliot B.J., 2005. Introduction to fiber optics. 3'rd edition, Newnes.

Dixon, N., Jones, D. R., 2003. *Stability of Landfill Lining Systems: Report no.2 Guidance.* Environment Agency Research and Development Technical Report P1-385/TR2.

Dixon, N., Jones, D. R., 2005. Engineering properties of municipal solid waste. *Geotextiles and Geomembranes* 23, pp. 205-233.

Dunnicliff, J., 1993. Geotechnical Instrumentation for Monitoring Field Performance, Whiley-Interscience, New York.

Environment Agency (England and Wales) (2002). Landfill regulatory guidance note 6. Interpretation of the engineering requirements of schedule 2 of the Landfill (England and Wales) Regulations 2002.

Fassett, J.B., Leonardo, G.A., Repetto, P.C., 1994. Geotechnical properties of municipal solid waste and their use in landfill design. Waste Tech'94, Landfill Technology Technical Proceedings, Charlesto, SC (USA), January 13-14.

Fowmes, G.J., Dixon, N., Jones, D.R.V., 2008. Validation of a numerical modelling technique for multilayered geosynthetic landfill lining systems. *Geotextiles and Geomembranes* 26, pp. 109–121.

Gotteland, P., Gourc, J.P., Alboura, A., Thomas, S., 2002. On site determination of geomechanical characteristic of waste. Proceedings GeoEng 2000, The Institute of Engineers, Australia.

Gourc, J.P., Berroir G., Stock R., Begassa P., 1997. Assessment of lining systems behaviour on slopes, Proceedings of *Sardinia 1997, Sixth International Landfill Symposium* , Cagliari, Italy.

Gourc, J.P., Camp, S., Viswanadham, B.V.S., Rajesh, S., 2010. Deformation behaviour of clay cap barriers of hazardous waste containment systems: Full-scale and centrifuge test. Geotextiles and Geomembranes Volume 28, Issue 3, pp. 281-291 IS Kyushu 2007 Special Issue on New Horizons in Earth Reinforcement, 5th International Symposium on Earth Reinforcement.

Jones, D.R.V., Dixon, N. (1998). Shear strength parameters of geomembrane/geotextile interfaces. Geotextiles and Geomembranes vol.16, pp. 45-71.

Kodikara, J., 2000. Analysis of tension development in geomembranes placed on landfill slopes. *Geotextiles and Geomembranes* vol.18, pp. 47-61.

Koerner, R.M., Bowders, J.J., Scranton, H.B., 1997, Instrumentation for Monitoring Field Performance of the Cincinnati GCL Test Plots *Geotextiles and Geomembranes* vol.15, pp. 341-365.

Koerner, R.M., Soong, T-Y. 2000. Stability Assessment of 10 Large Landfill Failures *Geodenver 2000 ASCE SPECIA* vol 103, pp. 1-38.

Loke, K.H., Kongkitkul, W., Tatsuoka, F. 2006. Fibre optic technology in geosynthetic instrumentation for monitoring of soil structures. Proceedings of 8th International Conference on Geosynthetics, Yokohama, Japan 2006.

Machado, S.L., Carvalho, M.F., and Vilaro, M., 2002. Constitutive model for municipal solid waste. Journal of Geotechnical and Geoenvironmental Engineering, 129. (4), pp. 372-378.

Mitchell, J.K., Seed, R.B. and Seed, H.B., 1990. Kettleman Hills Waste Landfill Slope Failure I: Liner-system properties, *Journal of Geotechnical Engineering, ASCE* **116** (4), pp. 647–668.

Muhsiung, C., 2005. Three-dimensional stability analysis of the Kettleman Hills landfill slope failure based on observed sliding-block mechanism. Computers and Geotechnics 32, pp. 587–599.

Najser, J., Masin, D., Bohac, J., 2010. Numerical modelling of lumpy clay landfill. International Journal for Numerical and Analytical Methods in Geomechanics. Published online in Wiley Online Library (wileyonlinelibrary.com). DOI: 10.1002/nag.990

Nakamura, J., Sakanishi, M., Kitazawa, A., Kumagai, K., Fujihashi, K., Ogasawara, T., 2006. *Deformation measurement technology of geomembrane at controlled landfills using optical fiber sensors.* Geosynthetics, Volumes 1-4. Proceedings of the 8th International Conference on Geosynthetics (8ICG), Yokohama, Japan, 18-22 September, 2006, pp. 203-206.

Nancey, A., Lacina, B., Henderson, J., 2007. Geotextile and optc fibres: feedback after four years of use in soil. Geosynthetics 2007 GRI-20 proceedings, January 16-17, Washington D.C.

Rowe, R.K., Hoor, A. 2009. Predicted temperatures and service lives of secondary geomembrane landfill liners. *Geosynthetics International,* 16, No. 2, pp. 71-82.

Rowe, R.K., Islam, M.Z., Hsuan, Y.G., 2008. Leachate chemical composition effects on OIT depletion in an HDPE geomembrane. *Geosynthetics International,* 15 No. 2, pp. 136-151.

Rowe, R.K., Sangam, H.P., 2002. Effects of exposure conditions on the depletion of antioxidants from high-density polyethylene (HDPE) geomembranes. *Canadian Geotechnical Journal* 39, pp. 1221-1230.

Seed, R.B., Mitchell, J.K., Seed H.B., 1990. Kettleman Hills Waste Landfill Slope Failure. II: Stability analysis, *Journal of Geotechnical Engineering, ASCE* 116 (4), pp. 669–691.

Sia, A.H.I., 2007. Landfill Lining Engineering Designs: A probabilistic approach. Unpublished PhD thesis, Loughborough University. November 2007.

Stoltz, G, Gourc, J.P., Oxarango, L, 2010. Characterisation of the physico-mechanical parameters of MSW. Waste Management 30 (2010), pp. 1439–1449.

Thusyanthan, N.I., Madabhushi, S.P.G., Singh, S., 2007. Tension in geomembranes on landfill slopes under static and earthquake loading—Centrifuge study. Geotextiles and Geomembranes Volume 25, Issue 2, April 2007, pp. 78-95 Special Issue on Geosynthetics in Harsh Environments

Villard, P., Gourc, J.P., Feki, N., 1999. Analysis of geosynthetic lining system (GLS) undergoing large deformations. Geotextiles and Geomembranes, Volume 17, Issue 1 February 1999, pp. 17-32.

Viswanadham B.V.S., Rajesh S., 2009. Centrifuge model tests on clay based engineered barriers subjected to differential settlements. Applied Clay Science Volume 42, Issues 3-4, January, pp. 460-472.

Yashima, A., Tsuji, S., Yoshida, K., Yokota, Y., 2009. A new optical fibre sensor to assess the stability of geogrid-reinforced soil walls. *Geosynthetics International,* **16**, No.4, pp. 2238-245.

Zhang, B., 2007. *Constitutive modelling of municipal solid waste.* Ph.D. thesis, Civil and Building Engineering, Loughborough University, United Kingdom.

APPENDIX C PAPER 3

Full Reference

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PAPER TYPE – JOURNAL PAPER

Landfill site slope lining system performance: study case vs numerical modelling analysis

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ABSTRACT: Low permeability engineered landfill barriers often consist of a combination of geosynthetics and mineral layers. Even though numerical modelling software is applied during the landfill design process, a lack of data about mechanical performance of landfill barriers is available to validate and calibrate those models. Instrumentation has been installed on the landfill site to monitor multilayer landfill lining system physical performance. The lining system comprises of a compacted clay layer overlaid by high density polyethelyne geomembrane, geotextile and sand. Data recorded on the site includes: geosynthetics displacements (extensometers), strains (fibre optics, Demec strain gauges, extensometers) and stresses imposed on the liner (pressure cells). In addition, temperature readings were collected by a logger installed at the surface of the geomembrane, at the clay surface (pressure cell thermistors) and air temperature (thermometer). This paper presents readings collected throughout the period of three years, along with the corresponding numerical modelling of the lining system. The significant influence of the effect of temperature on geosynthetics displacement is highlighted. Numerical modelling predictions are compared with the monitored behaviour and challenges with representing *in situ* behaviour of the geosynthetics are discussed, along with additional thermal analysis of the exposed geosynthetics.

1   INTRODUCTION

Geosynthetics are materials that have been widely used in the construction industry for decades. More importantly, they have been recognised as a suitable material for waste barriers and have become extensively applied in landfill engineering. Even though their *in situ* mechanical behaviour has not been fully measured or defined, multiple applications and ease of instalment meet the approval of many government agencies (*e.g.* Environmental Agency (England and Wales), designers and contractors). For over two decades geosynthetic interface shear strength has been a subject of investigations throughout the world. Dixon *et al.* (2006) present data from 76 sources of interfaces commonly deployed in landfills. Furthermore, developed methods of measurement, the procedures and variability in obtainable results are still the subject of many on-going discussions. To consider the complex nature of material behaviour and their interactions, landfill design methods incorporating geosynthetic materials can take the form of limit equilibrium or advanced numerical modelling analyses. The latter are often used for more complicated design cases, where the *in situ* conditions are not favourable and/or serious environmental implications would result from failure. Even though the number of designs based on numerical modelling has increased in recent years, very limited field data on performance exists to allow validation of models in order to confirm their accuracy and suitability. Often model verifications are based on analysis of landfill failures (Koerner *et al.* 2000, Dixon and Jones 2003, Muhsiung 2005) but these cannot deliver data on in service performance of the materials and lining systems such as barrier: displacements, strains or tensile stresses in geosynthetic components resulting from overburden pressures, process of waste placement during landfill cell filling and long term degradation of the waste body. Therefore, lining system stability (ultimate limit state related to large scale movements) and integrity (serviceability limit state - overstressing of liner elements and subsequently loss of original functions) in terms of construction safety, optimal and reliable design (accurate

prediction of imposed stresses, evaluation of strains and axial forces within geosynthetics) are still topics of research.

Since only limited information exists on *in situ* geosynthetic performance in the landfill environment, the need for numerical model validation and calibration is self-evident. Dixon *et al.* (2012) summarises the current state of the research on lining system stability and integrity, and emphasises common engineering problems related to geosynthetics in the landfill environment (*i.e.* staged construction, strain softening interfaces, progressive failure, tensile stresses in materials, representation of waste parameters and behaviour, ageing and waste biodegradation). The purpose of the study reported in the paper was to investigate interface strain softening design issues, as often interfaces between materials installed on landfill slopes (geosynthetics/ geosynthetics, geosynthetics/soil) reveal strain softening character (*i.e.* the interface shear strength value decreases to residual large displacement values after reaching its peak). Studies have been carried out to investigate these phenomena, incorporate these aspects in numerical analyses (*e.g.* Arab 2011, Sia & Dixon 2012, Fowmes *et al.* 2005), however data to verify actual *in situ* interfaces behaviour is still limited.

This paper presents results from a three year full scale investigation of mechanical performance of a multi-layered landfill lining system, carried out at the Milegate Extension Landfill. The lining system comprises a compacted clay layer, overlaid by geomembrane, geotextile and a sand layer. The project started in June 2009 and monitoring was carried out for the following 3 years. Instrumentation installed on the site consists of pressure cells (PC), extensometers (Ext), fibre optic strain gauges (FO), Demec strain gauges (DSG) and additionally thermometers.

This paper aims to provide improved understanding of lining system *in situ* behaviour and to highlight factors that influence interface mobilised strength and geosynthetic strains. A numerical model representing the Milegate slope configuration and construction sequence was created to satisfy the second aim of the project, which is to validate and calibrate the numerical modelling design approach. Numerical analyses were undertaken using FLAC software and the results compared with the measured *in situ* behaviour of the lining system materials. Analyses were carried out to replicate common design conditions including staged construction, a multiple mineral/geosynthetic lining system with associated multiple strain softening interfaces, and waste body compression under self-weight due to low stiffness.

2   MILEGATE EXTENSION LANDFILL STUDY CASE

Details regarding the project such as: slope geometry, instrumentation and its performance, installed lining materials and history of construction works undertaken on the site are reported by Zamara *et al.* (2012). Only a brief description of the main aspects of the site works is reported below.

2.1 The trial site

The monitored slope has a length of 31.2 m and height of 16 m with an inclination angle of 1v:2.5h (~21.8°). Figure 1 shows the site location, slope geometry and photographs of initial stage and one of the final waste placement stages. The lining system deployed during the experiment was placed in addition to the existing-clay liner, and therefore is an additional and hence sacrificial layer that does not form part of the approved containment system. The combination of materials forming the lining system was: clay, geomembrane, geotextile and a veneer of soil, which were chosen to represent common practice in the United Kingdom.
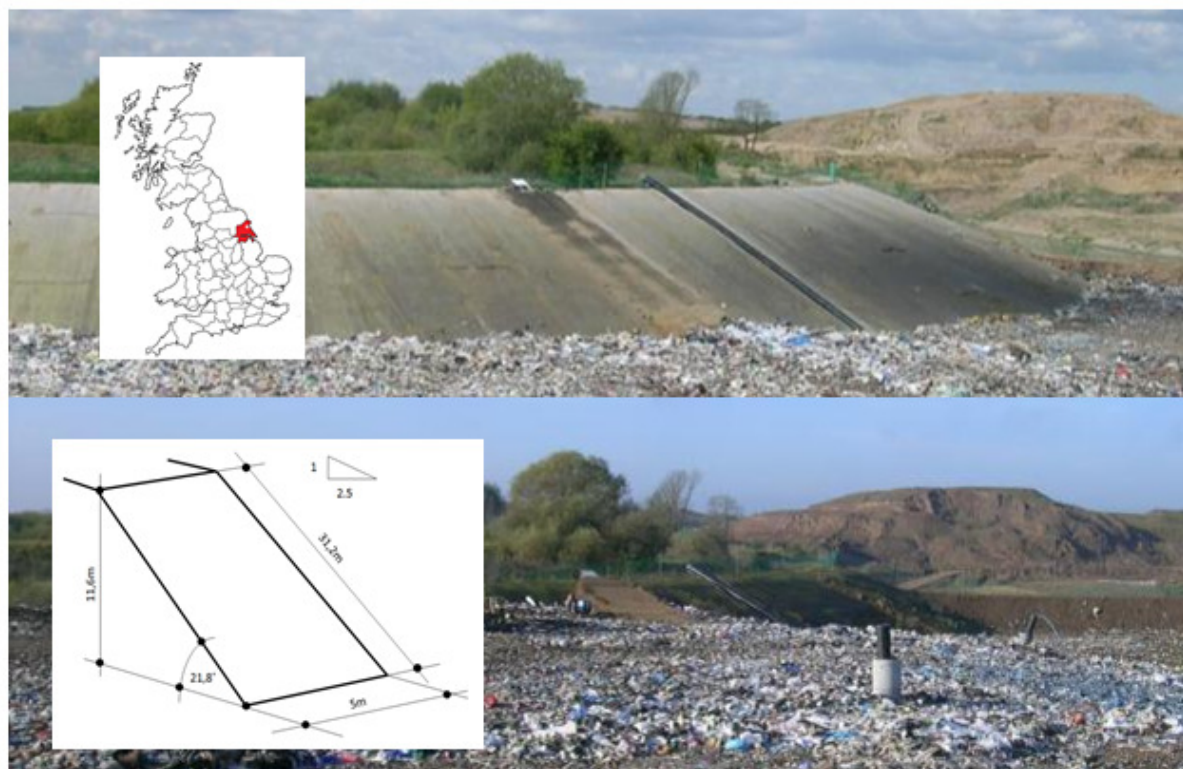
Figure 1. Milegate Extension Landfill – the slope view 2010 and 2012, location of the slope and basic geometry (after Zamara *et al.* 2012).

