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Analysis of Steep Sided Landfill Lining Systems

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ANALYSIS OF STEEP SIDED LANDFILL LINING SYSTEMS

By

Gary Fowmes

A Dissertation Thesis submitted in partial fulfilment of the requirements for the award
of Engineering Doctorate (EngD) of Loughborough University

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Abstract

The EC Landfill Directive (1999), which is enforced in England and Wales through the Landfill (England and Wales) Regulations (2002), has increased the technical challenge associated with the design and construction of landfill containment systems, in particular those on steep side slopes. Increased numbers of lining system components, varied configurations, and complex loading scenarios require advanced analysis tools to facilitate design.

This project involved the development of advanced numerical modelling techniques, based on the FLAC finite difference modelling code. The analysis toolbox can be used to predict the behaviour of multilayered geosynthetic and soil lining systems, during and after staged construction. The model can include non-linear interface and geosynthetic axial properties, represent complex loading, including downdrag from the waste mass, whilst retaining the flexibility to represent varied geometries and include engineered support structures.

Whilst numerical modelling is becoming increasingly commonplace in commercial design, there is little evidence of the validation of numerical models with field or experimental data. Validation of the analysis toolbox described in this document was conducted by back analysis of published data, modelling of landfill failure mechanisms, and comparisons to large scale laboratory testing. Design of field scale instrumentation has also been carried out as part of this project.

The influence of interface shear strength variability has been assessed through the compilation of a comprehensive database, and the effect of this variability on lining system behaviour assessed through reliability based analyses. This has shown probability of failures may be higher than proposed limiting values when adopting traditional accepted factors of safety.

A key area of interest identified during the project was the requirement for support, potentially through reinforcement, of the geological barrier. The inclusion of randomly reinforced fibres in bentonite enhanced soil has shown the potential for increased strength, without adverse effects on hydraulic barrier performance.

Additionally, the influence of geomembrane seams on lining system integrity has been investigated, showing that fusion welded seams can result in stress concentration and extruded seams can cause significant stress concentration.

Key Words

- Landfill engineering,
- Landfill design,
- Landfill directive,
- Steep sided lining systems,
- Steepwall lining systems,
- Geosynthetics, waste barrier interaction,
- Numerical modelling,
- Interface shear strength variability,
- Reliability based design,
- Randomly reinforced soil,
- Geomembrane seams.

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Abbreviations

BES	Bentonite enhanced soil
COV	Coefficient of variation
CQA	Construction Quality Assurance
DSA	Direct shear apparatus
FEM	Finite element model
FDM	Finite difference model
FISH	FLACish (FLAC inbuilt subroutine compiler)
FLAC	Fast Lagrangian Analysis of Continua (numerical modelling code)
GCL	Geosynthetic clay liner
GM	Geomembrane
GT	Geotextile
HDPE	High density polyethylene
LLDPE	Linear-low density polyethylene
MSW	Municipal solid waste
OMC	Optimum moisture content
PPC	Pollution prevention and control
RRBES	Randomly reinforced bentonite enhanced soil
RSA	Ring shear apparatus

Appendices

- Appendix 1 Paper 1 Fowmes, G.J., Dixon, N. and Jones, D.R.V. (2007b).
Landfill stability and integrity: the UK design approach. *ICE
J. of Waste and Resource Management*. May 2007, WR2,
51–61.
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shear strength variability and its use in reliability-based
landfill stability analysis. *Geosynthetics International*. Vol.
13, No1. p 1.
- Appendix 3 Paper 3 Fowmes, G.J., Jones, D.R.V. and Dixon, N. (2005c).
Analysis of a landfill directive compliant steepwall lining
system. *Proceedings 10th International Waste Management
and Landfill Symposium*, Sardinia. pp369.
- Appendix 4 Paper 4 Fowmes, G.J., Dixon, N., Jones, D.R.V. (2006a) Use of
randomly reinforced soils in barrier systems. *Proc. 5th
International Congress on Environmental Geotechnics*,
Cardiff, UK, June 2006. 709 – 716.
- Appendix 5 Paper 5 Fowmes, G.J., Dixon, N. and Jones, D.R.V. (2007c)
Validation of a numerical modelling technique for
multilayered geosynthetic landfill lining systems. *Geotextiles
and Geomembranes* 26 (2008) 109–121.

1.0 Introduction

1.1 Research Context

Despite the increasing reuse and recycling of waste materials, there is, and will be, for the foreseeable future, a requirement to dispose of waste and waste processing end product to landfill. Landfills must be engineered to protect the environment from the harmful and hazardous compounds in the waste materials, such that the materials are separated from the environment until they reach a stable state at which they will no longer pollute. As sites available for landfill become increasingly scarce, and despite the greater technical challenges associated with the design and construction of lining systems for steep sided quarries, the use of such sites is becoming increasingly commercially viable.

A typical landfill lining system is shown in Figure 1.1 and comprises of a geological barrier (typically in the UK a low permeability fine grained soil) overlain by a geomembrane, thus forming a “composite barrier”. The geomembrane provides a very low permeability layer, however, geomembranes are susceptible to damage during placement and compaction of subsequent layers. Therefore, the underlying clay, when placed with good contact with the geomembrane, prevents migration of fluids which pass through defects in the geomembrane. A geotextile is typically used to protect the geomembrane from the overlying drainage layer to avoid puncturing and reduce strains in the liner induced by the overlying drainage material. Many configurations of the lining systems exist, whereby additional geosynthetic clay liners (GCL) may be included and geocomposite drainage layers used in addition to or *in lieu* of granular drainage (Jones and Dixon, 2003).

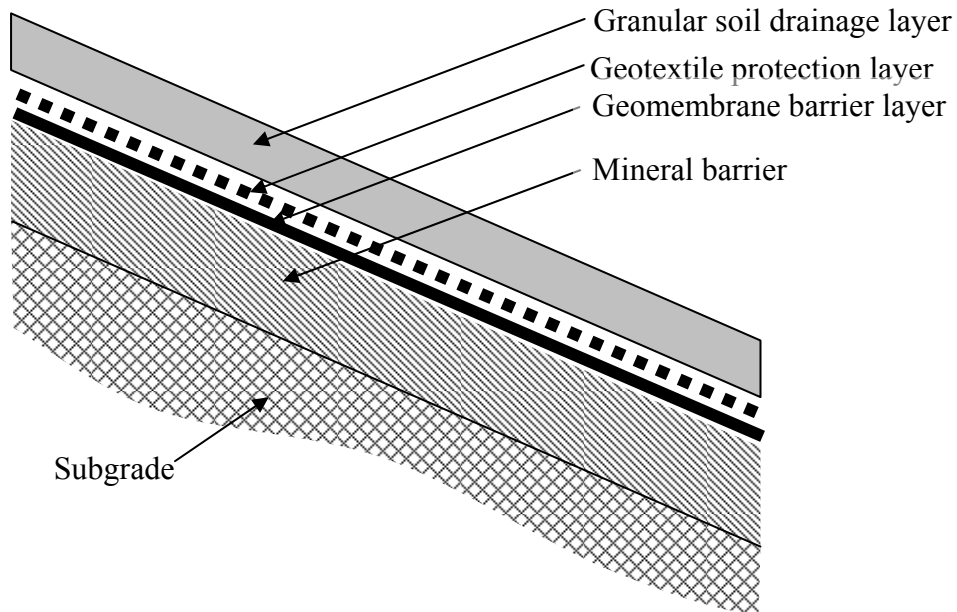


Figure 1.1 Typical landfill lining system components (reproduced from Paper 1)

In 1999 the EC Landfill directive was published, which is enforced in England and Wales through the Landfill (England and Wales) Regulations (2002). The Environment Agency Landfill Regulatory Guidance Note 6 (RGN6) gives an interpretation of the engineering requirements of the Landfill (England and Wales) Regulations (2002). This provides requirements of lining system construction and components, and has significant implications for steep sided landfill design with the *requirement* for inclusion of a geological barrier; usually in the UK this is compacted clay, along the base and up the side slope. Steep sided lining systems present a particular challenge as a geological barrier, granular drainage layers, geosynthetic and soil interfaces will exist at angles at which they are not naturally stable in the long term, and, therefore, support is required. Support can be provided by the waste mass, engineered fill buttresses, reinforced soil, or other engineered structures.

Under the EC landfill directive, waste is classified as hazardous, non-hazardous or inert. This project is primarily focused on municipal soil waste (MSW) landfill which is typically classified as non-hazardous, and references to waste and the waste body refer to MSW. MSW typically experiences large primary, secondary and creep settlements, thus making analysis of MSW landfill lining systems challenging. The

tools developed in this project can also be applied to hazardous and inert landfill lining systems.

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. Other definitions of a steep sided landfill suggest slope angles in excess of 30° are “steep” (Jones and Dixon, 2003). It is suggested that a classification of steep sided landfill be based on the stability of the internal components and the following definition of steep slope is suggested:

“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.” (Fowmes, 2007b)

Jones and Dixon (2003 and 2005) state that lining systems should be considered in terms of stability and integrity. Stability failure is considered as the ultimate limit state, where large scale movements occur with complete loss of function of the lining system, whilst integrity, the serviceability limit state, may involve small scale movements, resulting in overstressing, and hence loss of function in lining system elements such as geosynthetics and the geological barrier.

As landfill design has advanced, particularly in response to the EC Landfill Directive (1999), more complex multicomponent lining systems are being designed and constructed, particularly for steep sided lining systems. Analysis of multicomponent lining systems exposed to complex loads requires more sophisticated analysis tools than traditional limit equilibrium analyses.

One of the aims of this project is to develop and validate analysis tools for assessing landfill lining system behaviour, both in terms of stability and integrity, before, during and after construction. The analysis tools consider individual components and the behaviour of interfaces between them. The analyses also consider complex loading

scenarios arising from staged construction, waste mass loadings and waste settlement induced downdrag forces.

Input parameter variability is considered, in particular interface shear strength variability. This is of importance as typical design practice is to obtain only limited, if any, site specific parameters to justify designs. Additionally, the project also addresses areas of concern within typical designs and considers innovative materials to improve barrier response. The project can be divided in to sub-projects as listed below:

- Review of current state-of-the-art landfill stability and integrity analyses;
- Interface shear strength variability: Compilation of a large interface shear strength database and use of the derived values in probabilistic design;
- Development of numerical analysis tools for use in the design of steep sided landfill lining systems;
- Investigation into the use of fibre reinforced soil to reduce strains in mineral lining systems; and
- Validation of numerical modelling techniques using literature field data, site observations, and large scale field instrumentation.

Limit equilibrium techniques are typically used for lining system stability and integrity analyses, however, these do not take into account the complex loading conditions that can occur adjacent to a settling waste mass. Limit equilibrium analysis does not consider movements and strains required to mobilise strength in lining system components or at interfaces.

Section 2.0 introduced briefly models that can be used to assess stability and integrity of landfill lining systems, most of which have concentrated on particular aspects of the design. Whilst advances in interface modelling allow large displacements and consideration of individual geosynthetics (Villard *et al.* 1999), a modelling approach is required which can assess both stability and integrity for both geosynthetic and mineral lining systems, whilst also considering complex loading from waste weight,

waste lateral loading, and down drag. In addition, to model steep sided landfill lining system, additional engineering support structures need to be considered and the model needs be able to take this into account.

Villard *et al.* (1999) attempted to validate his model of lining system behaviour, however, this was only carried out for the unconfined state, (see section 4.7.1). This project is required to develop and to validate numerical modelling techniques, particularly, but not exclusively, with respect to shear strains and displacements in multiple layered geosynthetic lining systems. Further validation of numerical modelling is required to allow commercial application with confidence.

Whilst previous modelling using the FLAC code has considered a single strain dependent interface, to assess strain in geosynthetics, this project must include each geosynthetic as a separate layer, with an interface above and below.

1.2 Loughborough University waste research group

This project was conducted concurrently with three other research projects which researched the engineering behaviour of landfills. Figure 1.2 shows the areas of research carried out by the waste research group and the areas addressed by this project. The nature of the Engineering Doctorate programme allowed a broader spectrum of areas to be addressed than a typical PhD, and also allowed commercial application of developed techniques.