2.2 Materials

The instrumented lining system comprised of a 2 mm double textured HDPE geomembrane 5m wide panel, with density of 0.949 g/cm^3 (GM TMT from Atarfil S.L.) placed on top of the compacted 1.0m thick clay layer with a maximum permeability of 1×10^{-9} m/s . The geomembrane is overlain by a non-woven needle punched geotextile 5m wide panel. This protection layer has a static puncture strength [CBR] of 14 kN, thickness of 7.8mm and weight of1400 g/m^2 (HPS14 from GeoFabrics Ltd.). The multilayered landfill system is shown schematically in Figure 2. The geomembrane and geotextile were anchored in a "U" shaped 600mm x 600mm anchor trench at the top of the slope. The geomembrane/geotextile experimental panel replaced the existing geocomposite drainage material over a slope width of 5.0m. A nominally 0.5m thick sand veneer was placed in stages on the geotextile ahead of waste placement. This represents common practice of providing a mineral drainage layer on

side slopes. Prior to waste placement the sand layer was placed in lifts parallel to the slope along 10m of slope length. When the waste body reached the top of the first veneer a second 10m sand layer measured parallel to the slope was placed along the slope. When the waste reached the top of the second veneer layer a third and last veneer lift was constructed and the whole length of the slope was then covered with a 0.5m thick sand layer. Placement of the sand veneers provides opportunity to measure response of the underlying geosynthetic components to the applied load. In practice sand is not used for drainage layers due to its fine grading and hence low permeability but it was used in this study to produce loading equivalent to the medium to coarse sand typically used for mineral drainage layers.
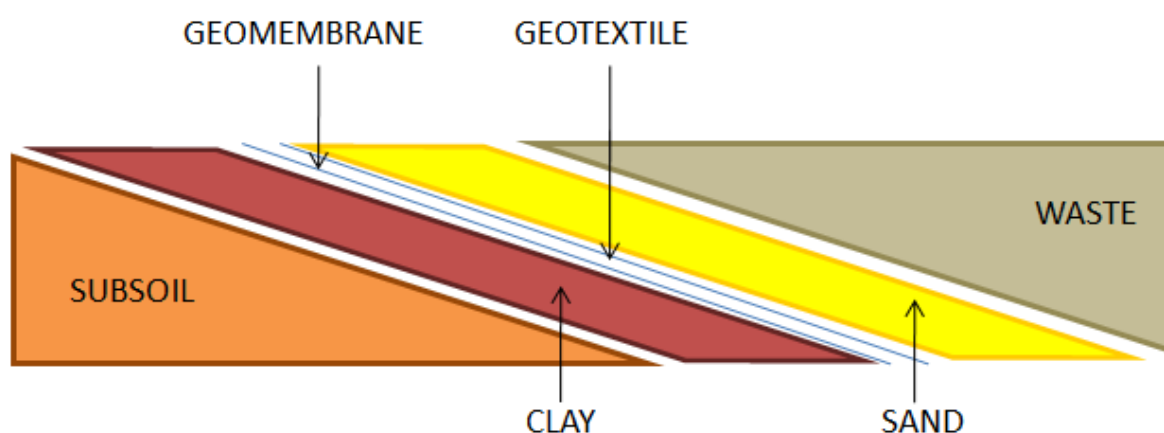

Figure 2. Multilayered side slope lining system – schematic view.

2.3 Summary of instrumentation

The instrumentation was designed to measure parameters which are the most important for the design process and hence long term performance of the lining system. The instrumentation delivers information about stresses imposed on the liner (three PC along the slope at the clay/geomembrane interface), displacements of the geosynthetic liner elements and relative displacement between the liner elements (extensometers located on the geomembrane and geotextile), strains on the geomembrane (DSG, FO, calculated from extensometers) and

geotextile strains (calculated from extensometers). Figure 3 presents the schematic location of the instruments along the slope. Table 1 details the type, number and measured parameters of instrumentation installed on the site.
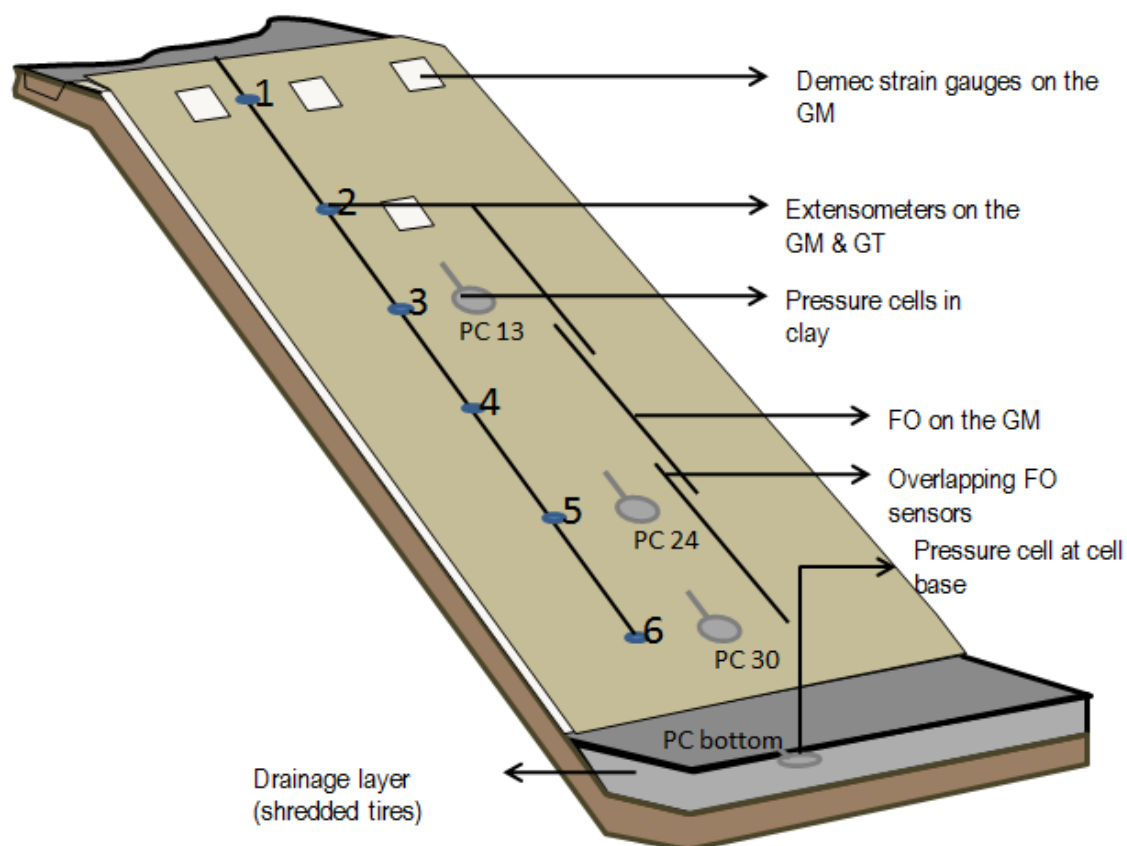

Figure 3. Instruments location along the slope – schematic view (after Zamara _et al._ 2012).

Instrument	Number of instruments/ sensors	Measured parameter
Vibrating Wire Pressure Cells	4	Normal Stress
Extensometers	12	Displacement (at 6 points on the GM, 6 on the GT)
Demec strain gauges	16 steel disks	GM strains across the slope
		GM strains along the slope
Fibre Optics	15	GM strains along the slope
	7	GM temperature
Thermistors	2	Clay surface temperature
Additional records		Waste height
Temperature logger	1	GM temperature

Table 1. Instruments and parameters measured at Milegate Extension Landfill.

3 MILEGATE EXTENSION LANDFILL NUMERICAL MODELLING

One of the main aims of the study was to validate the numerical modelling results against measured *in situ* behaviour of the lining system. The cell where the monitored slope is located was due to be filled with waste within 1-2 years after the instrumentation installation, however this process was delayed due to the current economic situation, which resulted in slower filling rate and hence prolonged exposure of the geosynthetics to atmospheric conditions.

3.1 Finite Difference Computer Software

A commercial software program FLAC (Itasca International Inc.) has been used to compute predicted behaviour of materials and interfaces on the monitored landfill slope. FLAC has been used in several previous landfill geotechnical engineering studies (*e.g.* Fowmes *et al.* 2005, Arab 2011, Sia & Dixon 2012). The code allows materials that undergo large strains to be modelled, hence could be used in landfill applications. It can represent waste body deformation, interface displacement and geosynthetics strains. FLAC analyses reported in this paper were based on a landfill design procedure developed by Fowmes *et al.* (2007).

3.2 The Landfill Model Geometry - general

The model was built in a way to represent directly all of the major aspects if the cell construction and filling process. The cell was formed from a clay layer modelled at the cell slope and base. The model was built taking into consideration staged construction: each sand veneer stage (0.5m thick sand veneer was placed in 3 lifts, in 10m long layers measured parallel to the slope), followed by 4 waste lifts. In total the model computes 16 stages of material placement (1st clay, 2nd sand veneer, 3-6th waste lifts, 7th sand veneer, 8-11th waste lifts, 12th sand veneer, 13-16th waste lifts). Each waste lift has a vertical thickness of approximately 1m.

3.3 Multilayer lining system

Soil and waste materials where represented by Mohr –Coulomb failure criterion and the properties assigned to the materials are given in Table 2. Waste properties are based on data available from literature (Jones & Dixon 2005).

The geosynthetics lining elements were placed along the clay slope. Since the *in situ* material comprised of well compacted clay over a strata with high strength properties, clay behaviour was not monitored and for the modelling approach it was considered to produce an insignificant affect on the lining system deformations. It should be noted that initially high stiffness values were assumed for the clay as no movement was expected within the compacted clay layer (Table 3, simulation FLAC_min), however, in further sensitivity analyses the clay stiffness was reduced to investigate the influence on geomembrane displacements (Table 3, simulation FLAC_max). Geosynthetics were modelled as beam elastic elements anchored at the top of the slope. Three interfaces between lining components were assigned: clay/geomembrane, geomembrane/geotextile, geotextile/sand, and additionally the sand/waste interface were given waste properties. Information on geosynthetic tensile behaviour was provided by the suppliers: geomembrane thickness is 2mm and Young secant modulus E= 338 MPa (for 5% strain), geotextile thickness is 7.8mm and Young modulus E=120 MPa (for 5% strain). Geosynthetics were not expected to fail due to excessive deformations, therefore using secant modulus values for 5% strain was used to generate results.

Material	Model	Density (Mg/m³)	φ' (°)	c' (kPa)	Young's Modulus (MPa)	Poisson's Ratio
Waste	Mohr - Coulomb	1.0	25	5	0.5	0.3
Sand layer	Mohr - Coulomb	1.7	35	0	70/20	0.4
Clay liner	Mohr - Coulomb	2.75*	23	5	150/50	0.3

Table 2. Materials properties for FLAC model analysis.

Interface	δ (°) Peak/Residual	α (kPa) Peak/Residual	Normal stiffness (kPa/m)	Shear stiffness (kPa/m)
Waste/sand	20	5	10000	5000
Sand/HPS dry wet	29.9/29.6 29.6/29.9	6.3/1.8 3.2/1.3	10000	4500
HPS/HDPE dry wet	19.9/13.3 20.8/14.7	2.3/1.4 4.0/2.9	10000	4500
HDPE/Clay drained undrained	22.0/22.0 31.1/25.1	8.0/8.0 7.6/3.2	10000	5500

Table 3. Interface properties used in FLAC model.

3.4 Interfaces

The importance of the interface strength parameters has been emphasised previously by various authors (*e.g.* Filz *et al.* 2001, Jones & Dixon 2005). In general it is accepted that landfill side slope lining systems might undergo interface shear strength softening behaviour. The Milegate case modelled in FLAC allows the interface between elements to be modelled. Interface shear strengths that were measured in a direct shear box machine in a laboratory test programme were used in the model and are presented in Table 3. For each interface tests were carried out with five different normal stresses: 10, 25, 50, 100 and 200 kPa. In order to acquire detailed information on the interfaces and define the weakest, tests were carried out with the following conditions: dry interfaces (soil/geotextile, geotextile/geomembrane, geomembrane/clay), submerged interfaces (soil/geotextile, geotextile/geomembrane), slow

displacement rate in an attempt to reflect drained conditions (geomembrane /clay). Each test was repeated at least three times with a new sample. Waste/soil interface properties were not investigated in the laboratory and the values used are based on the common approach of assigning the waste material properties to its interface with the granular drainage layer. In addition, strain softening behaviour of interfaces was incorporated in the model following the approach developed by Fowmes *et al.* (2007).

3.5 The modelling process

The most comprehensive and consistent site data was delivered from the extensometer measurements of geomembrane and geotextile displacements, therefore the first attempts to compare numerical modelling outputs with the site data were initially focused on geosynthetic displacements.

The numerical model was developed in an iterative process; gradually sub-procedures were added to the basic model and examined in terms of generated geosynthetic displacements during construction and waste placement. In the study four different interfaces shear strength properties scenarios were investigated: peak, residual, strain softening and reduced values. Additionally stiffness of the clay liner, geotextile and sand was altered in an attempt to reproduce monitored values. Values reduction was to represent materials/ interfaces degradation due to weathering.

4   COMPARISON OF MEASURED AND MODELLED BEHAVIOUR

4.1 Stresses imposed on the liner

Computed values of pressures imposed on the liner, are within the ranges recorded on the site using pressure cells (Figure 4). It can be concluded that stresses imposed on the side slope lining system, from waste unit weight and sand veneer can be represented by the numerical model. Time has not been explicitly considered in the numerical analysis but stages of

construction are defined. The reference point for the plotted, measured and modelled values is the waste height above the landfill base. The site records were plotted against time to provide the time framework for this study and to present the cell filling time-scale.

The two lower PCs include thermistors measuring temperature at the clay surface (Figure 3 PC24, PC30) which allows temperature correction of PC reading, which are plotted in Figure 4. It can be noticed that once the PCs were covered with waste, temperatures on the clay surface experience significantly less variation, and winter clay temperatures did not decreased significantly from summer values.

4.2 Extensometers readings vs FLAC prediction

Figure 5 presents an overview of all the displacements recorded by Exts through the three year period. Additionally, Table 4 summarises peak displacements of the geomembrane and geotextile computed in FLAC for various configurations of in-put parameters, also for comparison displacement values recorded on the site are included in the table.

4.2.1 Geotextile

a) Site data

Significant movements of geotextile are recorded on the site. Ext. located within the middle and upper parts on the slope indicated increased movements up to 80mm down the slope. These large movements were triggered mostly at the time of the second sand veneer placement. It was observed that site derived data presents less consistent (along the slope) and more localised displacements than the computed simulations. Loaded and weathered geotextile due to stretching does not act as a regular beam (as modelled), but the movement proceeds in sections (Figure 5).

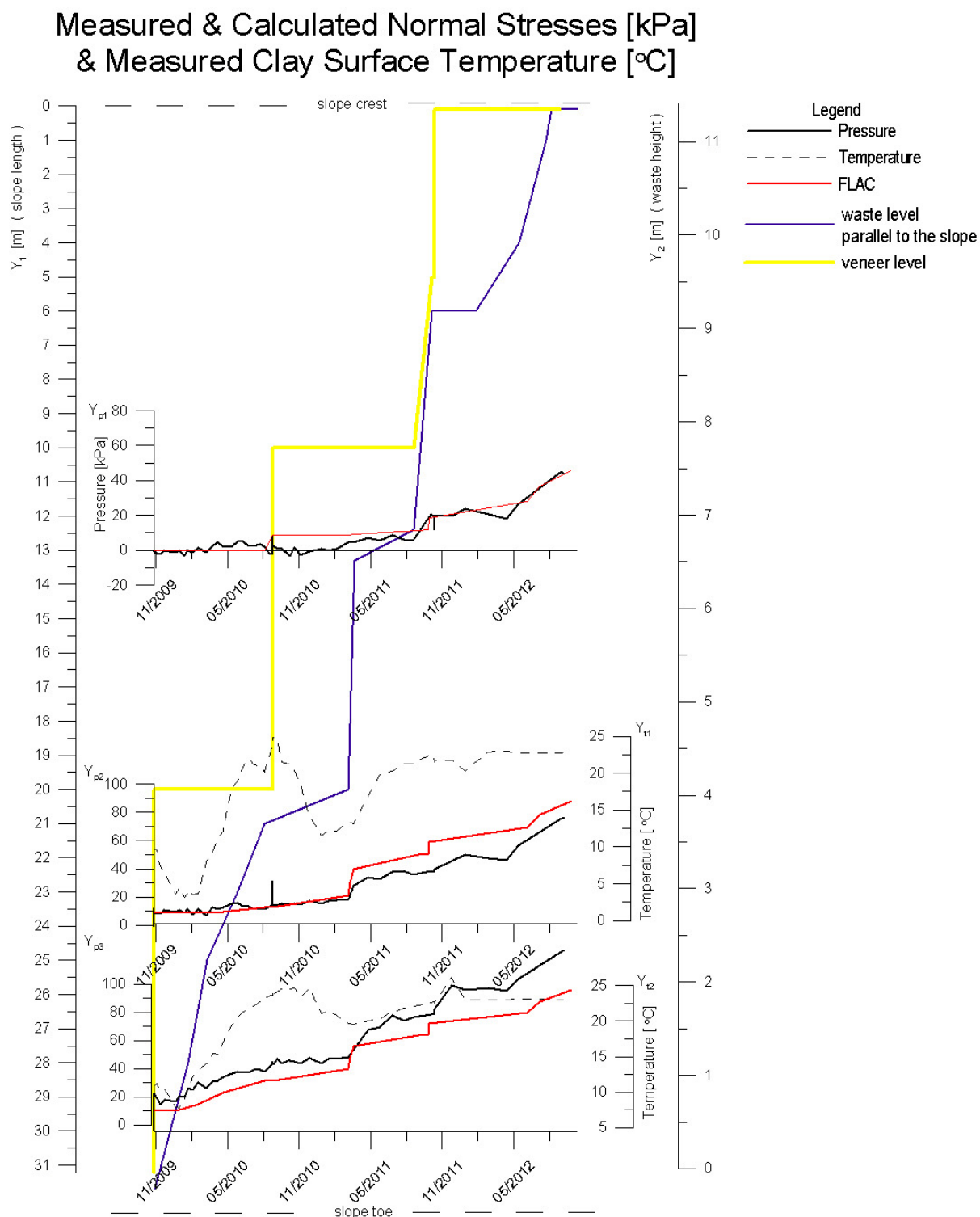# Measured & Calculated Normal Stresses [kPa] & Measured Clay Surface Temperature [°C]

Figure 4. Measured pressures (pressure cells) vs computed values (pressures imposed on the liners). All the results measured and computed are plotted correspondingly to the on-site instruments locations (left hand Y-axis represents slope length from the crest to the toe (0-31.2m), X- time axis are located in the relevant site instruments locations along the slope, right hand Y axis – represents waste height above toe with the corresponding plot of the slope waste coverage).

Analysis ID	IN-PUT VALUES COMBINATIONS			RESULTS		
	Applied interface shear strength	Clay Stiffness [MPa]	Soil/GTX Stiffness [MPa]	GTX Max Displacements [mm]	GMB Max Displacements [mm]	GMB Max Strains [%]
1_1 (MIN)	Peak	150	70/1.2	**30.5**	**20.1**	0.13
1_2 (MIN)	Residual			30.5	20.1	0.13
1_3	Strain Softening			35.3	20.7	0.18
1_4	Reduced values			45.5	20.4	0.15
2_1	Peak		20/1.8	30.9	20.3	0.13
2_2	Residual			31.0	20.3	0.13
2_3	Strain Softening			35.8	21.4	0.18
2_4	Reduced values			76.5	20.6	0.14
3_1	Peak	50	70/1.2	48.7	39.1	0.28
3_2	Residual			48.4	38.8	0.28
3_3	Strain Softening			50.7	38.2	0.27
3_4	Reduced values			70.0	39.9	0.27
4_1	Peak		20/1.8	50.1	40.2	0.28
4_2	Residual			50.4	40.3	0.28
4_3	Strain Softening			52.4	39.7	0.27
4_4 (MAX)	Reduced values			**92.0**	**40.9**	0.26
Measuring instruments			**Monitored values**			
Extensometers readings			**83.5**	**25.9**	0.26	
DG slope direction*			-	-	0.31	
DG across slope direction*			-	-	0.42	
Fibre optics readings*			-	-	0.78	

Table 4. Milegate readings /FLAC modelling summary.

b) Site data vs FLAC

In the presented graphs (Figure 5-8), simulation results FLAC_MIN refers to analysis presenting the lowest displacement of the lining components. These simulations were undertaken on the basic model with peak and residual interface shear strength properties. FLAC_MAX refers to analysis with degraded interface shear strength properties (lowest

Geomembrane & Geotextile Extensometers
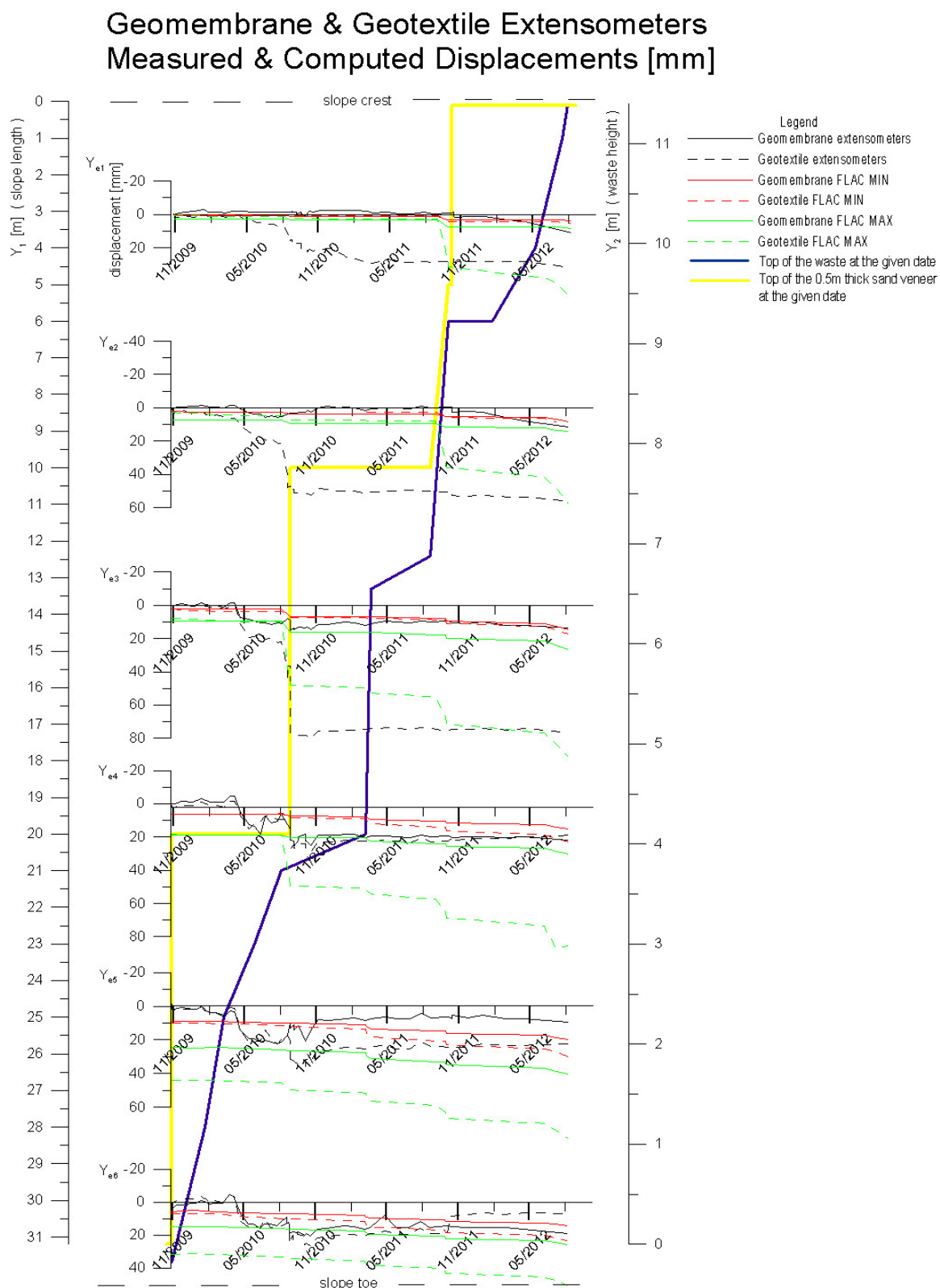## Measured & Computed Displacements [mm]

Figure 5. Extensometers readings – Milegate Extension Landfill (the plots are presented in the same manner as PC plots - Ext locations (X-axis) on the slope are reflected on the left hand side axis, which represents slope length starting from the crest measuring parallel to the slope. Each Ext couple (GMB and GTX) displacement is plotted on a relevant X- axis, with the same direction of the movement (up or down the slope) as it occurred on the slope).

credible values) and altered material stiffness. For this condition the computed displacements are the highest.

The numerical modelling results using the basic approach produced limited agreement with the measured behaviour especially in terms of geotextile displacements (plot FLAC_MIN, Figure 5). In general for the standard approach it was impossible to replicate geotextile movements in the middle/top section of the slope. The largest movement in the model occurred within the toe section. It can be noticed that geotextile do not behave in a manner as computed in FLAC. FLAC analysis plots are highly regular, with predictable trends, which increase steady until the final stage of loading. On the site no significant movements have occurred once the slope was covered by the second sand veneer. Furthermore, results for the geotextile are in a relatively good agreement for the section of the slope where the geotextile has been covered by sand veneer - being not exposed for extended periods. Although behaviour during staged construction is not well replicated, the final total displacements monitored at the lower section of the slope are within the computed range of 30mm. For the top section of the slope, geotextile *in situ* displacements are comparable to the computed ranges only when the values of interface shear strength of the lining components are reduced significantly (FLAC_MAX analysis), when geomembrane movement is increased by reduction of the clay stiffness and the geotextile stiffness increased (due to weathering after Lodi *et al.* 2008). The analysis FLAC_MAX represents relatively well the measured behaviour of the exposed geotextile, between placement of sand veneers (first and second), however displacements within the lower sections are significantly overestimated. Figure 6a-c present plots of selected FLAC analyses versus site derived data, for subsequent stages of the cell filling for three locations on the slope (extensometers locations: 8.4, 13.8 and 24.6 m below the crest). The main movement for the upper section occurred directly before and due

to the second sand veneer placement. In FLAC_MAX analysis geotextile movements occurs only when the loading is placed directly over the geotextile: top Ext locations (8.4m) – III veneer (Figure 7a), middle Ext (13.8m) – II veneer (Figures 7b), toe Ext (24.6m) – I veneer (Figures 7c) location, while the highest displacements on the site occurred prior to II veneer placement and directly in response to the II sand veneer placement.

It was not possible to represent geotextile behaviour in the above analysis. It was considered that most geotextile displacement might have occurred due to geomembrane thermal behaviour that is not directly modelled in FLAC, and therefore additional simulations were undertaken to reflect weather influence conditions on the site. These did not represent the actual thermal conditions on the site, but aimed to simulate behaviour using reduced values of interface shear strength, increased geotextile stiffness, reduced sand stiffness and reduced clay stiffness.