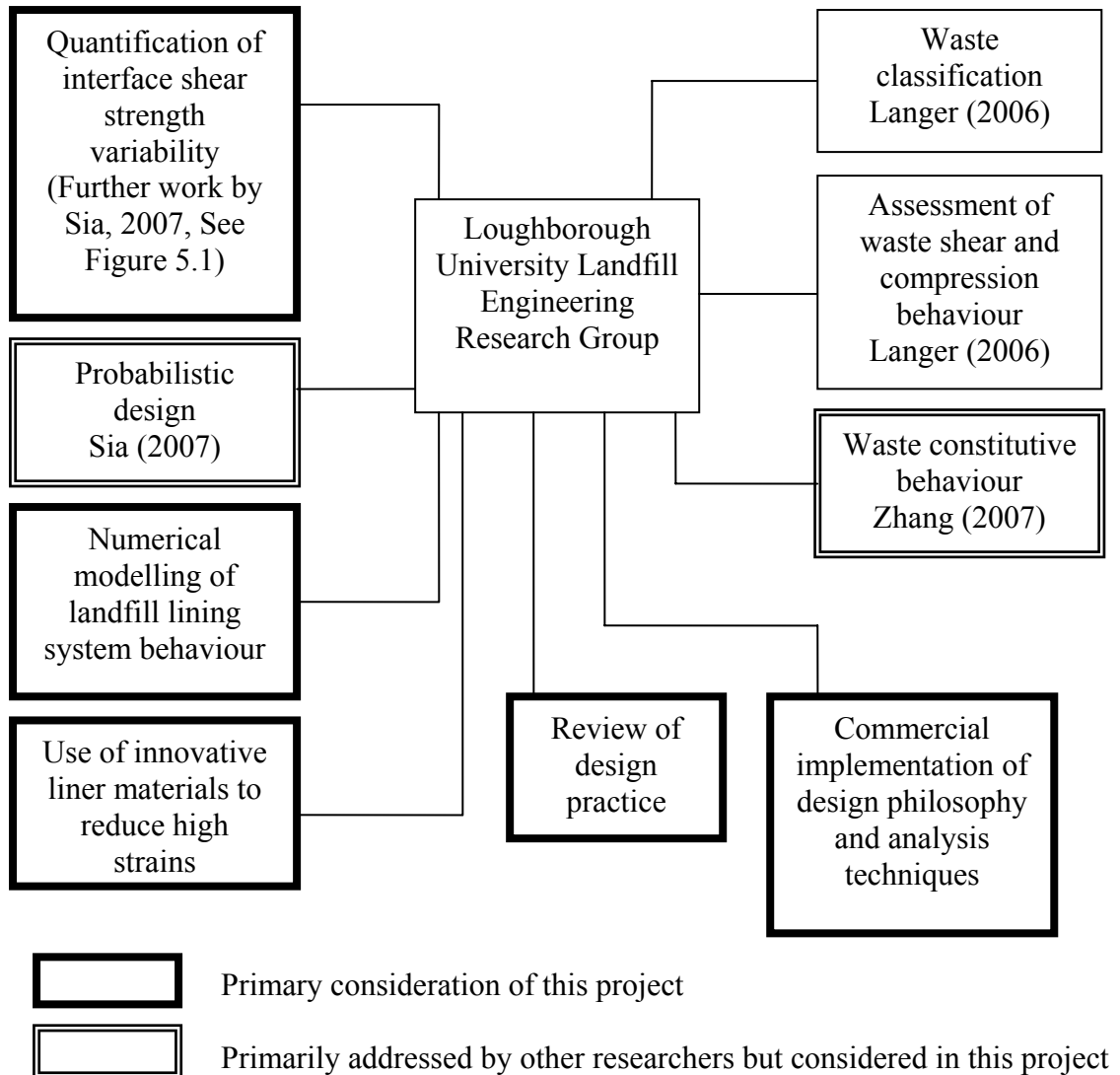


Figure 1.2 Loughborough University landfill engineering research group activities

1.3 Industrial Sponsor: Golder Associates

Golder Associates (Golder) is a consultancy specialising in a wide range of ground engineering and environmental market sectors. Golder was formed in 1960 and now has in excess of 6100 employees, 147 offices, operating from 28 countries globally (figures accurate as of 27th November 2007).

Golder is employee owned, and only permanent employees are able to purchase shares in the company. Share ownership is recognised at two levels, whereby,

combined with technical and managerial abilities, employees can be awarded Associate and then Principal status within of the company.

Golder was commissioned by the Environment Agency to produce the landfill engineering literature review and *de facto* guidance documents, R&D P1-385 TR 1 and TR2. (Jones and Dixon, 2003; Dixon and Jones, 2003), and Golder has a strong ethos for technical development and progress, not only in landfill engineering, but across the geotechnical and geoenvironmental engineering sectors.

1.4 Requirements of sponsoring organisation

Golder required the development of commercially applicable analysis techniques for use in landfill stability and integrity risk assessments of landfill containment systems. Analysis tools were required to assess both stability and integrity of multilayered lining systems, containing both mineral and geosynthetic lining components, and subject to complex loading conditions as a result of the adjacent waste material.

1.5 Aim

The aim is to improve and develop analysis tools and methodologies to be used for the assessment, in terms of stability and integrity, of landfill containment systems, particularly those involving steep side slope lining systems.

1.6 Objectives

The objectives of this project are:

1. To review current design state of the art and state of practice and to identify the critical controlling factors in the design of steep sided landfill lining systems;
2. To collate an interface shear strength database from all available internal (company and university) and published sources and to use the data to assess the variability, and the influence of this variability, on reliability of stability analyses;

3. To develop analysis tools for multicomponent landfill lining systems exposed to complex forces;
4. To investigate the use of innovative barrier materials for use on steep side slopes; and
5. To validate multilayered lining system design tools against laboratory and field data.

1.7 Justification of Objectives

The requirements for assessment of stability and integrity in UK design practice have increased since the Landfill (England and Wales) Regulations (2002) and the Environment Agency *de facto* guidance, P1-385/TR2, Dixon and Jones (2003) was introduced. Assessment of stability can usually be completed using conventional limit equilibrium analysis techniques, however, integrity was previously often ignored. This project, therefore, was required to develop an analysis framework for stability and integrity of landfill lining systems, and to identify, develop and validate tools for the assessment of integrity of the lining system.

The objectives were derived as a balance between the direct requirements of development by Golder and academic requirements to develop an integrated understanding of the behaviour of multiple layer steep sided landfill lining systems. Golder's primary requirement from the project was to develop a design toolbox to allow the analysis of steep sided landfill lining systems to be conducted. Work by Jones (1999) had demonstrated the capabilities of the FLAC numerical modelling code to analyse large displacement problems involving landfills, however, further development was required to produce commercially applicable design methodologies using the numerical code that allowed geosynthetic lining systems to be included and hence strains in lining system components to be assessed.

2.0 Background to research area

2.1 Landfill design practice

The Environment Agency has published a literature review (Jones and Dixon, 2003) on the stability and integrity of landfill lining systems, and a landfill design guidance document (Dixon and Jones, 2003). Paper 1 gives an outline of current UK design guidance for landfill containment systems, including steep sided landfill lining systems. This research updates the literature review by Jones and Dixon (2003) with additional literature published subsequent to the guidance. The paper is intended to provide a design framework identifying the factors that should be considered when assessing the stability and integrity of a landfill containment system.

2.2 Key landfill lining system components

Figure 1.1 shows a typical UK landfill lining system adopted on a shallow side slope. Jones and Dixon (2003) summarise alternative combinations which can include Geosynthetic Clay Liners (GCLs), Bentonite Enhanced Soils (BES) and geocomposite drainage layers *in lieu* of, or in addition to, the mineral barrier, geosynthetic barrier layer or drainage gravel. The barrier design will depend on the hydrogeological and gas migration requirements of the barrier and also the availability of local materials, however, the Landfill (England and Wales) Regulations 2002, now requires specific components of the lining system to be included.

2.2.1 Geological barrier

The Landfill (England and Wales) Regulations 2002 state that:

“(4) The landfill base and sides shall consist of a mineral layer which provides protection of soil, groundwater and surface water at least equivalent to that resulting from the following permeability and thickness requirements:

(a) in a landfill for hazardous waste: $k \leq 1.0 \times 10^{-9}$ metre/second: thickness ≥ 5 metres;

(b) in a landfill for non-hazardous waste: $k \leq 1.0 \times 10^{-9}$ metre/second: thickness ≥ 1 metres;

(c) in a landfill for inert waste: $k \leq 1.0 \times 10^{-7}$ metre/second: thickness ≥ 1 metres.

(5) Where the geological barrier does not meet the requirements of sub-paragraph (4) naturally, it may be completed artificially and reinforced by other means providing equivalent protection; but in any such case a geological barrier established by artificial means must be at least 0.5 metres thick.”

Whilst the requirements are for a geological barrier measuring 1m minimum thickness, paragraph (5) allows the reduction in this thickness with adequate risk assessment, of equivalent barrier protection. Although it may be possible to achieve the permeability and cation exchange requirements using geosynthetic clay liners (GCLs), the requirement for the barrier to be 0.5m minimum thickness precludes the use of these alone to represent the mineral liner. This is primarily due to concerns about puncturing and deformation of a < 500 mm thick layer. Paragraph (5) allows the inclusion of other barrier layers to account for the equivalent protection if the mineral layer is less than 1m thick and has a permeability in excess of $1.0 \times 10^{-9} \text{ ms}^{-1}$.

Reinforcement may be considered to provide support to the geological barrier, however, reinforcements which pass through the clay barrier can not be considered as there are concerns that a continuous foreign element may provide a preferential flow path for contaminant migration, either through the reinforcement material or at the interface between the reinforcement and the clay. Whilst the use of expansive clay materials, e.g. bentonite, could provide a seal between the reinforcement and the geological barrier, they would also provide a preferential slip surface, thus potentially compromising stability.

2.2.2 Artificial sealing liners

It is generally perceived that the artificial sealing liner will be represented by a low permeability geosynthetic layer. However, an artificially established barrier layer can, in fact, be an additional 500 mm of artificially established compacted clay (Jones, 2006, Pers. Com.). In steep sided landfill lining systems the requirement for an artificial sealing liner is usually met by a geomembrane, which is hung from temporary or final anchorage points during construction, and must be protected from

puncturing and overstressing due to waste downdrag and lateral stress imposed by the waste mass.

2.3 Steep sided landfill lining systems

Paper 1, Section 3.4 summarises the design issues associated with steep sided lining system behaviour. The following section addresses challenges in the design.

Two types of lining system are identified: self supported, which can be constructed to full height prior to waste placement, and waste supported, which rely on the horizontal support of the waste mass. Waste supported systems typically result in increase in void space and are cheaper to construct, although quantifying the horizontal support can be difficult.

Figure 2.1 and Figure 2.2 show the complex loading process and failure mechanisms that are typically associated with steep sided landfill lining systems, and show the key failure mechanisms that can occur. Figure 2.1 shows a generic post-Landfill Directive steep sided landfill on a benched quarry. Whilst Figure 2.2 shows a design that was adopted before the Landfill Directive, due to the absence of the geological barrier it is no longer acceptable design practice in the UK. Figure 2.2 shows the inclusion of a protection layer, in this case bulk sacks, filled with quarry fines, which are favoured to buffer the lining system from the waste.

The lining system is subject to loadings from the weight of the waste; however, steep sided landfill lining systems are also dependent on the waste for horizontal support. In addition, waste settlement applies frictional downdrag which is similar to the effect of negative skin friction on a pile. The designer must consider the stability of the structure as a whole, including the requirements for the support structure, and also the stability and integrity of individual lining system components.

Waste settlement comprises three main components: primary settlement due to the overburden of subsequent waste placement, creep settlement, and degradation induced volume loss settlement. The primary compression of MSW is can be significant, however, as these occur during waste placement, it is not observed at the surface as further waste placement conceals the settlement of the previously placed material.

This project focuses on the primary settlement during placement of waste in lifts, and subsequent overfilling of additional lifts to account for the settlement of previously placed material. Analysis of creep and degradation induced settlement are beyond the scope of this project.

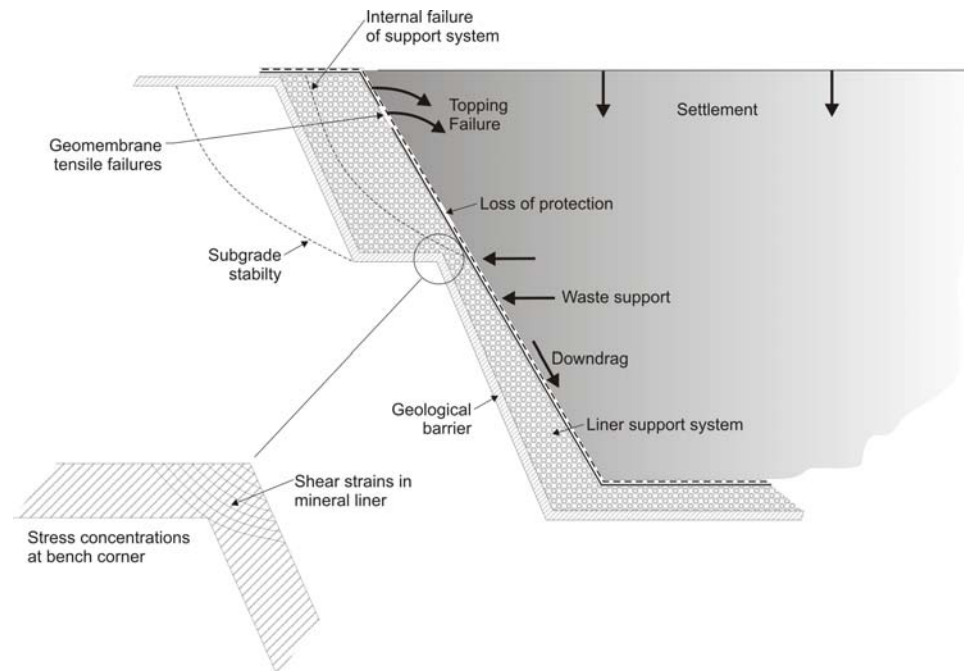


Figure 2.1 Steep sided landfill lining system loading failure mechanisms (reproduced from Paper 1)

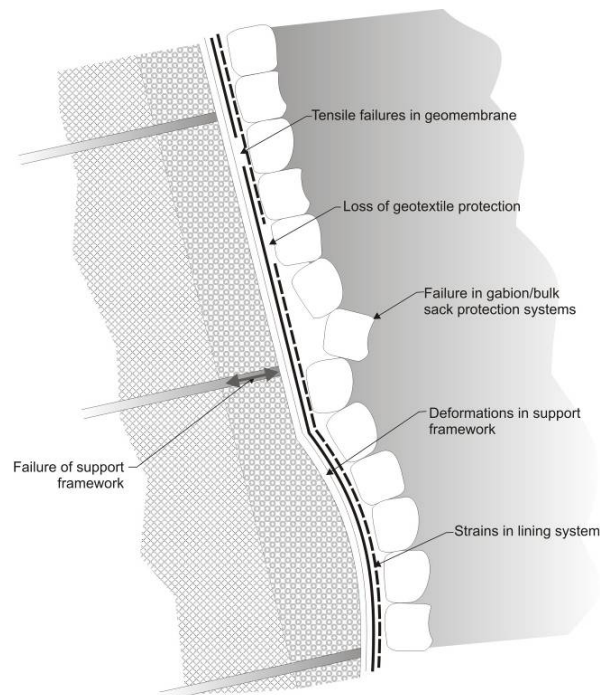


Figure 2.2 Steep sided landfill lining system failure mechanisms

With the introduction of the requirement for a geological barrier, support for the barrier is typically provided by engineered fill support buttress systems. The support buttress can contain reinforced soil which comprises horizontal reinforcing elements that allow construction of near vertical systems several metres ahead of the waste.