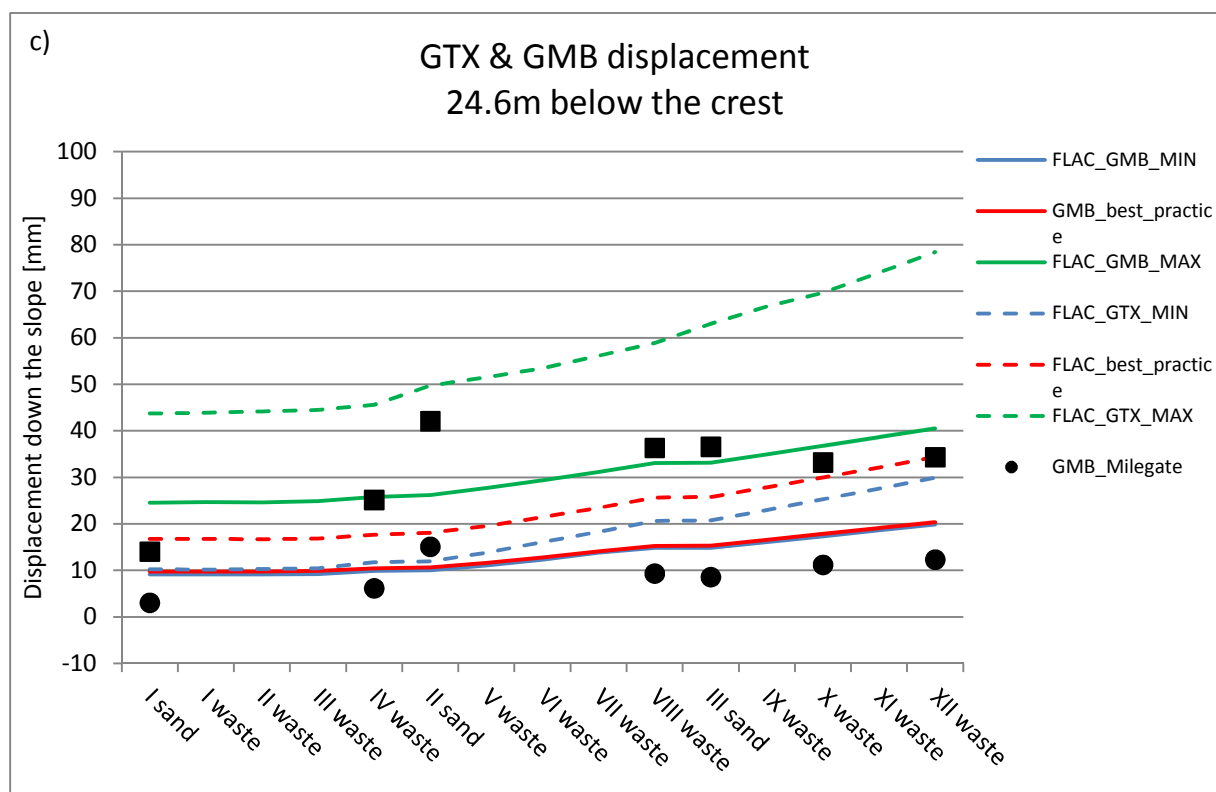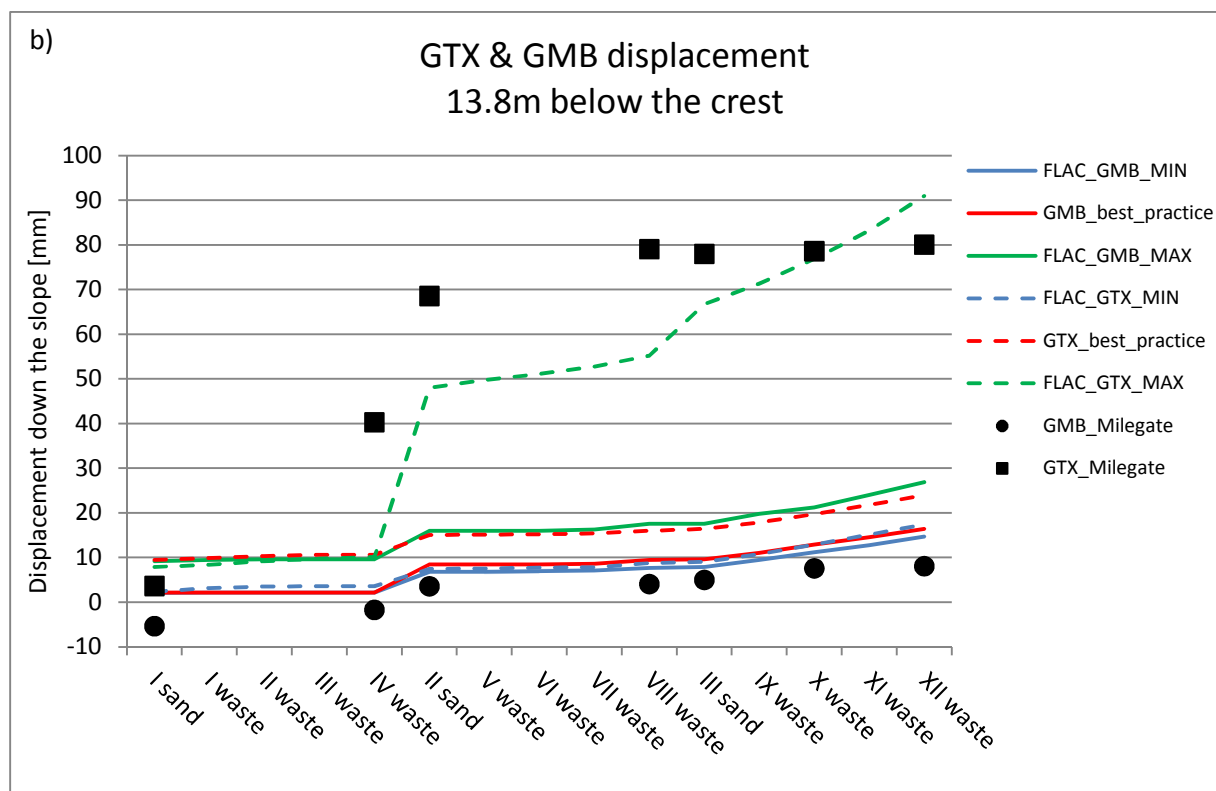a) GTX & GMB displacement
8.4m below the crest
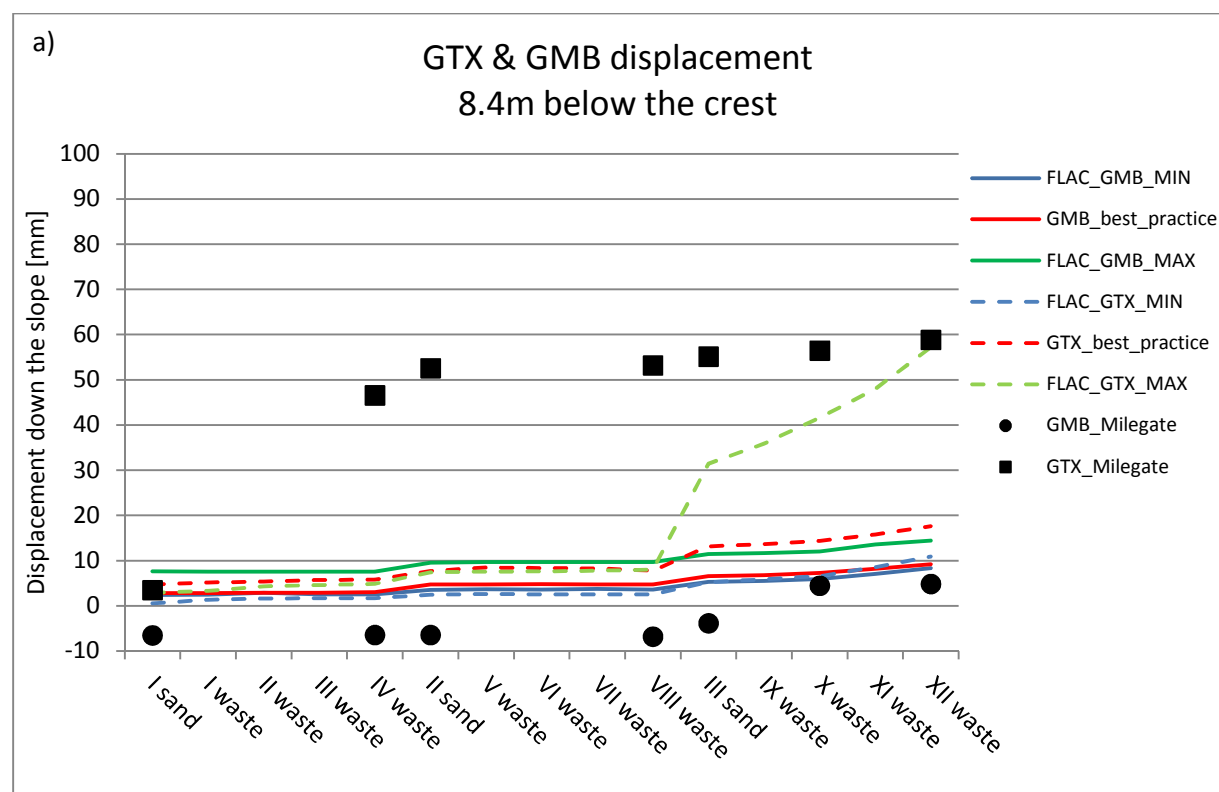
Figure 6. Extensometers displacement readings vs selected FLAC displacements prediction on the GMB for the staged construction at the following locations: a) 8.4m below the crest, b) 13.8m below the crest, c) 24.6m below the crest.
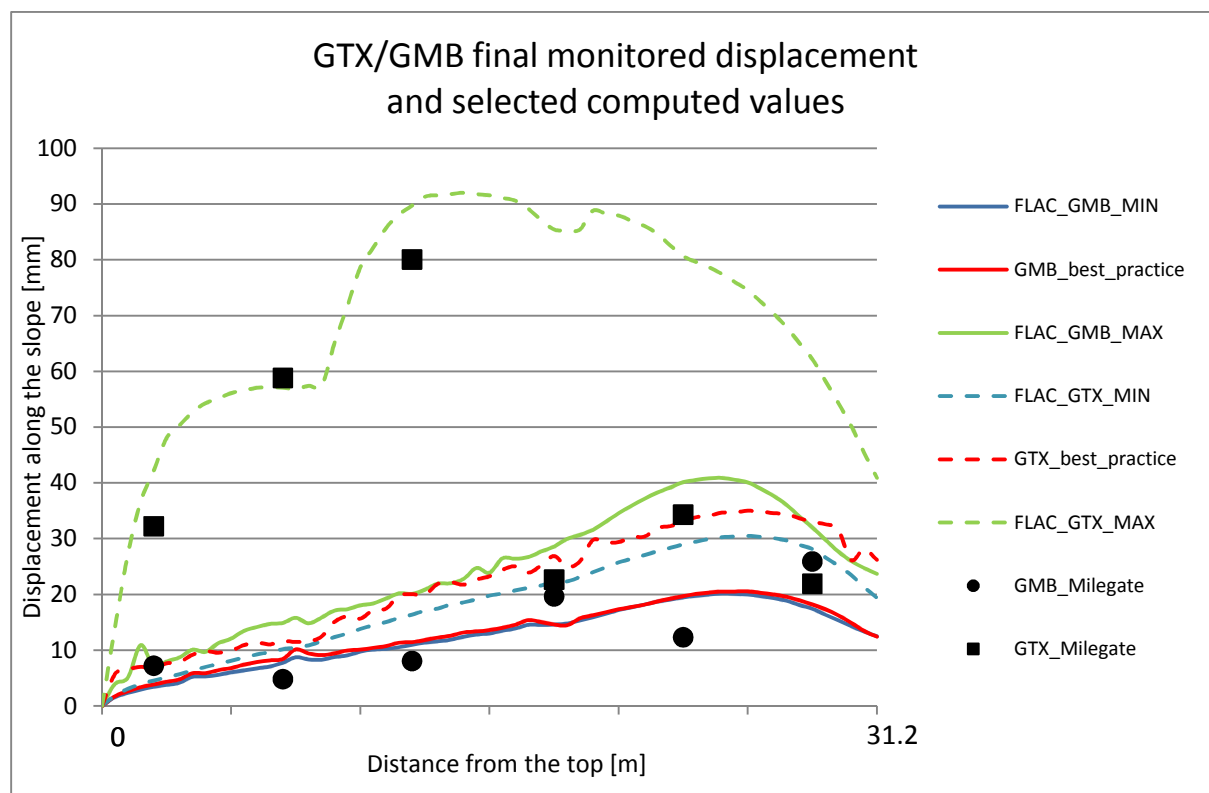
Figure 7. Displacements for the final stages of the computed analysis – min/max computed results and final readings at Milegate Landfill.

4.2.2 Geomembrane

a) Site data

Geomembrane displacements gradually increase during filling at all locations in different ranges, with only limited displacement caused directly by the veneer placements. Maximum displacement reaches 25.9mm within the middle section, but in general extensometers recorded displacements of 10-20mm within the geomembrane panel. No significant correlation was defined between geotextile and geomembrane increased displacement locations.

b) Site data vs FLAC

Geomembrane overall movement is represented in FLAC with relatively good agreement. For analysis FLAC_MIN (Figure 7) geomembrane movement is partially underestimated (lower section) and partially overestimated (higher sections). Additionally for the given clay

conditions, not much difference was observed between the results for the peak, residual, strain-softening and reduced interface shear strength. For the geomembrane most movement was caused by the clay stiffness reduction (Figure7) that represents the highest displacements, but overestimates the overall geomembrane performance.

4.2.3 Summary

Figure 7 presents displacements distribution of geotextile and geomembrane along the slope, for the final stage of construction. These plots highlight the significant differences in computed geosynthetics and measured final displacements distributions. The computed maximum geomembrane displacements for the basic FLAC analysis (FLAC_MIN) are in the range of the monitored values but the locations of the maximal geomembrane movements are different. In terms of staged construction (Figures 6a-c) it can be observed that basic analysis is sufficient to replicate the geomembrane behaviour within the upper slope section, however for all instrumented locations the geotextile movements are underestimated by basic analysis.

Simulations with peak and residual interface shear strength gave very similar results. For the basic simulation (FLAC_MIN), the maximal computed geomembrane displacement is in range of monitored geomembrane displacements – 20mm (although the location of its occurrence is not well represented in FLAC). This is consistent with the expectation that mobilised strengths are within peak ranges for a shallow slopes such as investigated in this study.
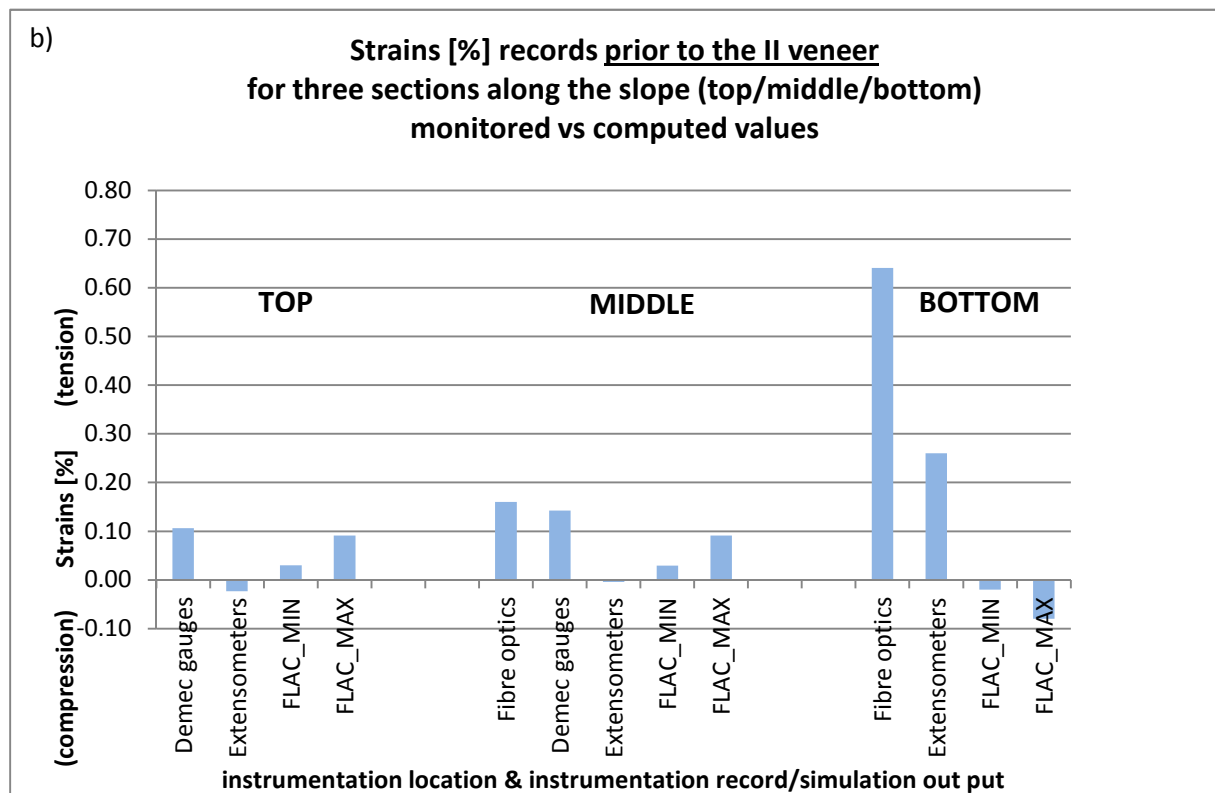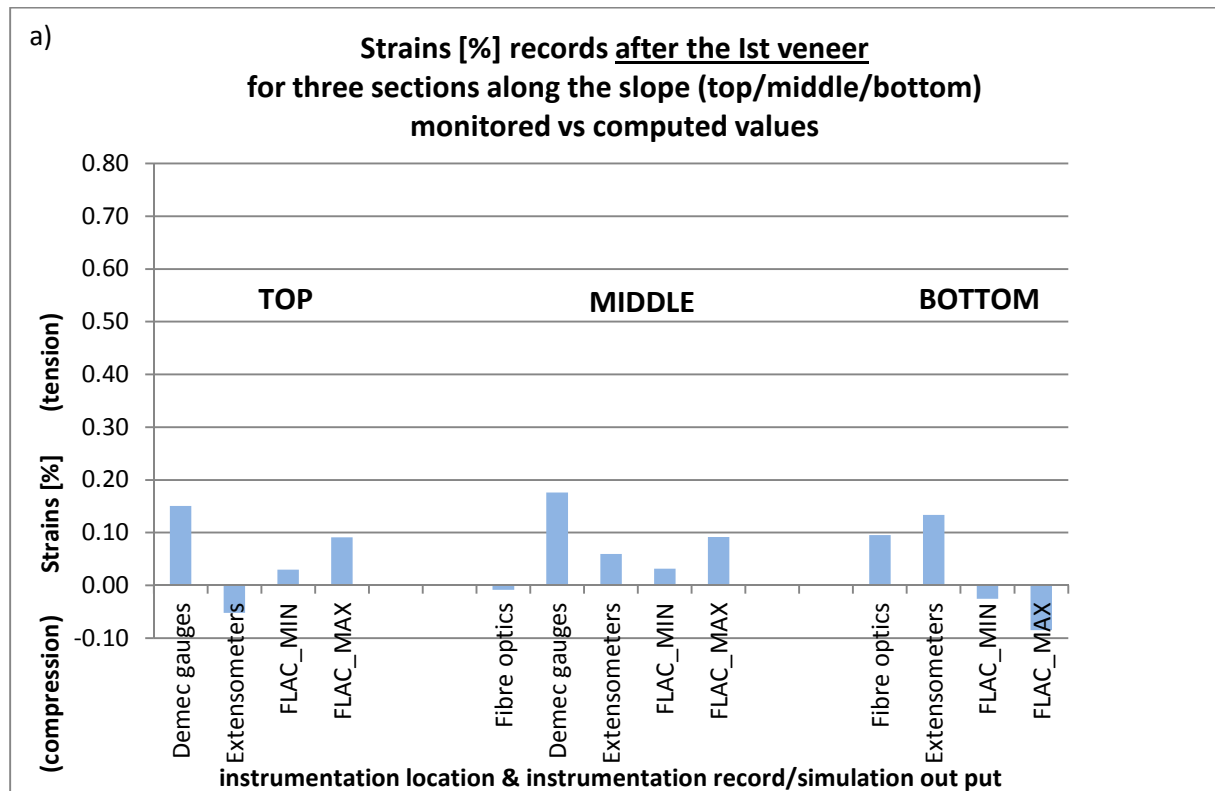
For the geotextile, difficulty was experienced trying to model the exposed section of the material, while the covered part is relatively well represented by the strain – softening approach. Due to the specific measured behaviour of geosynthetics and variability of the

conditions that materials were exposed to, it is difficult to select "the best fit" analysis, as the model at the same time can overestimate and underestimate the geosynthetics performance in different locations.

4.3 Strains in the geomembrane imposed by the veneer and waste loading vs FLAC predictions

Strains in the geomembrane were measured using three independent methods: DSG with reading points installed at the top and middle slope sections, FO sensors covered middle and bottom slope sections (all the sensors were lost prior to the 3rd veneer) and Ext – whole slope coverage. DSG represent strains calculated within relatively small distance of 20cm (at the slope crest values are calculated as average from 3 locations, in the middle of the slope - 1 location). Ext readings in terms of strains data present highly limited information, as these are calculated from recorded displacements of the subsequent measuring points installed on the geomembrane in a distance of 5.4m apart, and these may present movement of the localised area but not the entire section length. Therefore Ext readings are considered to be approximate. FO sensors represent the average reading collected for 1m long gauge sections. Three stages were selected for the strains comparison: after the 1st veneer, prior to the 2nd veneer and the final stage, when the whole slope was covered by waste, and these are discussed below. In terms of FLAC analysis – average values for the comparable section of the slope for the selected analysis were calculated. Monitored and modelled strains in the liner are presented in Figures 8a-c. Note that the exact value indicated by the instrumentation might be approximate (due to instruments reliability, performance, accuracy as reported by Zamara *et al.* (2012), however in general they indicate the magnitude of the strains, and due to installation of various instrumentation, cross check and comparison of the results is possible which increases confidence in overall indication of trends.

a)

Strains [%] records <u>after the Ist veneer</u>
for three sections along the slope (top/middle/bottom)
monitored vs computed values

b)

Strains [%] records <u>prior to the II veneer</u>
for three sections along the slope (top/middle/bottom)
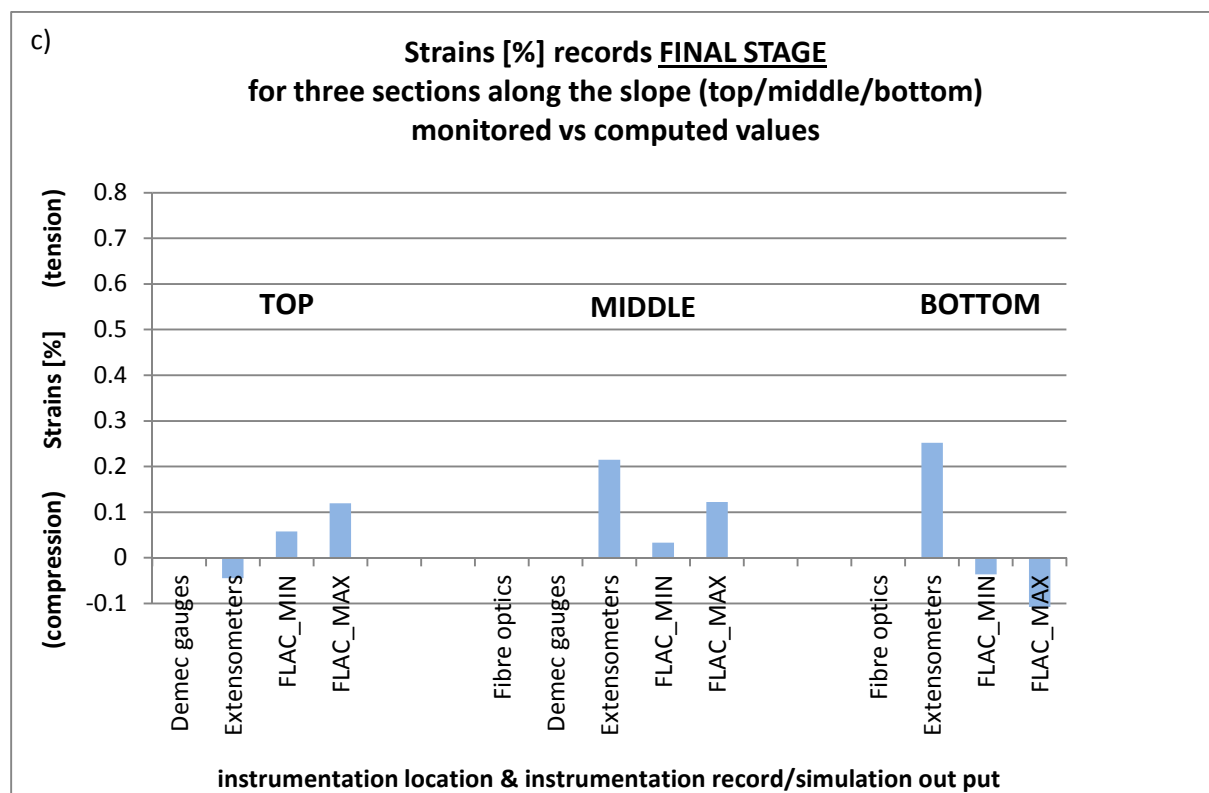**monitored vs computed values**

Figure 8. Strains recorded by the Milegate instrumentation and computed in FLAC for the following stages: a) strains after the Ist veneer, b) prior to the II veneer, c) final stage – whole slope covered by waste.

General conclusions drawn from the Figure 9a-c

Extensometers record compressive strains within the top slope section throughout almost all the stages of construction; this is suspected to be related to the monitoring system that might represent localised behaviour influencing globally the readings (additional temperature correction is applied to the Ext wire length, therefore the accuracy of the strains readings might be lower than the expected – 0.02%). In FLAC analysis, compressive strains are indicated for the toe section throughout the cell filling stages, while tension in this section is monitored on the site. This is due to fact that on the geomembrane the lowest point on the slope moved the most, while FLAC reaches peak displacements 25m below the crest. The middle section experiences a constant tension state and this is represented relatively well in FLAC, although it is underestimated in the basic analysis. Strains for the computed top

section are in tension throughout the filling stages and this agrees with DSG readings. For the final stages only Ext readings are available (DSG access unable due to sand cover and FO sensors damaged) and accuracy of these is limited. In general FLAC analyses present the same trends but different magnitude of strains.

I strains recorded after the 1st sand veneer

When the toe of the slope is covered by the 0.5m thick sand layer (parallel to the slope) FLAC indicates compression at the toe and tension within above sections (Figure 8a). However, *in situ* measurements shows that placement of the sand veneer causes tensile strains of over 0.1% within the loaded sections of the liner and reduced tension – 0.06% (or minimal compression for fibre optics) within above sections. Demec strain gauges records show relatively high tension for the exposed geomembrane 0.15% and strains of 0.18% for the crest and middle sections.

II strains recorded prior to the 2nd sand veneer

Prior to the second veneer most of the instrumentation show tension within the whole length of the geomembrane. At the toe section FO reached over 0.7% and for Ext over 0.2% while top and middle sections stay in a range of 0.1-0.2%. FLAC_MAX calculates uniformly distributed tensile strains of 0.09% for the exposed sections of the slope and indicates compression within the toe region.

Final stage strains

After III veneer placement no access to DSG was possible and FO were not operating, therefore only extensometer readings provide information on strains. These indicate tension within the middle and lower sections and compression within the top section of the slope. This only agrees with the computed in the middle section of the slope, which is in tension.

Summary

In terms of FLAC representation of *in situ* material performance, the overall calculated results are reasonable for standard theoretical analysis, however, monitored behaviour is more complex and description of all the incorporated factors is beyond the basic analysis. The basic simulations FLAC_MIN represent the geomembrane behaviour in a very limited way, with underestimated magnitude of the recorded strains. The highest tensile strains are recorded at the very top point of the slope directly adjacent to the anchor, while instrumentation records increased values within the lower sections, with fibre optics reaching 0.7% and extensometers around 0.2% throughout the monitoring period.

Additionally, FLAC is not able to represent compression/wrinkling/folding behaviour as this is complicated numerically to describe and requires confined compressive parameters for geosynthetics that are not routinely available.

5 LINER EXPOSED TO ATMOSPHERIC CONDITIONS

It has been observed that the basic FLAC analysis underestimates geotextile displacements, hence it could be concluded that the major geotextile displacements occur due to factors which are not directly modelled in FLAC. Temperature/solar radiation influences are not commonly considered in the standard design processes. Therefore additional analysis was undertaken in an attempt to investigate environmental influences on geosynthetic performance. Evidence was discovered supporting the hypothesis that HDPE cyclic expansion and contraction causes wrinkles formation on the geomembrane and subsequently geotextile. However while geomembrane materials contracts due to temperature drop, geotextile material remains stretched.   Milegate landfill data derived from extensometers revealed evidence of geomembrane/geotextile interaction and geotextile movement due to HDPE thermal deformation occurring within exposed sections. FLAC analyses with reduced strength

properties FLAC_MIN to some extent reflect those environmental issues (*i.e.* wrinkles formed on a geomembrane sheet decrease are of direct contact between materials reducing *de facto* interface shear strength between surfaces).

Additional analyses were undertaken in an attempt to better describe and quantify this trend.

5.1 Geosynthetics thermal behaviour - overview

It is widely accepted that black geomembrane will absorb heat from the sun. HDPE exposed to atmospheric conditions (*i.e.* solar radiation and high temperature amplitudes) will respond by expanding or contracting (retraction); and this will occur in response to seasonal changes in temperature, also due to daily cycles. This results in wrinkles occurrence within the geomembrane. Studies about geosynthetics thermal behaviour haves been carried out for over two decades. Giroud and Morel (1992) introduced a simplified model to describe wrinkle geometry and distribution on a horizontal geomembrane due to thermal expansion/contraction behaviour. However, this procedure has many limitations *e.g.* analysis was conducted for a horizontal surface while geomembranes are widely installed on slopes with varying inclinations, geometry of wrinkles in the geomembrane was simplified and predictions regarding wrinkle occurrence and overall behaviour are not fully considered. Studies regarding thermal behaviour of various geomembranes (*i.e.* Koerner *et al.* 1993, Peltie *et al.* 1994, Cadwallader *et al.* 1993, Ehrenberg & Recker 2012) show the significant influence of geomembrane colour on its temperature and hence behaviour (*i.e.* up to 30°C difference between white and black geomembrane). It has been recognised that HDPE surface temperature exceeds, often significantly, monitored air temperature, and depends on the solar radiation (Peltie *et al.* 1994). Moreover Take *et al* (2011) has observed that wrinkles have increased temperatures compared to the rest of the HDPE (due to air trapped underneath the wrinkle). Take *et al* (2012) reported temperature up to 15°C higher than the unwrinkled

HDPE. Additionally Akpinar & Benson (2005) report temperature effect on shear strength properties of geomembrane/geotextile interface. Friction angle increased with elevated temperature and decreased due to temperature drop (reported change was 2-3° for $\Delta T=33°C$). Existing research has focused on wrinkle behaviour when subjected to overburden stresses, as these affect leakage flow.

In this study the influence of geomembrane wrinkle formation and its influence on the overlaying geotextile material are considered, in relation with deformation of the lining system components during construction and waste placement.