Reinforced soil designs such as that shown in Figure 2.3 are becoming more common, however, mineral only “Christmas tree” (see Figure 2.4) and vertical clay barrier lining systems have been implemented, as these systems include a geological barrier and are cheap to construct. However, stability is of major concern as the clay overhanging the waste in a Christmas tree lining system tends to induce rotation as waste settlement occurs under it, and vertical clay barriers are prone to toppling failures due to insufficient waste support. Dixon *et al.* (2004) instrumented a mineral only vertical clay barrier lining system and showed that, despite the high short term undrained strength of the clay and the waste support, the barrier experienced significant strains.

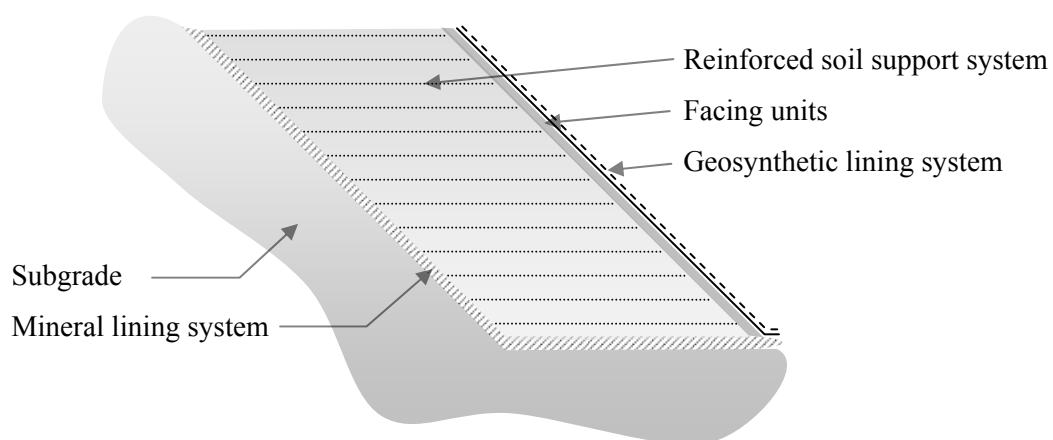


Figure 2.3 Reinforced soil mineral liner support system

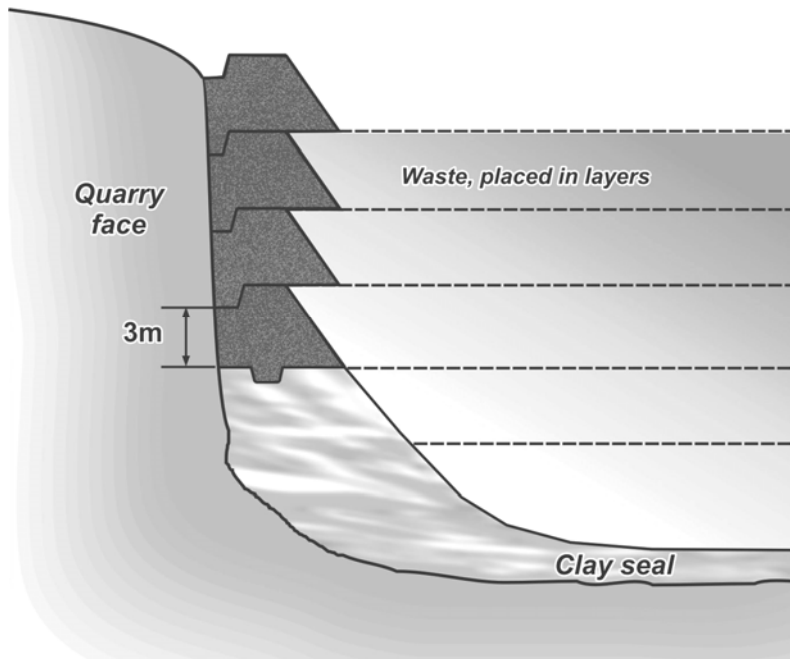


Figure 2.4 “Christmas tree” landfill lining system (after Jones and Dixon 2003)

2.4 Reliability based landfill design

Traditional design practice only requires a single lumped factor of safety value, and it is the responsibility of the engineer to make an assessment of the overall uncertainty in the model and input parameters. This responsibility is often overlooked, and generic lumped factors of safety used irrespective of the input parameter variability.

Reliability based design aims to quantify the actual variability of individual input parameters. In BS EN ISO 1997-1:2004 (Eurocode 7) this is addressed through the application of partial factors representing the perceived variability in the input values. Interface shear strength variability is beyond the scope of BS EN ISO 1997-1:2004, and hence typical lumped factors are used in design.

An alternative to partial factors is to prescribe a statistical variation function (usually a standard deviation) to each input parameter and apply this in a reliability based design. Landfill reliability analyses were carried by McCartney *et al.* (2004) and Koerner and Koerner (2001), who considered variability on shear strength and its influence on veneer stability, and Sabatini *et al.* (2002) who considered waste mass stability behaviour, and show that lumped factors of safety are not appropriate to represent the variability associated with interface shear strength.

2.5 Numerical modelling of landfill lining systems

Limit equilibrium analyses for landfill liner stability are still typically adopted in UK design practice, however, Reddy *et al.* (1996) identifies that limit equilibrium techniques lack the capability to compute displacements along critical shear planes and the resulting strain levels, and, therefore, numerical analyses are required.

Jones and Dixon (2005) stated that the design of landfills must consider stability both within and between elements of the lining system. To address this they used the limit equilibrium techniques and the explicit finite difference modelling code, FLAC, to assess the stability of a waste mass with a single strain dependent interface between lining elements representing the weakest layer of the system. In addition, comparisons between measured and predicted direct shear box behaviour were included to validate the modelled response of a single interface. Jones and Dixon (2005) stated that integrity should consider the loss of protection and geomembrane overstressing, however, with only a single interface, tensile response of the geosynthetics could not be assessed.

A survey of a sample of submitted Pollution Prevention and Control (PPC) applications in England was carried out as part of this project following Tranche 1 to 4 of PPC submissions, to identify the steep sided landfill lining systems, and analysis methods applied by a range of consultants. Whilst this study was not intended as a comprehensive study of current practice, it showed that numerical analyses were applied in a limited number of cases, and that where numerical analysis was applied, FLAC and PLAXIS were the primary codes used. A similar study carried out by Terry (2006) indicated that FLAC was the most commonly used numerical analysis technique for steep sided landfill lining systems. Limit equilibrium analyses were standard practice, as they are well suited to the stability analyses generally required in the PPC analysis. However, it has been observed, particularly for steep sided landfill lining systems, that numerical modelling is becoming increasingly common place.

Several attempts to analyse landfill stability and integrity using numerical techniques have been reported. These are summarised below.

Byrne (1994) carried out back analysis of Kettleman Hills landfill failure, with FLAC, and demonstrated that numerical techniques could be used to represent the progressive failure mechanism. Filz *et al.* (2001) also back analysed the translational failure at Kettleman Hills landfill and showed, using limit equilibrium analyses, that the maximum height using residual interface shear strength parameters was only 10% lower than the actual height, and using peak interface shear strength parameters the maximum height achievable was 35% greater than actually constructed, thus concluding that large post peak interface shear strength reduction had occurred. Numerical techniques with strain softening interface were used to show the progressive failure mechanisms, involving post peak shear strength reductions. These analyses concentrated on landfill stability failure and not integrity failure as a function of such strain softening behaviour.

Meißner and Abel (2000) used numerical techniques to consider geomembrane tensile forces from time dependent waste degradation. However, this model did not consider strain dependent interface behaviour.

Integrity analyses combined with strain softening interfaces were considered by Long *et al.* (1995) who used finite difference modelling to assess the integrity of a landfill lining system due to waste settlement loading. Complex waste loadings were simplified into applied loads and displacements imposed as boundary conditions at the upper surface of the lining system, thus limiting application in design practice.

Villard (1996) presented a model capable of representing large displacements at non-interpenetrating interfaces. Individual lining system elements were considered in this model and both stability and lining system integrity could be considered. Villard *et al.* (1999) used field data to attempt to validate this model; this is discussed in greater detail in Section 4.7.1.

Interface behaviour and formulation is summarised by Villard (1996), who considers three methods of interface formulation: (1) Penalisation or stiffness methods, where thin or zero thickness spring elements are used to define normal and shear stiffness; (2) Nodal compatibility methods, where contact at an interface is satisfied by force displacement compatibility equations; and (3) Hybrid interfaces, which are a

combination of (1) and (2) where the two parts are modelled separately and linked through constraint equations. Villard (1996) proposed a hybrid method that allows large displacements to occur with no interpenetration. Jones (1999) considers the interface methodology in FLAC where large displacements are also possible, although unlike Villard *et al.* (1999), only a single interface was considered so geosynthetic strain could not be assessed.

A variety of constitutive models for MSW have been considered: Meißner and Abel (2002) use a linear isotropic elastic model, Byrne (1994) and Jones and Dixon (2005) use a Mohr-Coulomb model and a hyperbolic model is adopted by Reddy *et al.* (1996). Machardo *et al.* (2002) and Krasse and Dinkler (2005) consider model behaviour of waste in terms of multiphase components where reinforcing elements and paste are considered separately, thus allowing improved representation of waste behaviour. Zhang (2007) included the influence of reinforcing elements and has also considered the influence of compressible particles.

Whilst modelling of multiple layered lining systems and waste mass stability has provided significant advances in analyses, the ability to include the complex components of steep sided landfill lining systems are not included in these models.

Byrne (1994) and Jones (1999) considered the use of FLAC to model slippage along a strain softening interface under a waste slope. Mohr Coulomb failure criterion was defined for the waste and for the interfaces, with displacement dependent failure envelope defined for the interface behaviour.

Connell (2002) used FLAC to model simple steep sided landfill lining systems and Connell (2005, pers. corr.) has considered complex loading acting on mineral steep sided lining systems, however, this work does not consider multiple layer geosynthetic interfaces, with strain dependent behaviour.

Chugh *et al.* (2007) use FLAC to reanalyse the slope stability failure near Cincinnati, Ohio, USA. Two and three dimensional continuum modelling was used to study the onset of instability, the failure surface location and geometry and the displacements that occurred. This study did not include geosynthetic integrity analysis.

Whilst the current state of practice allows individual aspects of multilayered lining system behaviour to be modelled, the requirement for this project was to develop an analysis technique that can include multilayer geosynthetic and mineral lining systems, and have the flexibility to consider real world landfill geometry. For steep sided lining systems the numerical modelling approach should also have the ability to model waste lateral support and the behaviour of engineered support structures.

3.0 Research methodology and task breakdown

This chapter identifies the research philosophy adopted during this project and summarises the individual task breakdown and places the tasks in relative context with one another and with the wider industry. A summary of the research tasks is presented in Table 3.1 and the context is addressed in Section 3.3.

3.1 Research methodology

Neuman (1997) describes a typical multi step research process taking the planning, choosing the topic, focussing on the research question, developing and designing the study, collecting information, analysing, and interpreting the data and finally moving to informing others. In practice this research method is far more of an interactive process in which the steps blend into each other, not linear but flowing in several directions. Research is an on-going process that can stimulate new thinking and fresh questions developing more issues than it actually answers.

There is often a tendency to distinguish between quantitative and qualitative research. However, it is believed to be inappropriate to make a hard distinction between qualitative and quantitative studies as they can often be used in combination with great effect. It would be inappropriate for a researcher to become so convinced of the paradigm they are using that they deny other methods. Engineering research, by its nature concentrates around data acquisition and verification analysis and validation, and, therefore, typically falls into a qualitative categorisation; however, an attempt has been made to also include subjective techniques, based around experience of practice and observation of and discussion with practitioners in order to direct the qualitative research.

3.2 Research task breakdown

In order to address the objectives identified in Section 1.6, the research required quantitative review and analysis of collected data. This was followed by experimental research using laboratory testing and numerical methods. Iterative development of numerical techniques was carried out using quantitative data gathered from both published and experimental sources. Whilst qualitative methods were adopted in eliciting the opinions of experts during the project, the primary focus was production

of a numerical analysis tool, which, by necessity, requires the use of quantitative experimental research techniques.

The objectives were approached through a series of individual research tasks, which are described in Table 3.1. The applied research methodology and the associated deliverables, in terms of published papers and technical reports are also given in Table 3.1. The research methodologies applied can be summarised into the following:

- **Literature review.**

Collecting information and data from published literature, internal company sources, internal university sources and personal correspondence and expert elicitation.

- **Experimental research**

Objective research, development of analysis tools, quantitative interrogation of hypothesis and validation of findings.

- **Commercial Application**

The application of developed techniques and tools in a commercial context to test performance, relevance, usability.