While there are conducted extensive studies of HDPE wrinkling, to the authors' knowledge, data available on geotextile wrinkle development and behaviour due to exposure to sun radiation are limited. Lodi *et al.* (2008) investigated geotextile properties exposed to weathering. It was reported that after three months materials properties reduction in tensile resistance and mass per unit area, but increased in stiffness. Information regarding accurate geotextile wrinkle locations, sizes and displacements along the slopes are not well documented. Temperature records for the upper surface of the geomembrane, beneath the geotextile (at the location of sand veneer top) are presented in the Figure 9. Daily changes in the temperature reach up to $\Delta T=10°C$, while seasonal changes are of over $\Delta T=30°C$. These data might not reflect precisely the peak temperatures on the HDPE due to location of the sensor at the lowest point of the exposed section. Air temperature recorded for this area in the same period, revealed seasonal temperature difference of $\Delta T=40°C$. The subject of wrinkle formation, HDPE contraction and expansion, its influence on geotextile stretching is considered to be an important behaviour explaining mechanisms of recorded increased displacements of geotextile.
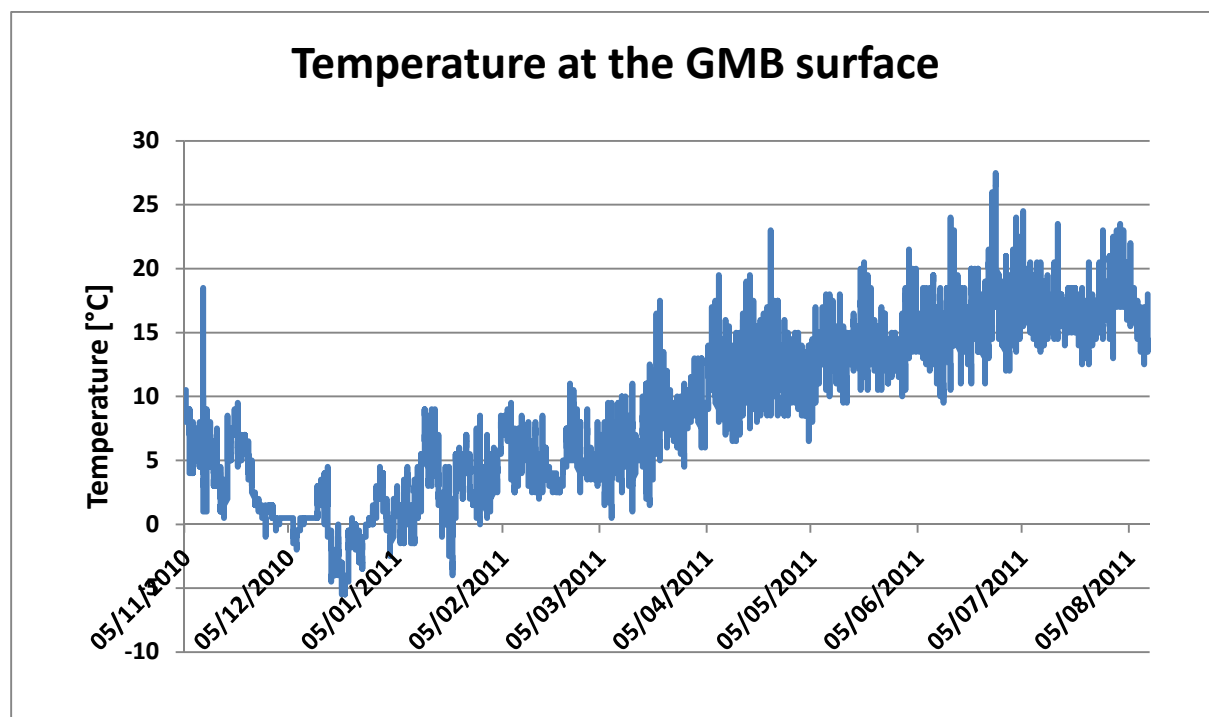
Figure 9. Temperature records at the top GMB surface.

5.2 Discussion on thermal factors

It has been observed that the most significant influence on geosynthetics performance on the slope occurred due to prolonged exposure to atmospheric conditions (the upper section of the slope remained uncovered until autumn 2011). Geomembrane undergoes deformations due to temperature changes that are partially reflected by the deformations readings (extensometers), but mostly by the readings collected with Demec strain gauges for strains across the slope (Figure 10). Cyclic daily deformations were not directly monitored; however geomembrane and geotextile deformations and wrinkling were observed during site visits and are documented in the site photographs (Figure 11). It is considered that geomembrane seasonal thermal expansion was reproduced by the overlying geotextile. The geomembrane was installed during the summer time (*i.e.* during a period of high temperatures), hence expansion would be at or close to maximum. Because the geomembrane is anchored, as is geotextile at the top of the slope, temperature decrease towards the first winter season would result in

material contraction, which is represented by readings from Ext1 and Ext2, which demonstrate small movements up the slope. Geotextile contracts only a small amount when temperature drop and hence although wrinkles in the geomembrane disappear, these in the geotextile do not. Figure 11 shows a geotextile wrinkle, which is not supported underneath by a geomembrane wrinkle. Although geomembrane wrinkle formation is replicated by geotextile the shrinking of wrinkles are not.
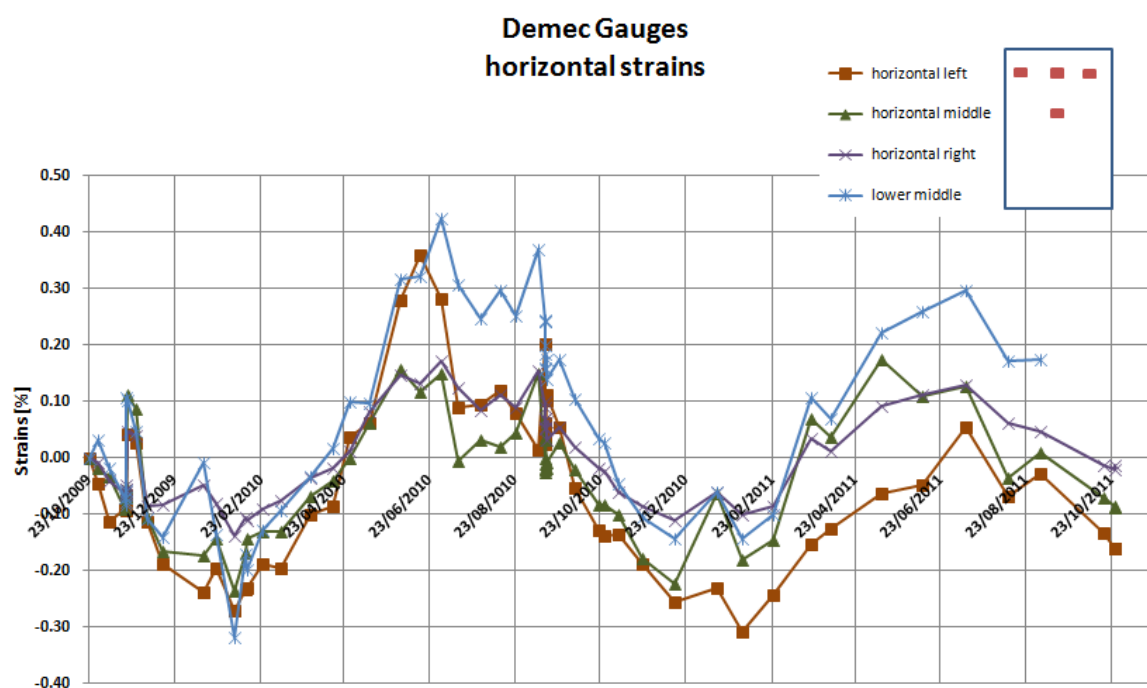

Figure 10. Demec strain gauges readings – strains across the slope.

In order to evaluate geosynthetics *in situ* behaviour analysis of HDPE thermal expansion calculated for two coefficients were undertaken: 1.1×10^{-4} cm/cm/°C (after Koerner 2005) and 1.5×10^{-4} cm/cm/°C (after Sheirs 2009) in correlation with recorded temperature variation $\Delta T = 30$°C (on the site) and $\Delta T = 40$°C (based on air temperature recorded in the area).

It is expected that total exposed geotextile Ext displacement due to veneer placement, expresses geomembrane extension due to seasonal temperature changes. The values are in the

good agreement with theoretical calculation of the HDPE sheet extension (Table 5). The mechanism describing that behaviour is schematically presented in the Figure 12.

Figure 11. Wrinkles on the GTX with GMB contracted underneath (07/09/2010).
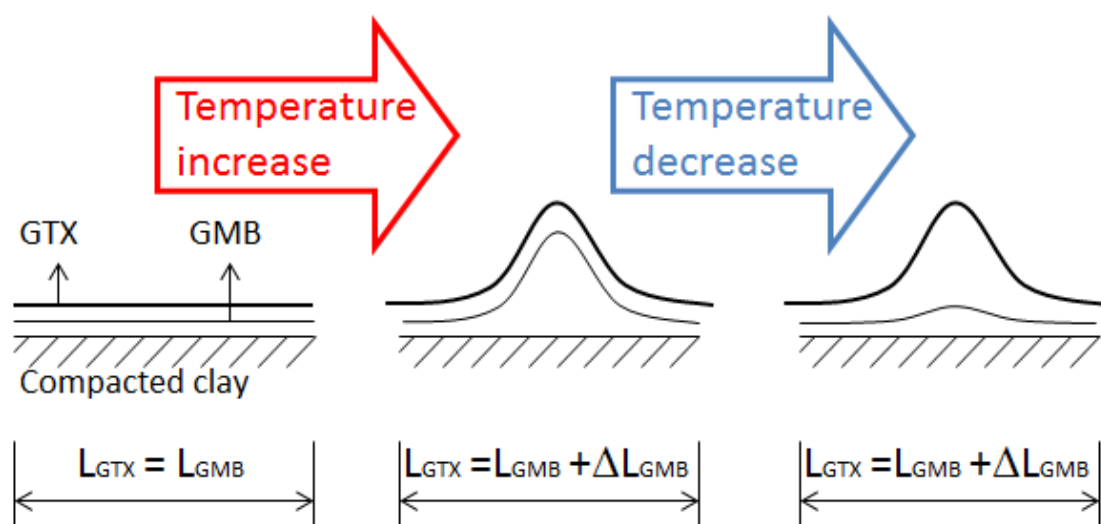

Figure 12. Schematic behaviour of the exposed lining system and GTX length progression.

Sensor ID	Sensor location below the crest / When covered by the sand layer	**Monitored GTX displacement** caused in the period of II sand veneer placement	**Theoretical geomembrane elongation** due to temp. change for the HDPE sheet length corresponding with the Ext location
Ext.1	3.0 m III veneer	20 mm	9.9 - 18.0 mm
Ext.2	8.4 m III veneer	50 mm	27.7 - 50.4 mm
Ext.3	13.8 m II veneer	80 mm	45.5 - 82.8 mm

Table 5. Extensometers readings analysis and theoretical elongation of the GMB sheet for the exposed section. Theoretical elongation are calculated for the lower boundary assuming ΔT=30°C and coefficient of thermal expansion equal 1.1×10^{-4} cm/cm/°C, and higher boundary ΔT=40°C and coefficient of thermal expansion equal 1.5×10^{-4} cm/cm/°C.

6 PROJECT LIMITATIONS/COMMENTS

Much effort was given to correct and reliable instrumentation installation, consistent readings acquisition and testing representative model. Nevertheless it is acknowledged that the project has several limitations related to various aspects of the study:

(1) All instrumentation is subjected to temperature changes, thermal corrections are based on the coefficients available from the literature (details in Zamara *et al.* 2012), but no direct thermal calibration of the materials and instruments was conducted;

(2) Extensometers readings, represent localised movement of the six points installed on the geomembrane and GTX, hence these may not adequately reflect the behaviour of the entire material sheet (this refers mostly to geomembrane and wrinkles formation);

(3) Strains calculated from Ext records are averaged over 5.4m gauge length;

(4) Geomembrane strain information acquired from fibre optics are more comprehensive (sensors on the slope are installed at smaller gauge lengths) than the Ext instruments, however, sensors were lost throughout the project therefore limited information is

available from these (last readings were collected prior to the 3rd sand veneer placement);

(5) Most of the instrumentation was installed along the middle part of the panel and plain strain conditions are assumed, therefore it is expected that necking effects are minimum;

(6) Additionally it is recognised that numerical modelling design approaches would benefit if more of this type of monitoring data was available, particularly for different lining system configurations, geometry and with consideration given to various speed of waste placement (not only in terms of geosynthetics loadings but also exposure to atmospheric conditions).

7 CONCLUSIONS

The Milegate Extension Landfill monitoring project was conducted throughout three years. The initial aim of the project was to validate standard design approaches incorporating numerical modelling of the landfill lining systems. Data including stresses imposed on the slope, geosynthetics displacements, strains and surface temperature are presented in this paper. The Milegate project brings information on the lining system performance before, during waste placement and it is planned to conduct further monitoring after the landfill closure, when the waste degradation and settlements will occur. Numerical modelling of the monitored slope was conducted, selected analysis are presented in this paper and compared with the site derived data.

In scope of the monitored geosynthetics behaviour, it is recognised that FLAC gives a good approximation of the results for the lining system where the ambient temperatures does not change significantly or if the geosynthetics materials are not directly exposed to solar

radiation (*e.g.* covered by soil). However significant thermal factor has been identified as the driving force in geotextile displacement. In an attempt to replicate this behaviour, reduced values of the lining system interface shear strength were assigned and modifications in soil and geotextile material properties applied (*i.e.* reduced interface shear strength of the lining components due to wrinkling of the exposed materials and ageing of the geosynthetics). It is acknowledged that sections of the Milegate landfill liner were exposed to weathering conditions and estimation of its influence on the liner performance is limited, hence analyses with reduced residual interface shear strength values were carried out to trigger movement on the clay/geomembrane interface and to enhance geotextile movement. Overall peak and residual values were highly similar in terms of the materials displacements and maximal axial forces occurring in the geomembrane.

It should be emphasised that for the section of slope covered by sand, where geosynthetics are not directly exposed to environment, no increased deformation occurs within the lining components occurs, and values computed in a standard approach are in a range of the monitored ones. That indicates that numerical model applicability is limited when prolonged expose and thermal effects become the dominant mechanism of the displacement.

Analysis of the climate data and its influence on the liner temperature and overall performance are uncommon and much more complicated than the standard design approach. However, attempts should be given to estimate environmental impact on the geosynthetic performance to better represent actual conditions, to limit possibility of failure occurrence, and to assure safe construction. It is advised that if geosynthetics might be exposed to weathering for a prolonged time, modified and reduced values for not only interface shear strength should be considered but also modified values for materials properties.

REFERENCES

Akpinar, M.,V., Benson, C.H., 2005. Effect of temperature on shear strength of two geomembrane-geotextile interfaces. Geotextiles and Geomembranes, v 23, issue 5, October 2005, pages 443-453.

Arab 2011, The Integrity of Geosynthetic Elements of Waste Containment Barrier Systems Subject to Seismic Loading. PhD thesis, Arizona State University. July 2011.

Cadwallader, M., Cranston, M., Peggs, I.D., 1993. White surfaced HDPE geomembranes: assessing their significance to liner design and installation. Proceedings of geosynthetics '93, vol. 2, pp. 1065-1079.

Chappel, M.I., 2012. A field scale evaluation of wrinkles in exposed HDPE geomembranes. PhD thesis, Queen's University, Kingston, Ontario, Canada. June 2012.

Dixon, N., Zamara, K.A., Jones, D.R.V., Fowmes, G., 2012. Waste/Lining System Interaction: Implications for Landfill Design and Long-term Performance. *Geotechnical Engineering Journal of the SEAGS & AGSSEA* Vol. 43 No.3 September 2012 ISSN 0046-5828.

Dixon, N., Jones, D. R., 2003. *Stability of Landfill Lining Systems: Report no.2 Guidance.* Environment Agency Research and Development Technical Report P1-385/TR2.

Dixon, N., Jones, D. R., 2005. Engineering properties of municipal solid waste. *Geotextiles and Geomembranes* 23, pp. 205-233.

Ehrenber, H., Recker, C., 2012. The influence of Geosynthetics Color on the UV resistance performance under natural and lab testing conditions. 5th European Geosynthetics Congress. Valencia 2012. Proceedings Vol. 5. Topic: Mining & Environmental Applications.

Filz, G.M., Esterhuizen, J.J.B., Duncan, J.M., 2001. Progressive failure of lined waste impoundments. Journal of Geotechnical and Geoenvironmental Engineering. October 2001, pp. 841-848.

Fowmes, G.J., Jones, D.R.V., Dixon, N. 2005. Analysis of a landfill directive compliant steep wall lining system. Proceedings 10th International Waste Management and Landfill Symposium, Sardinia. pp369.

Dixon, N., Jones, D.R.V. , Fowmes, G.J., 2006. Interface shear strength variability and its use in reliability-based landfill stability analysis. *Geosynthetics International*. Vol. 13, No1. p 1.

Fowmes, G.J., Dixon, N., Jones, D.R.V., 2007. Landfill Stability and Integrity Toolbox. Internal Golder Associates (UK) Ltd Design Guidance Document.

Fowmes, G.J., Dixon, N., Jones, D.R.V., 2008. Validation of a numerical modelling technique for multilayered geosynthetic landfill lining systems. *Geotextiles and Geomembranes* 26, pp. 109–121.

Giroud, J.P., Morel, N., 1992. Analysis of geomembrane wrinkles. Geotextiles and Geomembranes, 11, 1992, pp. 255-276

Gudina, S. and Brachman, R.W.I., 2011. Geomembrane strains from wrinkle deformation. Geotextiles and Geomembranes 29 (2011) pp. 181-189

Jones, D.R.V., Dixon, N., 2005. Landfill lining stability and integrity: the role of waste settlement. *Geotextiles and Geomembranes* 23, pp. 27-53.

Koerner, R.M., Hsuan, Y., Lord, A.E., Jr., 1993. Remaining technical barriers to obtaining general acceptance of geosynthetics, Geomembranes and Geotextiles, vol. 21, no. 1, pp. 1-53.

Koerner, R.M., Soong, T-Y. 2000. Stability Assessment of 10 Large Landfill Failures *Geodenver 2000 ASCE SPECIA* vol 103, pp. 1-38.

Lodi, P.C, de Souza Bueno, B., Zornberg, J.G., de Souza Correia, 2008. Analysis of mechanical properties on geotextiles after weathering exposure. Conference proceedings EuroGeo4 paper no 203.

Muhsiung, C., 2005. Three-dimensional stability analysis of the Kettleman Hills landfill slope failure based on observed sliding-block mechanism. Computers and Geotechnics 32, pp. 587–599.

Peltie, T., Pierson, P., Gourc, J.P., 1994. Thermal analysis of geomembrane exposed to solar radiation. Geosynthtetic international, vol. 1, no. 1, pp. 21-44.

Sia, A, Dixon, N., 2012. Numerical modelling of landfill lining system–waste interaction: implications of parameter variability. Geosynthetics International, Volume 19, Issue 5, 01 October 2012 , pages 393 –408 , ISSN: 1072-6349, E-ISSN: 1751-7613

Sheirs, J., 2009. A guide to Polymeric Geomembranes: A practical approach. John Wiley & Sons. ISBN 978-0-470-51920-2

Take, W.A., Watson, E., Brachman, R.W.I., Rowe, R.K., 2012. Thermal expansion and contraction of geomembrane liners subjected to solar exposure and backfilling. Journal of Geotechnical and Geoenvironmental Engineering.

Villard, P., Gourc, J.P., Feki, N., 1999. Analysis of geosynthetic lining system (GLS) undergoing large deformations. Geotextiles and Geomembranes, Volume 17, Issue 1 February 1999, pp. 17-32.

Zamara, K.A., Dixon, N., Jones, D.R.V., Fowmes, G., 2012. Monitoring of a landfill side slope lining system: Instrument selection, installation and performance. Geotextiles and Geomembranes, 35 (2012) pp.1-13.

APPENDIX D PAPER 4

Full Reference

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Instrumentation measuring hydraulic performance of a landfill cap: Case study. The 5th

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Instrumentation measuring hydraulic performance of a landfill cap: Case study

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ABSTRACT: Optimisation of landfill capping design is an important issue due to environmental and economical aspects. To date little attention has been given to analyse pore water pressure distributions above the low permeability liner and along the landfill cap slope. The aim of this study is to deliver data from a full scale field trial to add confidence in the design, enhance design optimisation and furthermore to validate numerical modelling of capping systems. This paper focuses on presenting the monitoring instrumentation installed at the Bletchley Landfill capping, where three different geosynthetic drainage materials were deployed: a cuspate core geocomposite, a geocomposite with band drains along the sheet and a non-woven needle punched geotextile. Monitoring includes water entering the system (inflow), leaving the system (outflow) and soil saturation. The site instrumentation consists of: a weather station, surface run off gauge, piezometers, volumetric water content sensors and a system monitoring water outflow from the drainage layers. Additionally, for

comparison a section without a drainage layer was instrumented. Initial results are presented of volumetric water content, precipitation records and drainage layer water outflow. Initial readings have demonstrated performance of the monitoring system but to date, due to the relatively dry conditions, no pore water pressure have been recorded although moisture content changes respond to precipitation events. It is planned to continue monitoring over the next 12 months

INTRODUCTION

Landfill capping is an important component of engineered landfill constructions. It minimizes exposure of the waste body, prevents water infiltration which determines production of leachate, controls gas emission from waste body and eventually creates a land surface that can support vegetation or be used for other purposes. There is currently very little site derived data about groundwater behaviour within landfill cover systems or pore water pressure (pwp) distributions along the slope in response to geometric, soil and atmospheric conditions. Hence, commercially applied methods of design use parallel submergence ratio (PSR) or horizontal submergence ratio (HSR) to define water pressures used to evaluate landfill slope stability or to design a cap drainage layer (Figure 1). If the cover system is not stable, with any pwp conditions, the inclusion of drainage beneath the cover soils would typically be specified to reduce the level of pwp's and therefore increase the factor of safety against both, stability and integrity failures. However, little attention is given to the actual drainage requirements, in terms of flow capacity required to reduce the pwp within the slope. Moreover calculation methodologies are based on assumptions that may not necessarily provide an optimal solution. In order to verify the assumptions, improve the calculation methodology and ensure optimal, cost effective drainage system design, there is a need to monitor an existing landfill cover system under operating conditions. This provides an opportunity to obtain valuable information to aid the design of future landfill capping drainage systems and to assess performance of existing ones.

Additionally, despite its importance, there is a lack of comprehensive information to investigate hydraulic performance of the capping system. Much research focuses on a vertical infiltration and overall water balance or so-called percolation studies (Strunk *et al.* 2009, Henken-Mellies & Gartung 2004, Cazaux & Didier 2000) but not much attention is given to analyse ground water behaviour in relation to slope geometry, atmospheric and soil conditions

at various heights along the slope (surface run-off and infiltration rate), water table behaviour and its variability along the slope. This information is of importance in order to optimise current commercially applied approaches and to validate complex numerical modelling software. This project addresses design of drainage layers; and examines requirements for flow capacity in relation to slope geometry, atmospheric data, top soil properties and drainage layer parameters. Accurate prediction of pwp for different weather scenarios, various soil parameters and slope geometry will allow design of containment systems covers with increased confidence. The subject of thin veneer soil covers is widely described in the worldwide literature (*e.g.* Koerner & Daniel 1997, Berger 2000), but still little attention has been given in the UK for drainage layer optimal solutions. An appropriate analysis of water balance within landfill covers and water table behaviour is crucial for designing an environmentally safe capping with a drainage layer suited to ensure optimal performance.

This research project aims to obtain information about hydraulic performance of landfill caps under various drainage conditions, to identify parameters which influence the system the greatest and to produce guidance for capping system drainage design. Monitoring instrumentation consists of weather station, surface run-off catchment, stand pipe piezometers, moisture content sensors and drainage outflow measuring gauge. Additionally, laboratory tests have been performed to obtain information about hydraulic properties of the cover soil. Monitoring of the landfill capping will be carried out for a 12 month period. Further research will investigate the correlation between observed and modelled behaviour with the aim of validating analysis approaches and producing design charts for stability assessment and drainage design.
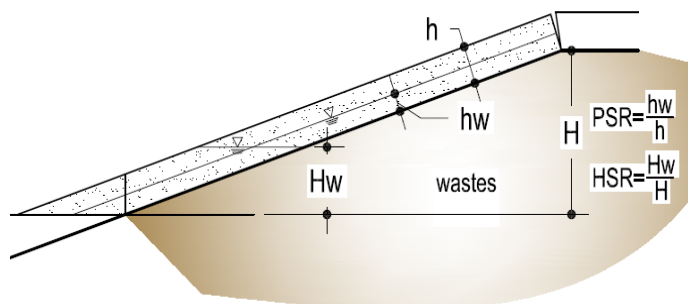
Figure 1. PSR and HSR ratios definition, h$_w$-saturated thickness parallel to the slope, H$_W$ - saturated height (after Koerner & Daniel 1997).

This paper explains the importance of acquiring information about drainage performance, it describes the site location and configuration of monitored system, and presents instrumentation and philosophy of instruments selection and installation. It also presents initial result of: (1) volumetric water content sensor (VWC) measurements in response to precipitation events, and (2) limited data from drainage system outflow.

2   NEED FOR MONITORING

Research has been conducted investigating problems related to hydraulic efficiency of landfill caps. For example Albright *et al.* (2006) and Henken-Mellies & Gartung (2004) built large scale lysimeters on landfill caps, in the USA and Germany respectively, to monitor water management of the caps and their efficiency. Nyhan (2005) reports water balance parameters recorded throughout 7 years on a landfill site in New Mexico, USA. Equally, attention has been given to validate numerical modelling studies of landfill covers. Choo & Yanful (2000) compare finite element methods predictions with laboratory test results. Bussiere *et al.* (2003a,b) investigate numerical modelling software functionality through laboratory tests, and site derived data with a focus on capillary barrier effect and efficiency. Very limited research focuses on drainage material performance and efficiency of materials available and installed on sites. Del Greco *et al.* (2011) report laboratory tests conducted on geosynthetic drainage materials. To the Authors' knowledge there is no data available about pwp distribution above the low permeability barrier within the landfill capping system in relation to cover stability, or

water balance parameters reflecting performance of various drainage materials and justifying application. Hence it was decided to conduct a full scale investigation that monitors drainage material efficiency in response to atmospheric conditions.

3　　MONITORING OF CAPPING SYSTEM PERFORMANCE

3.1　Case study: Bletchley Landfill. Geometry & Materials

The field trial at Bletchley Landfill is located near Milton Keynes, Buckinghamshire, UK. Three different drainage materials were installed on the capping: non woven geotextile (GS1), cuspate core geocomposite (GS2) and geocomposite with band drains at 1m centres spaced along the sheet (GS3). The hydraulic parameters of the drainage materials are presented in Table 1. A section without drainage material was similarly instrumented to provide a reference base. The geosynthetic material lengths are: 45m for the GS1 and GS3 and 40m for the GS2. The selected section of the slope was uniform with inclination angle 1v to 8h (7.1°). The veneer cover comprised of the soil available on the site with a moderate to low permeability, which overlies a compacted clay layer. The veneer soil properties were investigated in the laboratory; the results are presented in Table 2. Due to the design criteria of the existing capping it was decided to place a 0.5m thick soil layer on top of the low permeability barrier layer, however during instrumentation installation it was found out that average the depth of the top-soil over the drainage materials is 0.4m.

4　　PHILOSOPHY OF INSTRUMENTATION DESIGN

Parameters influencing the water budget of the cover can be grouped into three categories as shown in Table 3. Parameters in the IN-PUT/OUT-PUT category are crucial for the project objectives, whereas data collection about the mechanism of water movement in the cover soil and be used to understand the process. Therefore monitoring has focussed on the IN-PUT/OUT-PUT data in the first instance.

	FLOW	In-plane flow capacity (with hard platens for 20kPa)
	EN ISO 12958	EN ISO 11058
	[1/m²/s]	[1/s/m width]
GS1 (HPS5*) Geotextile	65 ASTM D4491	--------
GS2 (FCF6*) cuspate core geocomposite	80	i=0.1 0.5 i=1.0 2.0
GS3 (GPT5*) Geocomposite with drains along the sheet	40	i=1.0 0.2

* original names of products provided by GeoFabrics Ltd.

Table 1. Hydraulic properties of the geosynthetic materials.

5   DETAILS OF INSTRUMENTATION INSTALLATION

5.1   Weather station

Although data is available from meteorological stations near to the site, it is essential to collect local, on-site weather information. Since for this case precipitation and temperature are the driving forces influencing groundwater table behaviour, the weather station is crucial to produce input data for numerical modelling validation. A Vintage Pro 2 system is located at the landfill site and the operator made the data available. It records a set of atmospheric parameters every half hour. The most important parameters recorded are: precipitation, air temperature min/max, wind speed and direction, relative humidity min/max.

5.2   Surface run-off measurement

One of the essential factors within a water balance is the run-off rate. As investigation of this parameter is complicated, unreliable and quite often fails (Hudson 1993), little data exists on accurate parameter values. There are several approaches on how to theoretically assign the run-off parameter, however values obtained are questionable as the accuracy is unknown. This results in a wide range of possible applicable values (Koerner & Daniel 1997, ODOT 2005).

Soil parameters	Values
Bulk density [Mg/m³]	1.55
Moisture content [%]	~27
Permeability [m/s]	~1×10^{-8}*
VWC	0.27

* to be confirmed with additional test results.

Table 2. Soil parameters

IN-PUT	MECHANISM	OUT-PUT
-atmospheric conditions	-volumetric water content of soil cover	-drainage layer out flow
-surface run-off		-pore water pressure
-slope geometry		
-soil initial parameters		

Table 3. Philosophy of instrumentation selection.

One surface run-off plot was installed on the site. A guttering pipe collects surface run-off water and transports it to a tank with a tipping bucket system used to measure run-off flow rate (Figure 2). A tipping bucket self empting gauge with a logger was installed for this purpose. It is expected that due to a low inclination of the slope the surface run-off amount will be small and most of the water will infiltrate into the cap. Also, with the spring season approaching, vegetation appearing on the cap surface will minimise or eliminate run-off.

5.3   Volumetric water content loggers

In order to monitor spatial and temporal moisture movement within the cover soil in response to precipitation events and drying periods VWC sensors were installed (water content reflectometers, model CS616, Campbell Scientific). The reflectometer sensors are widely used in agriculture and soil sciences, although their accuracy depends on a number of factors: method of insertion, maintaining the geometry of the device, surrounding soil conditions, inserting sensors without air void generation, maintaining original soil density, removing stones (Campbell Scientific 2011). Nevertheless data obtained from sensors can still provide useful information, especially on the timing and relative rate of change in moisture content.
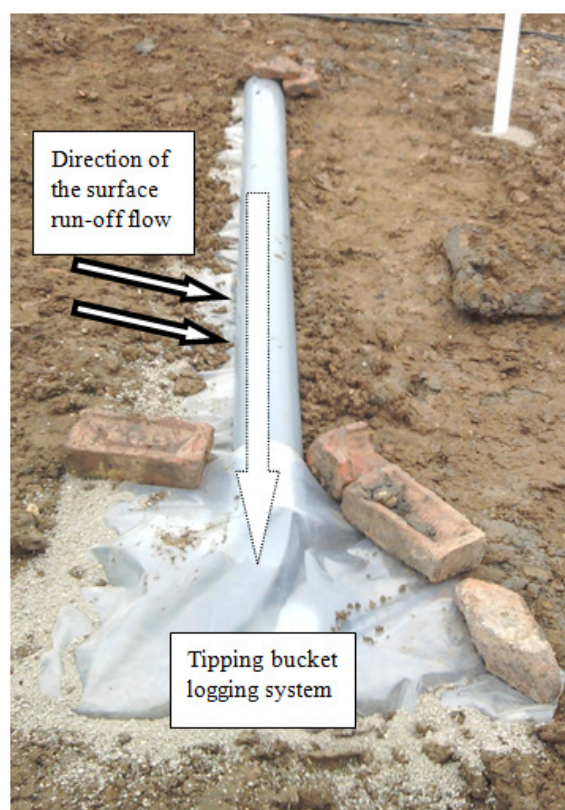
Figure 2. Surface run-off measurement system.