Table 3.1 Research tasks and associated publications

Research task	Research methodology	Objective	Technical Report	Paper
Review state of the art UK design philosophy	Literature review.	1	Analysis Toolbox report (Fowmes 2007b)	Paper 1
Develop an interface shear strength testing database from all available interface shear strength data	Literature review.	2	Variability of Shear Strength Data Report (Fowmes 2004a)	Paper 2
Consider the effect of the variability on the on the reliability of design		2		
Development of FLAC modelling to include multiple layer lining systems	Experimental research.	3	FLAC modelling reports 1 and 2 Fowmes 2004b and 2005b)	Paper 3, Paper 5, Fowmes <i>et al.</i> (2006b) and Fowmes <i>et al.</i> (2007a)
Code production using FLAC to develop strain dependent interface		3		
FLAC modelling of multilayered steep sided landfill lining system		5		
Randomly Reinforced Soil (RRS) literature review	Literature review.	4	RRS Literature Review (Fowmes 2005a)	
RRS laboratory testing programme	Experimental research	4		Paper 4
Commercial application of developed modelling techniques	Commercial application.	3	Analysis Toolbox report (Fowmes 2007b)	
Analysis of South East Asian landfill failure using these techniques		5		Fowmes <i>et al.</i> (2006b)
Back analysis of reported lining system interface behaviour using FLAC		5	FLAC modelling report 2 (Fowmes, 2005b)	
Design of instrumentation for validation of numerical analyses, field and laboratory analysis	Experimental research.	5	Validation of numerical methods report (Fowmes 2007a)	Paper 5
Laboratory testing of multilayered landfill lining systems		5		
Validation of FLAC model against laboratory testing results		5		
Assessment of the influence of horizontal welded geomembrane seams		Additional		
FLAC waste behaviour models and strain dependent geosynthetics		3		
Development of analysis toolbox for the analysis of landfill stability and integrity	Experimental research and commercial application.	Additional	Analysis Toolbox report (Fowmes 2007b)	

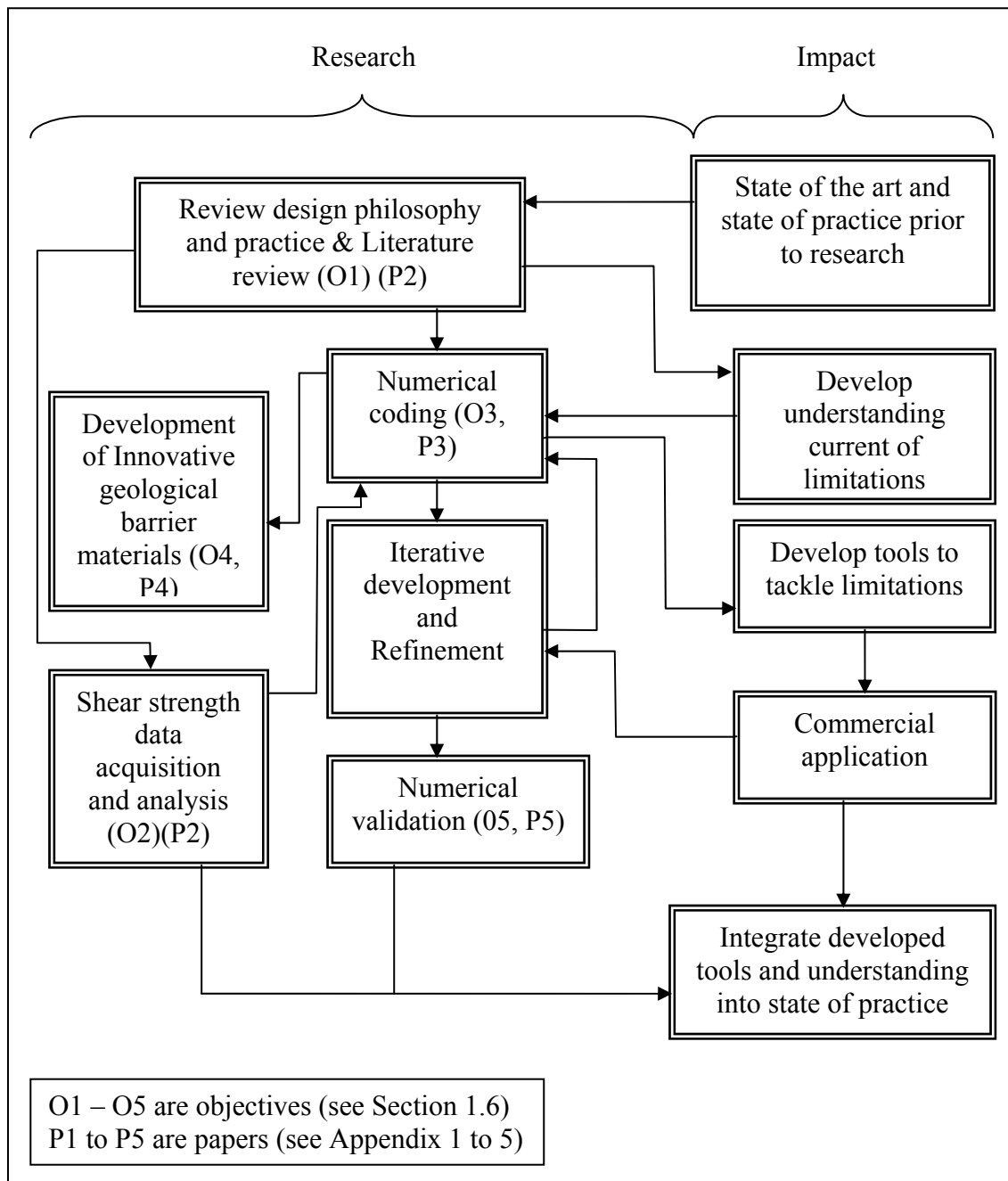


Figure 3.1 Research map

3.3 Research overview

The first stages of the research concentrated around identifying both state of the art and state of practice. It was identified during this stage that the interface shear strength behaviour was a controlling factor for steep sided landfill lining system integrity and stability (objective 1). At this stage of the development the requirement for experimental research into the interface shear strength variability (objective 2) and the requirement for development of a numerical analysis tool (objective 3) were identified. Following the development of a numerical analysis tool (objective 3), high shear strains in the lining system were also identified as a potential problem, thus resulting in the requirement for investigation into the use of high strength, high plasticity geological barrier layers (objective 4). “The numerical analysis tool allowed analysis lining systems exposed to complex loading scenarios, however, the commercial application required greater confidence in the response of the model, and hence validation against measured data was required (objective 5).

3.4 Methodology Summary

The research methods have been identified for each of the objectives in Table 3.1 and the inter-relationships identified in Figure 3.1. By the nature of engineering research, the main focus has been on literature based understanding followed by quantitative experimental research. In addition, the EngD programme has allowed commercial application of the developed tools, allowing iterative refinements to be made.

4.0 Research Summary

This section, in addition to the papers in Appendices 1 to 5, details the work that has been undertaken and the key findings and developments of this research project. This is not intended to provide a complete report on the research, but to provide a commentary to the published papers included in the appendices.

4.1 Review of landfill design philosophy

A review of current design methodologies and practice, including current legislation and technical state of the art has been carried out. Paper 1 details the current UK design approach, in terms of stability and integrity, for landfill containment systems, addressing the analysis considerations for 6 main components of landfill containment system (see Paper 1, Figure 2):

- Subgrade;
- Basal lining system;
- Side slope lining system;
- Steep side slope lining systems;
- Waste mass; and
- Capping lining system.

Each section is considered in terms of the design detail which must be satisfied prior to construction of the lining system. Paper 1 Figure 3 identifies the primary design considerations for each aspect of design and these are considered in greater detail in Paper 1, Section 3 and Figures 4 to 9.

Although each aspect of design should be considered, this does not imply that a calculation must be done in every case. A logical argument can be put forward as to why a particular failure mechanism is not considered to be likely. Where analyses are required, the complexity of the analysis will depend on the lining system components and configuration. This will then control the skills, effort and information required to conduct the analyses.

4.2 Controlling failure mechanisms

Bergado *et al.* (2006) and Koerner and Soong (2000) identified interface sliding as a major cause of landfill failure. Jones and Dixon (2003) conducted a survey of UK landfill failures and showed interface shear strength to be a critical controlling factor, yet with the greatest uncertainty associated with it. Other controlling mechanisms involved the waste strength, and the construction quality. There is a dearth of information on the mechanical behaviour of municipal solid waste, and it is beyond the scope of this research to quantitatively assess the uncertainty surrounding waste behaviour, however, attempts to improve commercially applicable constitutive models have been made. Recent improvements in regulation have been made, and it is believed that the requirements for construction quality assurance (CQA) will reduce failures caused by substandard construction practice.

4.3 Interface shear strength variability

BS EN 1997-1: 2004 (Eurocode 7) states that characteristic values should be selected as “a cautious estimate of the value affecting the limit state design”. Current practice is to carry out a limited number of site specific tests, however, this provides insufficient information on the variability of the parameter to allow such a cautious estimate to be made. The work presented in Paper 2 has been carried out to provide a global database of interface shear strength parameters and an assessment of variability thus aiding the designer in selection of a ‘cautious estimate’.

Jones and Dixon (1998) presented interface shear strength summary plots for use in preliminary design, however, there is evidence that these have been used *in lieu* of site specific testing. In such practice, global variability will apply and the associated probability of failure values may be unacceptably high, even with an apparently acceptable global factor of safety.

It is worth noting that a cautious estimate of a value affecting the limit state is not always lower than the average. In stability analysis, values of interface shear strength lower than the average will typically result in lower factors of safety, however, in terms of integrity, higher values of interface shear strength may result in greater transferred stress and hence greater tension in the geosynthetics, therefore, a higher

value would be cautious. An example is shown in Figure 4.1, where for a stability analysis low values would be selected for all interfaces, however, when assessing tension in a geosynthetic element all interfaces above the geomembrane should be assigned cautiously high values and those interfaces below the geomembrane should be assigned cautiously low values.

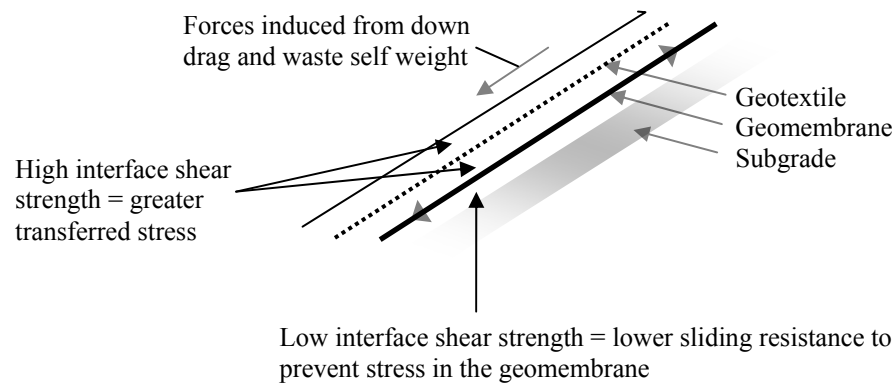


Figure 4.1 Cautious estimate interface shear strengths for geomembrane tensile stress analysis

The objectives of this research were to develop a database of interface shear strength data for interfaces between commonly used geosynthetic and soil materials, and to define the distribution at a range of normal stresses. In addition, this data could be used to provide information on the statistical variability of interface shear strength and its impact on the design process.

It had been identified in previous studies (e.g. Dixon *et al.*, 2002; Dixon and Jones 1998) that significant variability was present in the data for interface shear strengths thus a comprehensive literature search was conducted. The literature search covered journals and conference proceedings along with in-house data from Golder Associates and Loughborough University. European inter-laboratory comparison tests were also included in the analysis. The full reference list for the database is given following Paper 2. The data was divided into categories dependent on the interface material (e.g. polymer type and textured/smooth surface) to give a database of interface shear tests for 22 generic interfaces, which has been compiled consisting of over 4200 data points each representing the measured peak or residual shear stress at a given normal

stress. The data for 7 of these interfaces was included in Paper 2 to analyse probability of failure. The appendix to Paper 2 also includes the full reference list for the database, which was not included in the published paper.

A similar database has been produced by Koerner and Narejo (2005) which again demonstrated the large variability associated with interface shear strength literature and inter-laboratory testing programmes values. Following publication of Paper 2, the Fowmes (2004a) and Koerner and Narejo (2005) databases have been combined by Sia and Dixon (2007), with care to remove double counting of references) to produce a combined dataset against which values can be compared.

The global data set contains several sources of variability, which include (Stoewahse *et al.*, 2002):

- Differing material types
 - Polymer types and additive
 - Age and previous UV/stress exposure of samples
 - Types of texturing (blown film, impinged, etc...)
 - Soils type and moisture content
- Laboratory equipment
 - Shear apparatus (e.g. DSA, RSA)
 - Shear box design and size
- Test conditions
 - Shearing rate
 - Temperature
- Different equipment operators

Statistical analyses were carried out for data within each of the categories (where there were > 5 data points at a given normal stress $\pm 5\%$). The first and second order statistical moments were obtained for the input and equations derived for the statistical variability as a function of normal stress. Table 1 in Paper 2 details the mean interface shear strength parameters in terms of interface friction angle, δ , and apparent adhesion (y-intercept), α , and also gives slopes for the standard deviation in

peak and residual shear strength as a function of normal stress. Normal distributions were assumed for the interface shear strength. Sia and Dixon (2007) carried out subjective and objective statistical tests and showed that the normal distribution is appropriate for interface shear strength parameters.

To avoid the need for consideration of linked pairs in the statistical analyses (i.e. α and δ) the interface shear strength is considered in terms of shear stress at a given normal stress, with values for standard deviation derived at each normal stress level.

Several methods of considering variability are used in design:

- Use a single lumped factor of safety;
- Use partial factors of safety; or
- Reliability analysis based on the measured statistical variability of the input parameters.

A single lumped factor of safety is typically applied in UK landfill design, however, the introduction of BS EN 1997-1:2004 (Eurocode 7) will require the use of partial factors of safety. A more statically robust method, which is also applicable under BS EN 1997-1:2004 is to define the statistical variability of each input parameter to determine the probability of failure of the system. Reliability analyses were carried out in this project, and were a progression from those by McCartney *et al.* (2004) and Koerner and Koerner (2001), who considered variability on shear strength and its influence on veneer stability, and Sabatini *et al.* (2002) who considered waste mass stability behaviour.

The coefficient of variation (COV) of the measured shear strengths was calculated at given normal stresses, using:

$$COV = \frac{\sigma_m}{X_m} = \frac{\text{standard deviation}}{\text{mean}} \quad \text{Equation 4.1}$$

Probability of failure is calculated as a function of the COV of the factor of safety and the FS_{MLV} (most likely factor of safety). Tables in Koerner and Koerner (2001) were

used to calculate the factor of safety. These are based on a lognormal distribution for factor of safety which is considered appropriate by Duncan (2000) who states that FS is log-normal, however, this does not imply that the values of individual variables must be distributed in this way. Appendix 1 of Paper 2 identifies the analysis methodology used in assessment of the Probability of Failure. The Table of probability values from Koerner and Koerner (2001) is also included as an appendix to Paper 2.

Negative 'y-intercepts' have been allowed for best fit lines for both mean shear strength parameters and standard deviation parameters. In the cases where negative values occur they are produced by best fit lines through a number of the data sets included in this paper, and these demonstrate limitation of data sets in terms of number of points and their distribution, hence they should not be forced through the origin.

Variability due to different testing laboratories conducting tests on the material from the same source has been shown to be as high as the variability in the total data set including different material types. This implies that the testing procedures need to be standardised. An electronic database of the interface testing has been compiled which allows the designer to compare values from testing to the database showing variability, thus allowing an assessment of the confidence in the magnitude of the measured value to be made.

Additional complications such as pore fluid pressure and temperature may also influence interface behaviour. Jones and Dixon (1998) identified that the interpretation of interface shear strength results can depend on the pore pressure conditions at the interface, and interpretation can be challenging if these are not recorded. Sharma *et al.* (2007) used a miniature pore pressure transducer to measure pore pressure at a soil – geomembrane interface and showed that soil suctions contributed to increased shear strength at low normal stress, however, at higher normal stress the strength appears to be governed only by the total normal stress.

Criley and Saint-John (1997) presented repeatability testing for geosynthetic interfaces, and this data showed significantly reduced variability when compared to

the global dataset, as these were carried out by the same user, in the same laboratory on the same material types.

Dixon *et al.* (2002) identify that the typical variability of test data for a particular interface may be used to derive a characteristic value. However, the derived probabilities of failure from the datasets are very high even with factors of safety in excess of 1.5 (see Paper 2 Figure 15 and 16). Whilst failures do occur in practice, the probability of failures suggested using global datasets are unrepresentatively high, and suggestions that a factor of safety nearing 2 would result in over conservative designs. This does, however, highlight the dangers associated with the selection of literature values with no site specific information and without an appreciation of variability as vastly different design parameters can be selected depending on the source. Paper 2 allows the designer to compare site specific testing to a large global dataset, and identify the position of obtained results relative to the data.

Sia and Dixon (2007) compared the datasets assimilated as part of this research to repeatability testing of interface shear strength and found that variability using global databases was 3 to 5 times (sometimes up to 8 times) higher than variability when repeatability tests were carried out. Sia and Dixon (2007) concluded that variability and uncertainty computed using global and inter-laboratory datasets yield overly conservative outcomes when adopted in design.

Whilst preliminary designs can be carried out based on literature values, site specific testing must be carried out prior to construction to develop rigorous designs. It is not recommended that the variability associated with the global database, reported in Paper 2, is used in conjunction with site specific testing, however, it would be prudent to compare the site specific results to a global database to avoid the use of erroneous results in design (Dixon *et al.*, 2002). Koerner and Koerner (2007) discuss the adoption of interface shear strength parameters derived from interface shear strength testing (in accordance with ASTM D5321-02), whilst no discussion of variability is given, this document describes state of practice. The role of the designer to subjectively select 'conservative' design parameters is relied upon. In reliability based designs under Eurocode 7, designers must be confident in justifying the parameters

that they have selected and comparison to global databases and previous experience will aid the designer to carry out this task.

4.4 Numerical analysis in landfill engineering

A primary aim of this project was to develop analysis tools for multicomponent landfill lining systems exposed to complex forces during and post construction. An internal research and development report has been produced for Golder Associates detailing the methodologies used in the design process and how these are applied. This section gives details of the analysis tools that have been developed, the development process and when and how they can be applied.

Section 2.4 summarises the state of the art in numerical modelling of landfill lining systems, and identifies the requirement for a model which can include multiple layer geosynthetic lining systems *and* mineral lining systems, whilst including the engineered support structures and complex geometries typically found in steep sided lining systems.

4.4.1 Selection of numerical modelling code

The need for the direct applicability to industry required the selection of a commercially available, proven and widely used numerical modelling package to carry out analyses. The finite difference numerical explicit modelling code FLAC (version 4.00) has been selected to analyse side slope lining systems primarily due to its ability to model large strains and previous applications of the code for analysis of landfill lining systems. Byrne, (1994), Jones (1999), and Connell (2002) have used FLAC for assessing lining system integrity, and it is suggested in the Environment Agency *de facto* landfill stability guidance R&D Technical Report P1-385 TR2.

There are a large selection of finite element and finite difference modelling codes on the market. The majority of these codes carry out calculation within a continuum, where a series of adjacent nodes and/or elements are used to define the system to be modelled. Finite element programs have the central requirement that the field variables vary through each element, and the formulation consists of adjusting these to minimise error. It is typical for finite element programs to combine element field variables to produce a global stiffness matrix. In finite difference formulation, every

derivative in the set of governing equations is replaced by an algebraic expression, with the resulting variables written in terms of stress or displacement, and these remain undefined within the element.

In finite difference formulation, as there is no formation of a stiffness matrix, it is efficient to reproduce the equations, therefore allowing the use of an explicit method to solve the algebraic expressions (Itasca, 2002). The explicit solution mechanism is adopted in FLAC, and the solutions are reached through a process known as time-marching or timestepping, which is simply adjusting the values of each node in the mesh through a series of cycles or steps. These adjustments take place on the basis of the selected constitutive model and equations of motion. The adjustment continues until the error (e.g. unbalanced force in the system) becomes very small (Sivakugan, 2006). The calculation cycle adopted in obtaining a solution is shown in Figure 4.2. Explicit formulation is well suited to non-linear problems and those involving large strain.

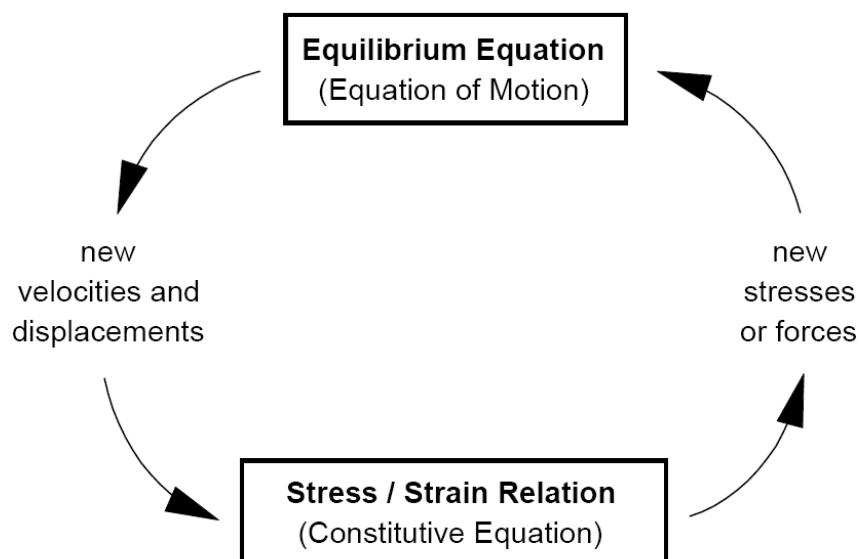


Figure 4.2 Calculation cycle in FLAC (Itasca 2002)

FLAC allows Lagrangian formulation. This calculation method is not tied to a fixed co-ordinate system, and the co-ordinate system and the co-ordinates are independent variables of the model. For geotechnical numerical analysis using a finite difference modelling mesh, Lagrangian formulation allows the position of the grid zones to

update continuously throughout the solution. Over a large number of time steps, the points from which the calculations are being carried out in a Lagrangian analysis can significantly deviate from their starting positions. This is particularly useful in landfill engineering where large waste displacements are commonplace. Whilst other codes such as PLAXIS have large strain options, interface behaviour is limited by the interface elements maintaining the continuum (see Figure 4.3), and thus large scale relative movements of the modelling grid cannot occur. In FLAC interface elements require a break in the continuum (see Figure 4.5), therefore, during Lagrangian analysis, large relative displacements of the co-ordinates on either side of an interface can occur without concern for deformation of the continuum. This is particularly beneficial when considering waste behaviour, as large relative displacements can occur between the waste mass and the lining system. Imposed restrictions due to the modelling continuum may result in incorrect stress transfer, waste stress state, and a failure to represent the unconfined region generated by waste settlement.

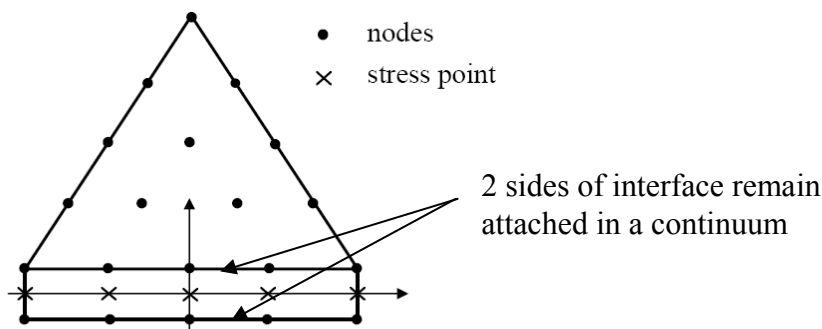


Figure 4.3 Continuum interface elements in PLAXIS (attached to 15 node soil elements)

Industry requires advances in the use and applicability of numerical modelling codes, and whilst other authors have developed bespoke interface and liner modelling codes, none have the ability to consider all factors influencing steep sided landfill performance, in particular, deformations in the geological barrier and the behaviour and affect of engineered support systems. FLAC, and its associated FISH programming language allows advanced modelling of multilayer interfaces to be developed, whilst retaining the flexibility of a commercial modelling package allowing a wide variety of geotechnical structures to be modelled.

4.4.2 FLAC Numerical modelling terminology and features

In order to understand the modelling morphology, it is necessary to define some basic components within the FLAC model. A labelled simple modelling grid is included in Figure 4.4.

- **Grid.** The modelling grid is constructed from rectangular elements, and subdivided into triangles for calculation purposes. The rectangular grids are systematically numbered in i,j space and may be moved to represent the geotechnical structure in x,y space.
- **Nodes.** Nodes are located at the corners of the rectangular grid elements and are systematically numbered in i,j space.
- **Structural Beam elements.** Structural beam elements form linear structures in the model, outside of the grid zones, and may be attached to grid zones, or interact with grid zones through interfaces.
- **Interfaces.** Zero thickness interface elements are used to define connections between grid continua and structural elements. Interface behaviour is controlled by normal and shear stiffness with limiting shear strength values allowing slippage. (See section 4.4.3).
- **$x,y/i,j$ space.** The x,y coordinates are the coordinates in global reference space with constant scale (see Figure 4.4) whereas the i and j coordinates refer to the FLAC grid (see Figure 4.5). If the grid is deformed and reshaped, the x and y coordinates (i.e. the material positions) of grid points will alter but the i and j coordinates will remain constant. The x,y space is used for constructing the geometry, whilst modelling, i,j space is used to define material types and interactions. Structural nodes are defined in terms of x,y space. In i,j space structural nodes have a non zero i value and a $j = 0$.

4.4.3.1 Interface stiffness

Interface stiffness controls the initial stress displacement response of an interface until a limiting shear stress value, controlled by the interface strength, is reached. The interface shear strength is defined in Pa/m (Pascals per metre), defined as shear stress per unit displacement, rather than unit strain in which stiffness would be defined as Pa. Selecting an unrealistically high value can result in restricted movement and cause calculation difficulties, resulting in long solution times, or no solution being achieved. However, selecting a value that is too low can result in excessive movement as strength is not mobilised, and in the worst case, peak strength values can be missed from the calculations. Figure 4.6 demonstrates the affects of setting the interface stiffness too low. The test shows a textured LLDPE geomembrane against a non-woven geotextile tested at 10 kPa confining stress. The 10 MPa/m represents the initial tangential modulus, whilst the 1MPa/m represents a secant modulus at 10mm displacement; these can be seen to generate an acceptable correlation with the measured shear strength data from laboratory direct shear tests. The small discrepancies in the measured and recorded values are due to the data points used to define the strength envelopes not plotting on a perfect straight line. Where interface shear stiffness of 0.5 MPa/m and 0.1 MPa/m were used the initial stiffness is too low (see Figure 4.6), to mobilise the full strength on the first portion of the curves. In the case of 0.1 MPa/m shear stiffness, the peak strength is omitted completely as a result of this. If used in modelling, although correct strength parameters may be defined, use of incorrect interface stiffness prevents the peak strength from being mobilised and hence unrealistic large displacements may be predicted.