At each monitored section (GS1/GS2/GS3/soil), VWC sensors (20 and 38m down slope from the top edge of the geocomposites) were placed at various depths that were determined by the thickness of the permeable layer at the particular location. The sensors were installed in the cover soil above the geosynthetics/clay and parallel to the slope. The aim is to detect moisture movement across and within the soil layer, to determine soil VWC response to various weather conditions. The logging system records data every half hour. Schematic instrumentation location is presented on Figure 4. To allow comparison, each panel was instrumented in the same configuration.

5.4   Standpipe piezometers

Knowledge of the pwp's acting on the impermeable layer/cover soil interface are critical when estimating veneer cover stability. Current design calculations often assume the worst case scenario (this usually assumes a fully saturated cover layer), in order to obtain a

satisfactory factor of safety against slippage. To measure pwp's stand pipe piezometers have been installed immediately above the drainage layer. Initially, piezometers were installed at the lower and middle part of the slope (Figure 3), at the same locations as the VWC loggers. If water is observed within these sections, additional stand pipes will be installed within the top section of the slope.

Site visits are made on a regular basis and therefore it was decided to monitor water table behaviour manually with a dipp meter. If a groundwater table develops, loggers will be installed.
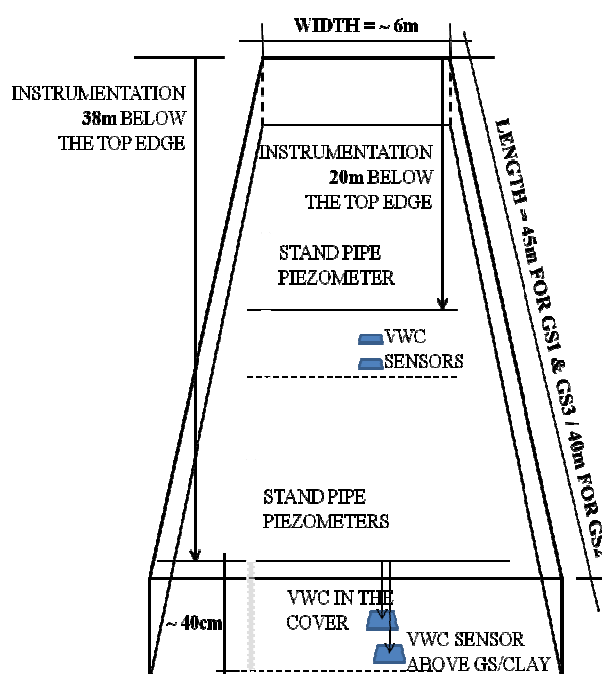

Figure 3. Schematic location of the measuring instrumentation.

5.5   Measurement of flow from drainage systems

A system of pipes (made of guttering pipes) has been installed along the lower edge of each drainage material panel to collect all out-flow from the drainage layer. The water then runs into a tank placed down slope, where manual measurements of flow are taken at each site visit. Water leaving the tank is transported by a system of the pipes to the toe of the capped slope (Figure 4). Additional logging system is to be installed to monitor outflow.
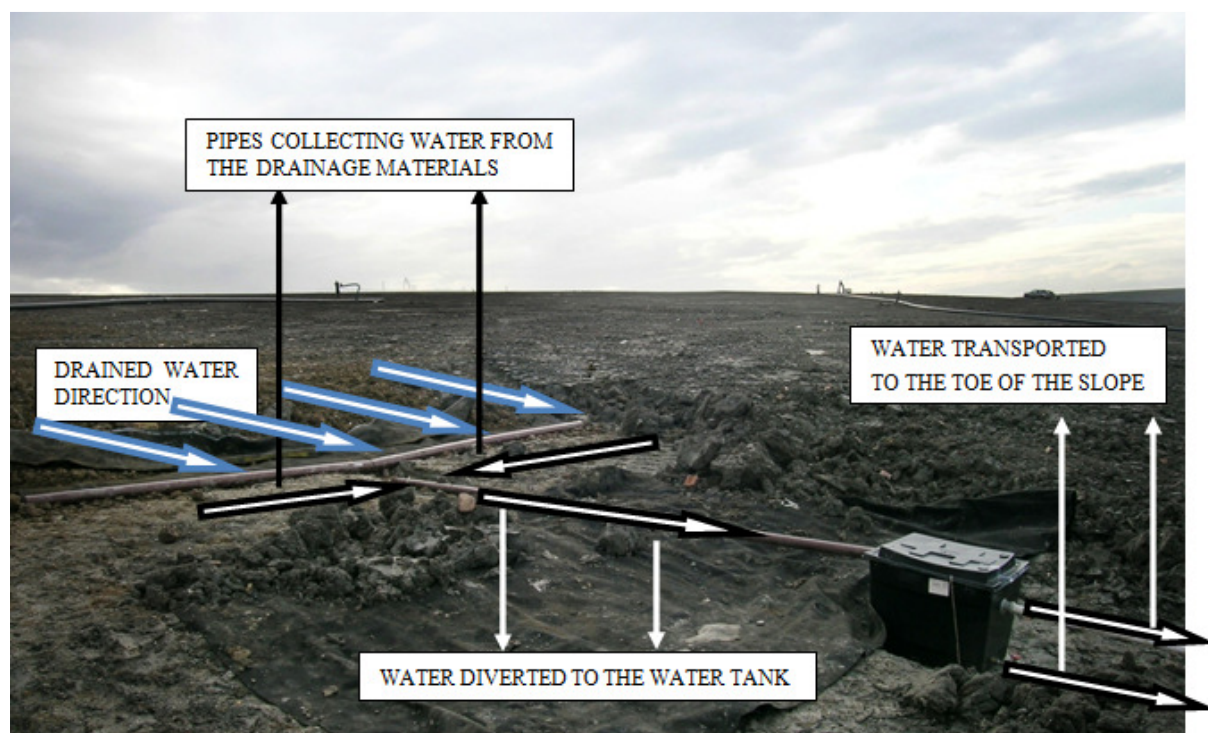
Figure 4. System for measuring water out-flow.

6 INITIAL RESULTS

Figure 5 presents the initial records from the site instrumentation. Relative changes of VWC above the two geocomposite materials, the geotextile and compacted clay layer are presented. Readings from the sensors located within the lower sections of the slope are shown.

Precipitation records are presented on the inverted right hand axis as are flow volume measurements from the drainage materials taken during site visits. A rapid VWC sensor response to some of the rainfall events is noticed during or shortly after the event. This provides confidence in instrumentation in-service performance. The behaviour observed for the sensors installed above GS3 (sensor located between band drains) and clay (VWS Soil) is very similar, wetting process progresses almost parallel. In general GS1 seems to keep lower VWC within the above soil layer, whereas VWC values above GS2 are higher. Regarding the interpretation of the results, site specific conditions and limitation of measuring instrumentation should be taken into account. Hence, additional aspects worth considering while analysing the data trends are as follow:
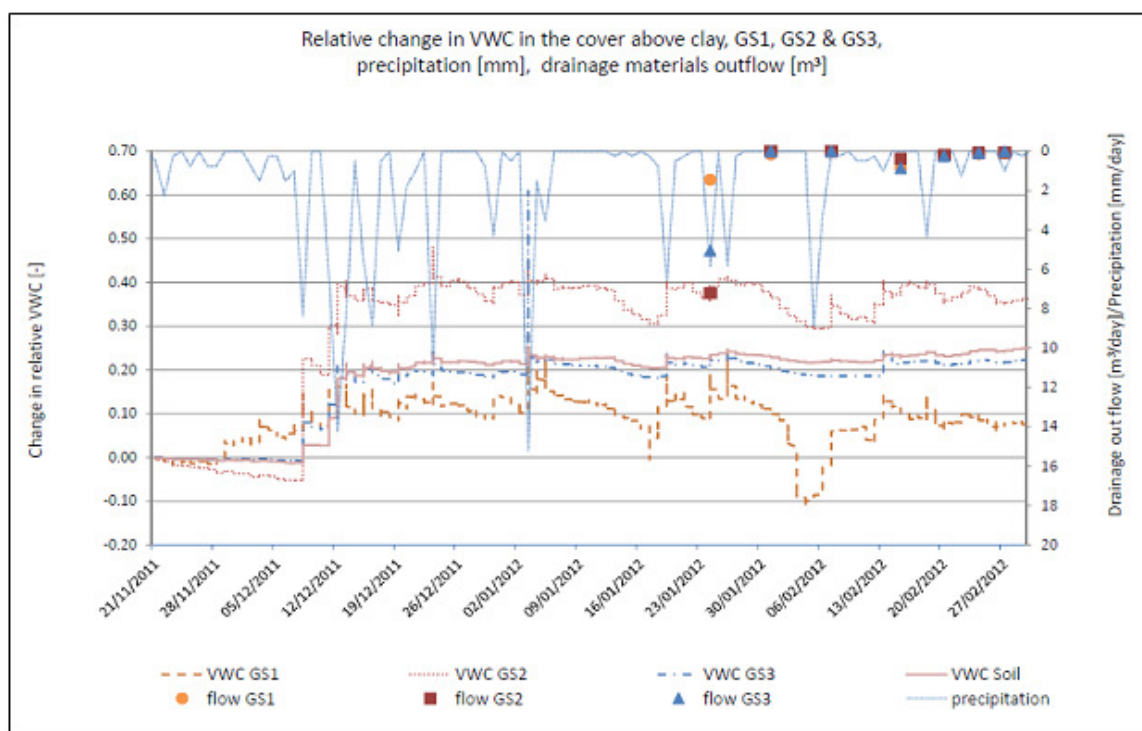
Figure 5. Initial records from the instrumentation

- spatial thickness variability - design soil thickness was 0.5m, however due to on site conditions, the obtained actual thickness on average was 0.4m (minimum over GS2- 0.31m and maximum in the soil only section, 0.47m). The response of the VWC sensors for the locations with thicker cover soil layers should be slower, however that is not expected to influence significantly drainage water outflow,

- spatial variability in the cover soil permeability coefficient. The cover soil is assumed to a have constant value of the coefficient, however this might vary, and

- uncertainties related to sensors installation, operation, calibration.

First records of the water out-flowing from the pipe systems were taken in January 2012, on a day when high precipitation was monitored. The subsequent records were taken on days with relatively low or no precipitation, hence the water volumes drained by the systems are significantly lower after the first measurement and overall fairly similar. Since initially only point time measurements have been collected of the water flows out of the drainage layers,

these produce limited information on the drainage layer performance. Therefore, it is planned to take manual readings at time intervals throughout a rainy day, as well as installing a tipping bucket flow measurement device to continuously log water outflow. This will help establish the direct relationship between rainfall and drainage flow. To date the stand pipe piezometers have not measured water levels within the capping soil layer, and there have been only occasional instances of a wet geotextile and/or clay line surface being observed.

7   SUMMARY

Site derived data is valuable for future development of engineered landfill facilities not only for optimised and economic construction but also to validate numerical modelling of landfill capping systems and increase confidence in design. This paper describes monitoring instrumentation that was installed on a landfill site capping. The instrumentation compries VWC loggers, piezometers, a run-off plot, drainage layer outflow monitoring systems and a weather station. These allow recording of data influencing the water budget of the landfill cap. Three sections with geosynthetic materials and one section without drainage has been instrumented. The results to date show relatively rapid response of VWC sensors to rainfall events. Monitored values of water leaving the drainage system initially show interesting data; Even though the cover soil has relatively low permeability the drainage system responds to the atmospheric conditions relatively rapidly. This effect will be further investigated and instrument modification made in order to acquire more detailed information on the systems performance.

   The landfill capping site full scale monitoring project comprises part of research investigating landfill capping drainage systems performance. This aims to investigate and establish drainage requirements for various atmospheric, geometric and soil permeability conditions to ensure stability.

References

ASTM D4491 Standard Test Methods for Water Permeability of Geotextiles by Permittivity.

Albright W.H., Benson, C.H., Gee, G.W., Abichou, T., Tyler, S.W., Rock S.A, 2006, Field Performance of Three Compacted Clay Landfill Covers, Vadose Zone Journal Vadose Zone Journal 5:1157–1171.

Berger, K., 2000. Validation of the hydrologic evaluation of landfill performance (HELP) model for simulating the water balance of cover systems. Environmental Geology 39 (11), pp. 1261-1274.

Bussière, B., Aubertin, M., Chapuis R. P., 2003a. The behaviour of inclined covers used as oxygen barriers. Can. Geotech. J. 40: 512–535

Bussière, B., Aubertin M., Chapuis, R.P., 2003b, The behavior of inclined covers used as oxygen Barriers Can. Geotech. J. 40: 512–535.

Campbell Scientific 2011 User Guide. CS616 & CS625 Water Content Reflectometers
Cazaux, D. & Didier, G., 2000. Field evaluation of hydraulic performances of geosynthetic clay liners by small and large-scale tests. Geomembrane and Geotextiles Volume 18, Issues 2-4, Pages 163-178.

Choo,L.-P., Yanful, E.K., 2000. Water flow through cover soils using modelling and experimentall methods, Journal of Geotechnical and Geoenvironmental Engineering, Vol.126, No. 4.

Del Greco, O., Lefevre, P., Oggeri, C., 2011. Performances of draining geosynthetics for applications in landfill construction. Proceedings of 13th International Waste Management and Landfill Symposium, Sardinia 2011.

EN ISO 11058:1999 – Geotextiles and geotextile-related products – Determination of water permeability characteristics normal to the plane, without load.

EN ISO 12958:1999 – Geotextiles and geotextile-related products – Determination of water flow capacity in their plane.

Henken-Mellies, U. & Gartung, E., 2004. Long-term observation of alternative landfill capping systems – field tests on a landfill in Bavaria. Land Contamination and Reclamation, 12 (1), 2004.

Hudson, N., 1993. Field measurement of soil erosion and runoff. Volume 68 of FAO soils bulletin.

Koerner, R., M., Daniel D., E., 1997. Final Covers for Solid Waste Landfills and Abandoned Dumps. American Society of Civil Engineers 1997.

Nyhan, J.W., 2005, A Seven-Year Water Balance Study of an Evapotranspiration Landfill Cover. Vadose Zone Journal 4:466–480.

ODOT 2005. Oregon Department of Transportation, Hydraulics Manual 2005

Strunk, R.L, Dobchuk, B.S., Nieminen, G., Barbour S.L., O'Kane, M., 2009. Performance Monitoring of Soil Cover Field Trials at the Regina Municipal Landfill. Web site accessed on the 15.02.2012:

http://www.okaneconsultants.com/pdf_publications/gmm/Strunk%20_Dobchuk_%20Regina_landfill_paper.pdf