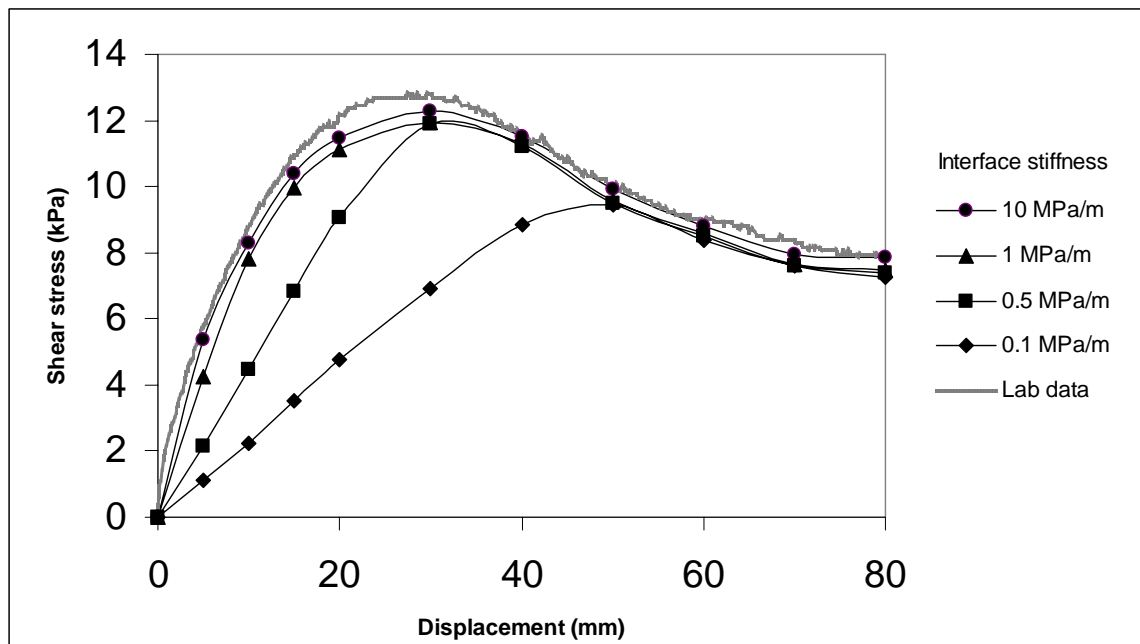


Figure 4.6 Influence of interface shear stiffness

This has can have a serious impact on commercial design, as the interface shear stiffness in numerical analysis is often assumed. Selecting an interface stiffness that is too low, often selected to reduce calculation difficulties and solution time, can result in underestimation of interface shear strength. Whilst initially this may be seen as conservative, if the interface shear strength at the waste barrier interface is underestimated, the stress transfer into the lining system will be reduced and resulting integrity issues may be unconservative.

4.4.3.2 Interface interpenetration.

Interface interpenetration is a numerical issue, where one side of the interface moves into the other side of the interface so that an overlap is created (see Figure 4.7), as discussed with respect to geosynthetic interfaces by Villard (1996). At worst, one side of the interface will move uncontrollably through the interface. However, even small interpenetration will cause errors in interface displacement calculations. This process is controlled by the normal stiffness of the interface. The normal stiffness should be increased to approximately 10 times the stiffness of the adjacent material (Itasca, 2002). If there is a significant difference between the material stiffness across the interface, the material with the lower stiffness should be considered. Whilst slight interpenetration will occur even with very high normal stiffnesses, it is greatly reduced by selection of the correct normal stiffness.

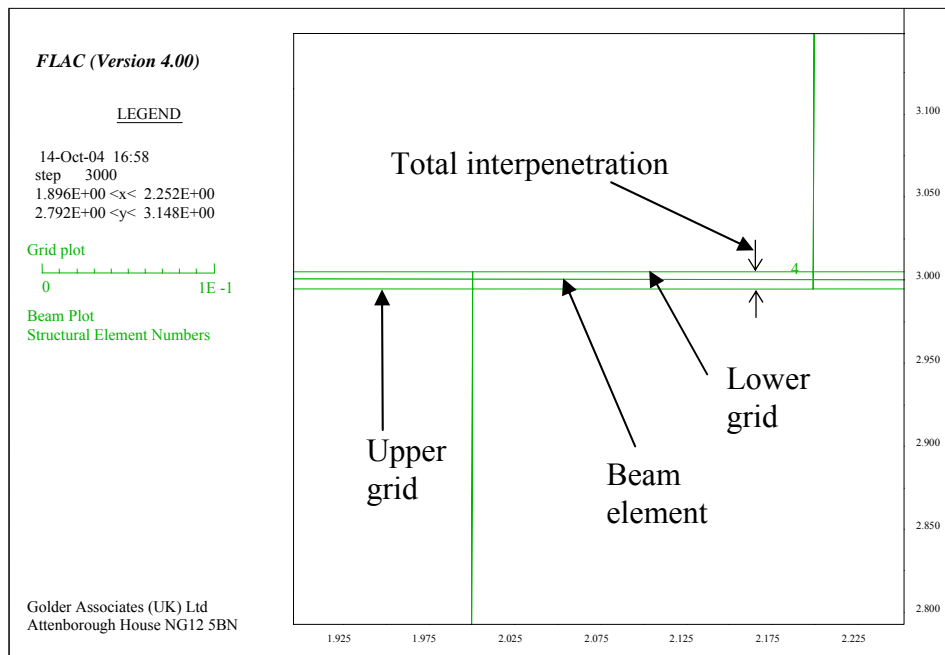


Figure 4.7 Interface interpenetration

4.4.4 Parameter acquisition

4.4.4.1 Waste properties

Waste properties are very difficult to measure, primarily due to the particle size, and hence the size of samples and testing equipment required. Kavazanjian (2006) provided a state of the art summary of waste properties including: Unit weight, K_0 , stiffness and strength. Dixon and Jones (2005) summarised waste parameters, identifying which parameters are required for different landfill design cases. It is considered acceptable practice, if by necessity, to use published data, and not site specific testing in design. As waste pre-treatment becomes more commonplace it is envisaged that the waste will become less heterogeneous in size, density and composition, and site specific strength testing may become more feasible.

4.4.4.2 Geosynthetic tensile properties

Tensile tests of geomembranes are typically carried out using a dumbbell specimen (ASTM D638 : 2003). For other geosynthetics wide width samples (BS EN ISO 10319:1996). The elastic modulus can be derived from the stress strain curve under axial tensile load. However, Giroud (1994) showed that the initial portion of the slope

is non-linear, and thus the user must select an initial, secant at a given strain level or tangential modulus. Typical practice is to select secant modulus at yield. However, this can underestimate the modulus at low strains. This is discussed further in Section 4.5.4.

4.4.4.3 Interface testing

Direct shear apparatus specifically designed for interface testing is typically used to derive shear strength data. A minimum of 3 tests should be carried out at varying normal stresses. The peak and large displacement shear stress values are plotted against normal stress to give the peak and large displacement shear strength envelopes, defined by a friction angle, δ , and a y-intercept, or apparent adhesion, α (kPa). When deriving strain dependent shear strength parameters, the shear strength envelopes are plotted at regular displacement intervals (e.g. every 5 mm), thus allowing strain dependent α and δ parameters to be derived. The term large displacement is adopted to avoid confusion where true residual values have not mobilised.

4.4.4.4 Soil strength and stiffness

Soil laboratory testing, typically using triaxial apparatus is used to derive the drained and undrained shear strength parameters of soils for use in numerical analyses. Acquisition of stiffness parameters is significantly more complex, hence, literature values are typically adopted.

Young's modulus and Poisson's ratio are linked to the bulk and shear moduli, G and K respectively, through the relationships shown in Equation 4.2 and Equation 4.3. Young's modulus can be estimated from the deviatoric stress and axial strain in triaxial tests. In drained triaxial compression tests the shear modulus can be estimated from the slope of the deviatoric stress, to distortional strain curve, although this requires measurement of the axial *and* radial strains. The relationship between the bulk and shear modulus, and hence the Poisson's ratio can be determined from the ratio between the deviatoric and volumetric strain increments.

$$G = \frac{E}{2(1+\nu)}$$

Equation 4.2

$$K = \frac{E}{3(1-2\nu)}$$

Equation 4.3

where:

E = Young's modulus

ν = Poisson's ratio

K = Bulk modulus

G = Shear modulus

In practice limited test data are typically available to determine the stiffness of soils, and literature values are normally selected. The stiffness is particularly important in assessing the deformation behaviour of compacted clay mineral lining systems using numerical modelling techniques. It is recommended that site specific drained triaxial testing is carried out on the materials to determine the input strength *and* stiffness parameters.

4.4.5 FISH programming language

The FISH programming language (standing for FLACish) allows the user to execute subroutines within the FLAC code. These can be used to:

- Aid construction of the model;
- Control material behaviour; and
- Acquire data from the model.

4.4.5.1 Linked list data structure

FLAC users can directly access the FLAC data structure using FISH. Each piece of data for model components (e.g. grid zone, structural element or interface) is stored in lists, and the user can traverse between different pieces of information for the model component using "pointers". Itasca (2002) provides ".fin" files which contain the pointers to each piece of data. If data is accessed using the linked list data structure, then the associated ".fin" file should be called first. The linked list data structure can be traversed using the respective pointers, starting from a "control block". A control

block is an initial list which acts as a directory for the other lists of the data. Direct access to the data structure allows tabulated data for any parameter, and control of the model parameters prior to and during model cycling.

4.5 Development of FLAC analysis toolbox

This section gives a synopsis of the advanced FLAC modelling techniques that have been developed to analyse landfill lining systems. Publication of an example demonstrating application of this model is given in Paper 3 and also in Fowmes *et al.* (2006b) and Fowmes *et al.* (2007a). Subsequent developments have been made to the modelling accuracy and developments, some of which have been demonstrated in Paper 5.

Particular areas to be considered during lining system analyses include:

- Strain softening interfaces;
- Multiple layered geosynthetics using beam elements;
- Staged construction;
- Strain dependent geosynthetic elements;
- Geosynthetic anchorage; and
- Pore fluid in waste and subgrade.

Whilst bespoke methods have been developed for studying interface interaction (e.g. Villard, 1996) and the behaviour of geosynthetic lining systems, the numerical modelling code FLAC allows the user to consider interface behaviour coupled to soil and waste behaviour. Whilst FLAC has been used for single strain softening interfaces, and to represent single beam elements (e.g. Jones and Dixon, 2005), at the time of project inception there was no published work available on the inclusion of multiple layered geosynthetics, or the use of strain softening interfaces with structural elements used to represent the geosynthetics.

Paper 3 shows an analysis of a steep sided landfill lining system using the FLAC numerical code. The modelling carried out and presented in Paper 3 involves the staged construction of a steep sided landfill lining system on a benched quarry

subgrade. A compacted clay liner is placed immediately on top of the rock side slope with a reinforced soil buttress in place to support the clay liner. Polystyrene facing units are located on the reinforced soil buttress to provide a flat surface for placement of a geomembrane followed by a geotextile protection layer.

4.5.1 Staged construction

In limit equilibrium modelling, staged construction is carried out to identify the most critical state in terms of stability. In numerical analyses, where deformation is considered, material deformations evolve and may be cumulative through the material construction and waste placement process. Representing staged construction in numerical analysis has the following benefits:

- The construction and support sequence can be represented with appropriate sequencing and representation of waste;
- The worst case stability case may be at an interim stage and deformations underestimated by single stage models that often “wish” the materials into place;
- Staged construction using an explicit code prevents the unrealistically high loadings which occur when large volumes of materials are placed within a single time step, generating large velocities and hence unrealistic deformations;
- The load and stress strain histories of materials can alter behaviour of subsequent lifts; and
- Cumulative strain through an incremental loading sequence can be recorded.

Temporary waste slopes will be formed during waste placement. During staged construction the designer should consider if a temporary waste slope is likely to influence the steep slope lining system. The FLAC code can be used to represent the waste filling sequence adjacent to the lining system, thus better representing the horizontal support and settlement behaviour. The model can be modified to account for increasing waste stiffness during filling as compression of the waste occurs. The FISH programming language can be used to aid model construction, particularly where repetitive lift based construction occurs.

4.5.2 Modelling multiple layer geosynthetic lining systems

Previous analyses using FLAC (Byrne, 1994; Connell, 2002; Jones and Dixon, 2005) did not consider multiple interfaces or geosynthetic elements, and hence the axial forces and strains in geosynthetic elements could not be calculated. Itasca (2002) recommends the use of beam elements to represent geosynthetics, with moment of inertia set to zero to represent a flexible sheet, which is unable to support any bending moment.

Beams must be defined outside of the continuum, and can interact with the continuum through interfaces or attachments. When two grid zones are brought into contact to form an interface, beam elements can be defined at the interface and zero thickness interfaces defined either side to allow interaction with the soil above and below.

As interactions between the beam elements is only defined through interfaces, and interface logic does not require the geometric positions to be absolutely representative of actual position (e.g. the thickness of 2 mm is very small compared to the model dimensions) beams can be defined with the same x,y co-ordinates and the sequence of interaction (i.e. which is the upper and lower beam in the sequence) between the upper grid, beam elements and lower grid defined through interfaces.

A structured numbering system is very important when defining the beam elements as this allows the beams to be easily identified during interface definition. A recommended method is to use node numbers 1-99 for the geomembrane, 101-199 for the geotextile *et cetera*.

4.5.3 Strain dependent multiple layered interfaces

Direct interface shear strength tests generate shear stress displacement curves. However, typical practice is to select the peak and large displacement shear stress values to derive shear strength envelopes. Concern surrounds the use of peak strength parameters in analyses and design. Jones and Dixon (2003) and Filz *et al.* (2001) report failures where post peak interface shear strength reductions were shown to contribute to failures. However, selection of residual shear strength parameters, or a selected post peak value, can be overly conservative, as it ignores the fact that the

peak shear strength may exist. To assess the strains in geosynthetic elements, the actual shear strength transfer at given displacements should be used, and not a simplified value based on peak or residual conditions. As the system evolves with loading, the interface shear strength will change and the transferred stress will alter.

Following back analysis of a landfill failure, Filz *et al.* (2001) stated that strain softening interfaces must be considered in all designs. Whilst guidance was proposed for using factored values in limit state design, numerical analysis was required to be able to reproduce the failure. Byrne (1994) states that numerical techniques are capable of modelling the kinematic behaviour of strain softening interfaces.

A strain softening interface code was produced by Itasca (2002), however, this is unsuitable for modelling interfaces involving beam elements as the code is unable to recognise the velocity of the structural node and as such is unable to interpolate the velocity of the structural beam elements. The strain softening interface code has been re-coded (Fowmes, 2005b) and adapted to be able to recognise what interface is present and then, if a beam element is involved, the code selects the nodal velocity, in order to calculate the relative shear displacement. A new code has been written, SSint_beam13.fis (Fowmes, 2007b), which can be used for all types of interfaces; grid – grid, grid – beam and beam – beam. The code uses “if” functions to determine which type of interface is present and then derives the node/grid velocities accordingly. As the new code is more complex it increases solution time. For simple grid – grid interfaces it is recommended that the user applies the original Itasca (2002) SSint.fis code.

A schematic of the displacement dependent interface code is given in Figure 4.8. The strain softening interface code stores a relative interface shear displacement, in a spare FISH extension, for each interface number. As an individual interface relative shear displacement data point relates to each node number, the number of interfaces controls the number of sections that can individually strain soften. Hence if one interface is defined for the whole slope, the interface properties can change with movement, however, the properties will remain uniform across the entire interface length. If an individual interface is defined relative to each zone or structural element on one side of the interface, then each of these can strain soften/harden independently,

and a profile of interface shear strength parameters will be produced over the slope length.

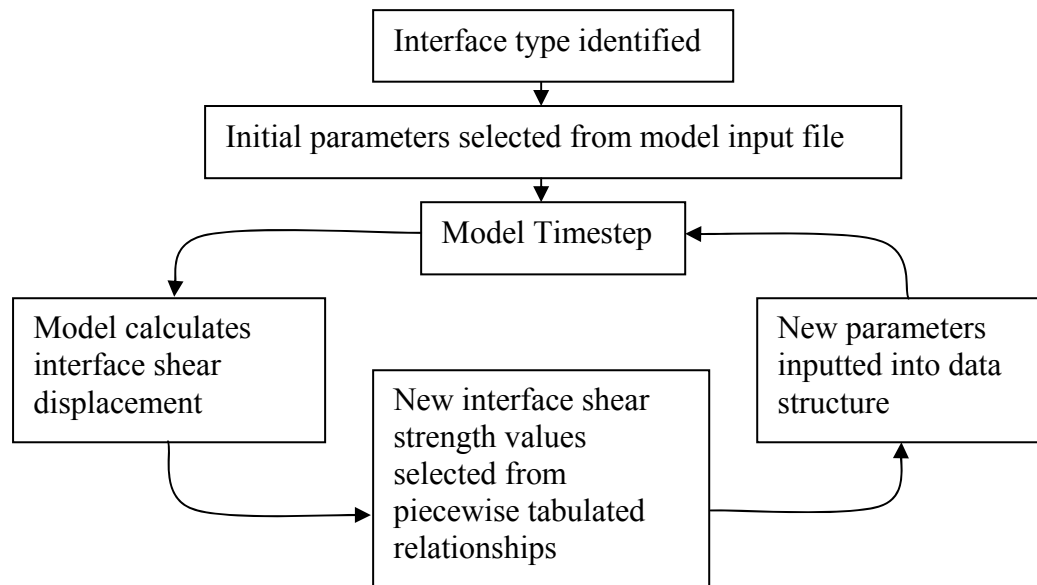


Figure 4.8 SSint.fis FISH code cycle

The strain softening interface code has been written to generate cumulative displacements. The relative shear velocity at each timestep is summed irrespective of sign, hence if movement occurs, and the interface is sheared back to the starting position, the calculated relative shear displacement is the total distance the interface has moved; not zero. Whilst in landfill engineering it is considered unlikely that large cyclic movements occur, the designer must be mindful of the movements of the waste during the solution of the problem, as during the time stepping process, the waste may ‘rebound’ as a static solution is achieved. If this occurs then the relative shear displacement may overestimate the actual displacement, and inappropriate interface properties defined. Use of volumetric strain controlled waste properties can result in waste ‘rebound’ caused by strain incompatibility and this is not representative of an actual physical process, which may result in an overestimation of relative shear displacement.

The strain dependent interface code developed in this project has been applied in further research to assess the affect of interface shear strength variability. Multiple shear strength - displacement profiles have been applied through multiple realisations using the FLAC code, in order to demonstrate the significant effects induced by the variability on displacements and component strains (Sia, 2007).

4.5.4 Axial response of geosynthetics

Calculation of the axial response of geosynthetics can be challenging, as manufacturers often quote values without representation of the test methods that have been used, or the correction factors applied. Geomembrane axial stiffness corrected from test data should account for the following:

- Thinning of the sample with strain (Merry and Bray, 1996);
- Necking of the sample with strain (Giroud, 2004);
- Change in Poisson's ratio, ν , with large strains (Giroud, 2004);

Often, no axial data is presented, as it is not considered necessary by the manufacturer when demonstrating the performance of their materials. From the results of conformance testing to ASTM D638 (2003), which are commonly available in design practice, the yield stress can be corrected for reduced cross sectional area using the equation:

$$\sigma_i = \frac{F_t}{t \cdot w \cdot (1 - \nu \cdot \epsilon_{nt})^2} \quad \text{Equation 4.4}$$

where:

σ_i = Yield stress

ν = Poisson's ratio

F_t = Peak force

t = initial thickness at zero strain

w = initial width at zero strain

ϵ_{nt} = Natural Strain, given by:

$$\epsilon_{nt} = \ln(1 + \epsilon_i) \quad \text{Equation 4.5}$$

where:

ϵ_i = engineering strain = elongation/original length.

To account for Poisson's ratio changes at large strain the following correction is applied:

$$\nu = \frac{1}{\varepsilon_{nt}} \left(1 - \sqrt{\frac{1 + \varepsilon_{nt}(1 - 2\nu_0)}{1 + \varepsilon_{nt}}} \right) \quad \text{Equation 4.6}$$

where:

ν_0 = Poisson's ratio at zero strain

The secant modulus can then be calculated from the peak force, F_t , value at a given strain. In addition, for numerical analysis, where plane strain conditions are adopted, the axial stiffness should be corrected to account for the two dimensional conditions using the following equation.

$$E_{tensile.sec(PS)} = \frac{E_{tensile.sec}}{1 - \nu^2} \quad \text{Equation 4.7}$$

where:

$E_{tensile.sec}$ = Tensile secant modulus.

$E_{tensile.sec(PS)}$ = Tensile secant modulus corrected for plane strain conditions.

Villard *et al.* (1999) identified that different stiffness values should be applied to geosynthetic elements in tension and compression to allow for the formation of folds and the differing axial response under compressive loading to those typically measured in tensile tests. A code has been developed in FLAC to allow a different value of Young's modulus to be applied in tension and compression. The number of unique beam properties that are defined controls the number of sections to which axial beam properties can vary independently. The beam property numbers are defined by the code user, and an individual beam property can be defined for each beam segment, thus allowing each segment to be assigned an individual property dependent on the axial strain (i.e. compression or tension) at the start of the timestep.

Giroud (1994) has identified that the initial portion of the stress - strain curve representing the axial response is highly non-linear, as the secant modulus decreases from the initial modulus. Giroud (1994) showed that the 2% secant modulus of a HDPE is over 3.5 times greater than a secant modulus at yield (the value typically

adopted in analyses), see Figure 4.9. One of the main reasons cited for use of the secant modulus at yield is the stress strain curves are often presented up to values in excess of 100% strain; hence the initial portion of the curve appears linear. Designs should specify that the area of interest is up to the yield point, as strains beyond yield are usually considered to be at failure. In Paper 5 a factored 2% modulus value was adopted. However, this required an appreciation of the strains that would actually occur, to identify the range over which to apply the secant value.

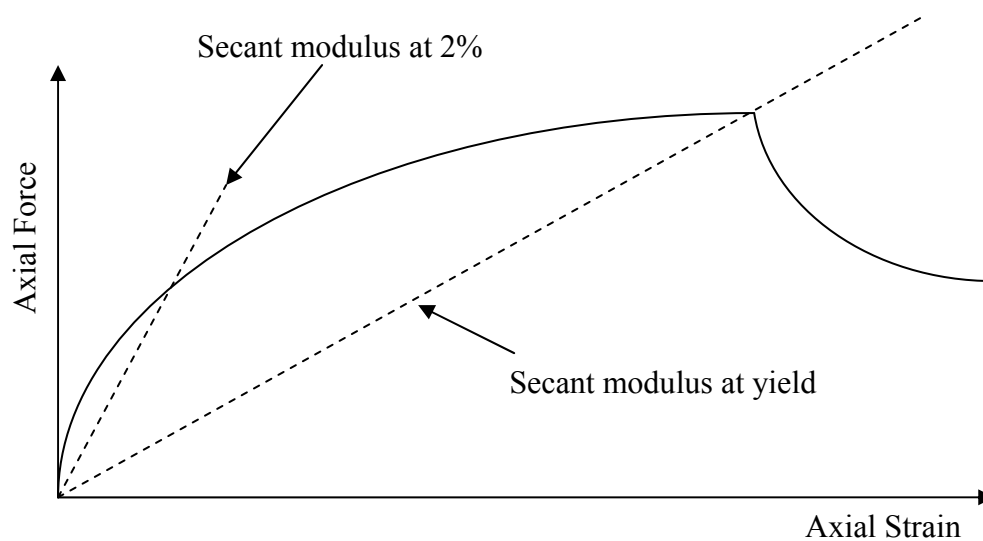


Figure 4.9 Schematic non-linear geomembrane axial response

Further developments since Paper 5 was written allow piecewise (see Figure 4.10) stress - strain curves to be included so that the Young's Modulus of the geosynthetic element may be varied with elastic strain. Two piecewise curves are defined using this code: the first representing tangential tensile modulus against axial strain and the second tangential compressive modulus against axial strain. As with the code used in the modelling for Paper 5, each beam element should be assigned an individual material number to allow independent strain dependent modulus values to be assigned. It is recommended that the same zero strain modulus be set on the compressive and tensile curves to avoid strain incompatibility in the initial calculation steps.

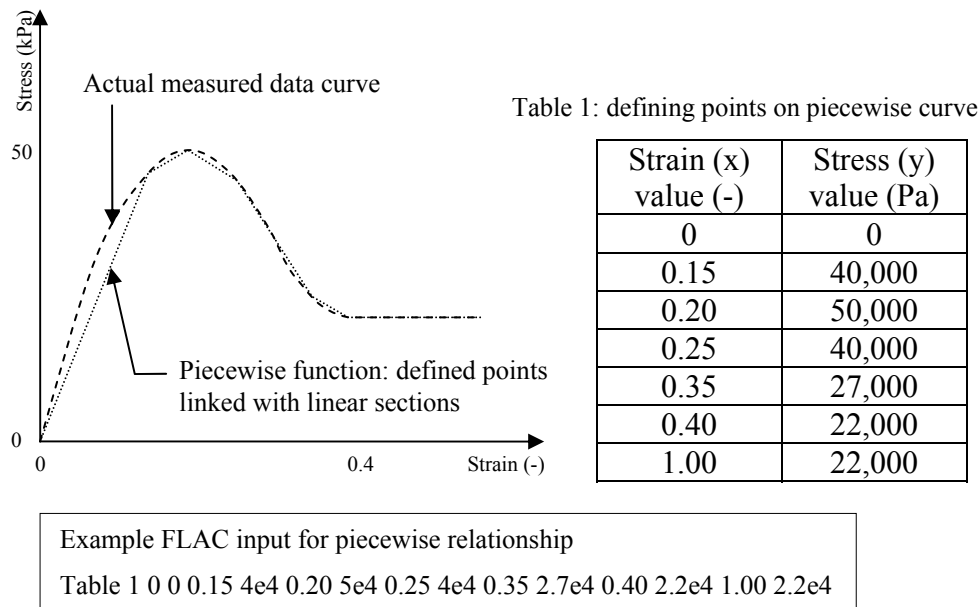


Figure 4.10 Schematic definition of a “Piecewise” stress-strain relationship

4.5.5 Influence of waste stiffness

Waste stiffness controls two important aspects of the numerical model behaviour:

- The magnitude of primary settlement; and
- The horizontal support to the lining system from the waste mass.

The influence of stiffness on the horizontal support increases as the slope angle of the waste barrier interface increases. Figure 4.11 shows a comparison from a parametric analysis of the influence of waste stiffness on waste deformations for 55° and 75° degree steep sided landfill lining systems. The influence of waste stiffness is clearly much greater on the steeper side slope, and hence the stiffness input parameters require greater consideration. If parametric studies show waste stiffness is a controlling factor, a depth dependent stiffness profile can be defined to better represent known behaviour (see section 4.5.6).

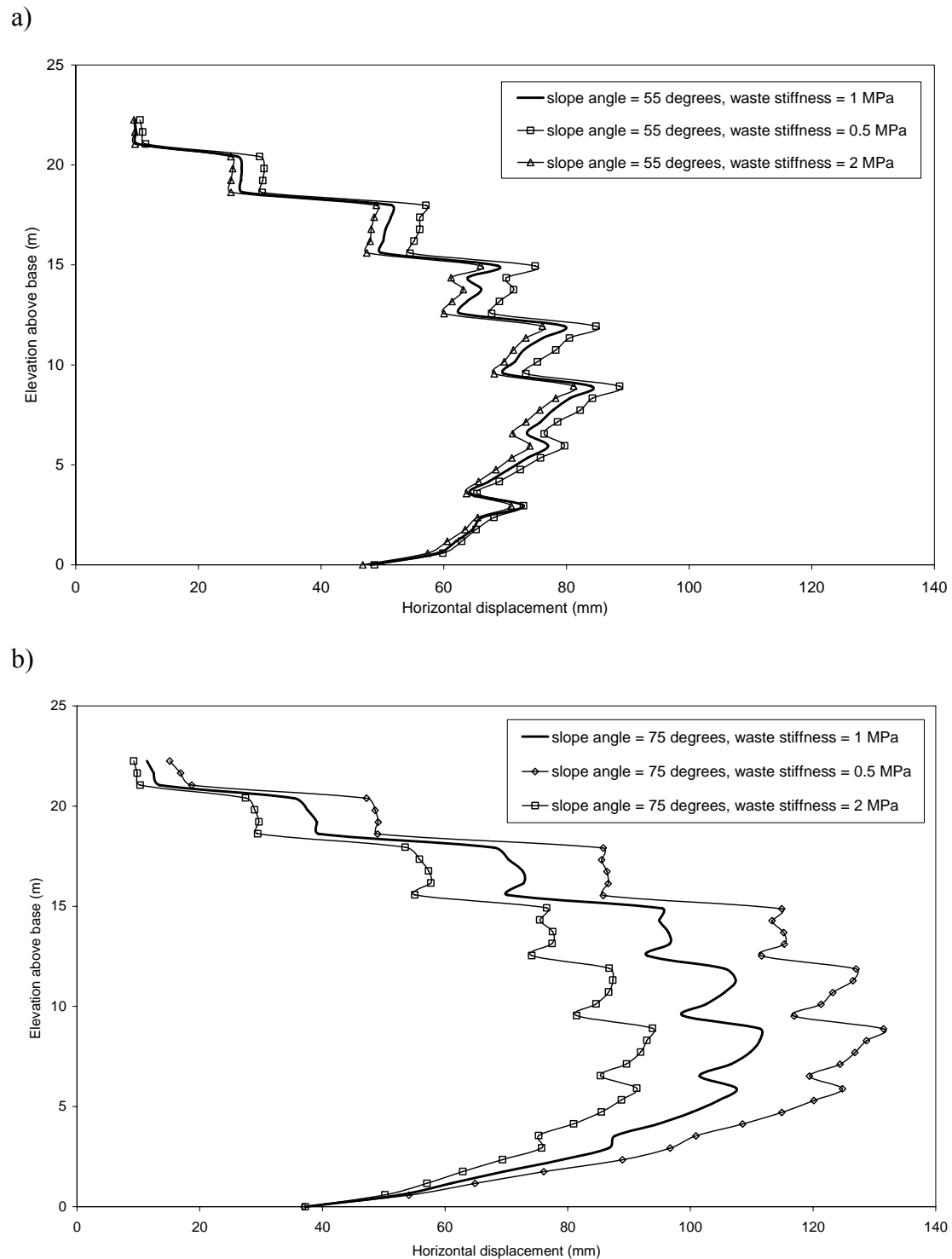


Figure 4.11 Influence of waste stiffness on deformations in a sided landfill lining system for (a) a 55° and (b) a 75° steep sided landfill lining system.

Waste settlements of 15 – 20 % are suggested by Watts and Charles (1990) and in Waste Management Paper 26B (1995), whilst 25% is suggested by Oweis (2006), however these are primarily focused on post filling settlement and allowable overfilling. To represent such settlement magnitudes under purely compression

loading alone, using simple linear elastic models, low stiffness parameters must be selected. As waste stiffness controls both the horizontal support and the generated primary settlement, the objectives of a numerical model must be clearly defined to demonstrate either the horizontal support, or to use a pseudo-stiffness value to model a representative settlement value. Jones and Dixon (2005) adopt a waste stiffness of $E = 0.5\text{MPa}$, $\nu = 0.3$ to represent a waste settlement in the order of 20%. This study was intended to model the influence of degradation settlement on the lining system, however, the mechanism causing the waste settlement in the numerical models was compression loading from the waste self weight. Dixon *et al.* (2006) measured stiffness of *in situ* waste using a pressuremeter, with the measured shear stiffness increasing linearly with depth, where the shear modulus G (MPa) $\sim 0.25 * \text{depth (m)}$, which is clearly significantly higher than the values adopted by Jones and Dixon (2005). Dixon *et al.* (2006) measured the stiffness of *in situ* waste, therefore, not taking into account the initial compaction settlement immediately following placement.

Whilst low waste stiffness can be selected to represent large settlements, it does not necessarily represent the stiffness response to loading, which will be the controlling factor when assessing deformations. If deformations in the lining system and associated support system are of primary interest then modelling of the waste with realistic parameters is required. However, if waste downdrag induced tension in geosynthetics is of primary concern then a pseudo-stiffness to represent the waste settlement due to loading may be more appropriate. Whilst the use of pseudo-stiffness may generate the appropriate settlement levels, it can also affect the normal stress at lining system interfaces. It is suggested that to model steep sided lining system deformations realistic modelling parameters must be used to represent the stiffness of waste, for short term behaviour. However, to model lining system tensile behaviour, downdrag should be considered.

The primary focus of this research is to represent primary compression of the waste during the construction process. In Paper 3, stiffness modelling parameters were selected to represent waste settlements in the order of 20%, which included initial compression of the waste. More recent advances have allowed a depth and volumetric strain dependent stiffness to be specified, allowing large initial compression, whilst

better representing waste horizontal support to the lining system at depth (see section 4.5.6)

There is a tendency in current commercial modelling of municipal solid waste to try to use a low stiffness linear elastic model in all circumstances. An area of further research is to model the waste mass behaviour with realistic stiffness values during construction then to allow settlement of the waste mass to include direct modelling of the degradation behaviour. This would involve assessment of the relative magnitude and relative timeframes of compression due to overburden and self weight and compression due to the time dependent degradation processes. Volume reduction and representation of the stress behaviour of waste is highly complex and beyond the scope of this current investigation. Attention should be given to developing such models to have the correct physical properties and changes that occur with time.

4.5.6 Waste constitutive behaviour

Several authors (Zhang 2007, Machardo *et al.* 2002, Krasse and Dinkler 2005) have proposed complex models for the behaviour of municipal solid waste. Zhang (2007) considers a waste compression model which includes compressible and reinforcing elements within the waste, to represent volume changes and shear behaviour of waste subjected to triaxial loading. Whilst such models are important steps in the understanding and quantification of waste behaviour, they are not, at the current stage of development, commercially applicable, due to their complexity and the input data requirements. Therefore, there is a requirement for a simple numerical model which can alter material properties, whilst retaining simplicity of application.

In steep sided landfill lining systems the waste resistance to deformation of the lining system is given by the waste stiffness. Dixon *et al.* (2006) demonstrated that there is an increase in waste stiffness with depth and as such the resistance to deformations of the lining system increases with depth. Whilst initial waste stiffness can be very low, to apply low stiffness for the entire height of the waste mass can result in over conservative, and hence uneconomic, designs. Codes to allow depth dependent parameters have been written using FISH, which allow piecewise definition of [depth] – [input parameter] curves for the waste to increase in stiffness with depth.

Variation of FLAC parameters with depth can be carried out using geometric measurements between a given grid zone and the surface, however, the simplest and most readily applicable method is to define functions in terms of ' s_{yy} ', the vertical stress component. An alternative is to define variable parameters in terms of volumetric and shear strain, or a combination of the two. Whilst parameters for behaviour against strain are often harder to acquire due to limited element test data on waste, the use of volumetric strain rather than vertical stress does eliminate some calculation deficiencies, as discussed below.

Depth dependent waste FISH codes have been commercially applied, adopting waste stiffness increasing with depth as compaction under self weight of subsequent waste lifts occurs (as identified by Dixon *et al.*, 2006). Whilst this process is also complicated by the degradation of waste, it is widely accepted that waste stiffness increases with depth. Two codes have been written to represent this:

1. Waste stiffness vs. s_{yy} (vertical stress). This code refers to two piecewise tables which represent 1) s_{yy} vs. bulk modulus and 2) s_{yy} vs. shear modulus.
2. Waste stiffness vs. waste vsi (volumetric strain increment). This code refers to two piecewise tables which represent 1) waste vsi vs bulk modulus and 2) waste vsi vs. shear modulus.

The first code allows easier application of modelling parameters, however, a slight computational error occurs when applying this code due to strain incompatibility immediately following placement of an additional lift. When a new lift is added there is an immediate increase in s_{xx} , and, therefore, the bulk and shear modulus of the waste increases in the next timestep, this causes a strain incompatibility as the stiffness increases before compression of the waste layer has occurred. The result is a slight recovery of volumetric strain immediately following placement of a new waste layer, which is then followed by the subsequent compression. In practice the second code should be applied where possible.

4.5.7 Geosynthetic anchorage

Fowmes *et al.* (2007a) suggests three methods for representing anchorage in numerical models:

- Fixing the geosynthetics;

- Flexible fixing to allow for displacement of the anchor point; or
- Detailed model of the actual anchor trench.

Fixing of the geosynthetics is the simplest method and was adopted in Paper 3, including temporary anchoring during staged construction. In this case the geotextile was free allowing displacements, as adopted by Meißner and Abel (2000). Fowmes *et al.* (2006b) adopted a flexible fixing to allow deformation of the anchorage. Villard and Chareyre (2004) used discrete element modelling (DEM) to model full anchor behaviour and to reproduce failures. Such methods are highly complex and currently beyond the scope of most commercial applications, and, therefore, were not considered appropriate for this investigation.

4.5.8 Reinforced soil

Reinforced soil support of the geological barrier is a becoming increasingly common in steep sided landfill lining system designs (see Section 2.3). Cable elements in FLAC are used to represent planar reinforcements such as woven geotextiles and geogrids. Unlike beam elements, cable elements can be placed within a modelling grid, applying a force to the modelling grid at the nodes adjacent to the reinforcement. Axial properties of the reinforcement element are defined using limiting tensile strength and a modulus of elasticity, which represent mobilisation strains within the materials. Additionally, the interaction between the cable element and the modelling grid is defined as a shear bond strength and stiffness, which can be used to represent interface shear strength and stiffness. Mobilisation of the reinforcement tensile resistance requires both interface displacement and axial elongation.

4.5.9 Compacted clay liners

Drainage conditions in the mineral lining system can be represented through either the use of effective strength and stiffness parameters with applied pore fluid pressures, or through the definition of undrained strength and stiffness parameters. Alternatively pore fluid distributions can be established by imposing hydraulic boundary conditions and allowing FLAC to generate the pore fluid pressure distribution, based on the material properties. Flow can then be restricted (uncoupled analysis) or allowed to occur (coupled analysis) whilst a physical model solution is achieved. Coupled analysis of fluid migration during fill placement is beyond the scope of this analysis.

It is typical commercial practice in landfill design to use drained conditions during the entire construction sequence and then to reassess the model with undrained parameters to decide on the “worst case”. However, in practice the material properties will change over time, as the initial undrained conditions, tend towards long term drained conditions and the material stress history cannot be represented by a single set of parameters or drainage conditions. Modelling has been carried out where the material properties are changed during the construction process, however, a sudden change in material properties can cause incompatibility. A better approach has been to define a pore pressure distribution and boundary drainage conditions, which can be altered as the construction process is modelled. This is considered to be a primary area requiring future research.

4.5.10 Subgrade and waste fluid pressures

Application of pore fluid pressures in the subgrade and waste mass can be applied as described in Section 4.5.9. Whilst fluid pressure in the waste can be considered, the heterogeneous and layered nature of waste makes prediction of fluid flow particularly difficult, and preferential flow paths are likely to form through the waste. However, the formation of excess pore pressures is unlikely in the long time frames in which settlement occurs. Nevertheless, if clogging of drainage layers occurs, undrained conditions and increased pore fluid pressure may persist. Bioreactor type landfills are becoming more common and the behaviour of the waste mass may be influenced by the recirculation of fluids through the waste mass.

4.5.11 Data acquisition and management

A series of data acquisition codes were used to directly access data from the FLAC linked list data structure. Any information within the linked list data structure can be acquired and output in tabular format. The examples below list some of the data access FISH functions that have been commercially applied:

- Node x,y positions;
- Beam/Cable axial strain;
- Beam/Cable axial force;

- Axial force; and
- Interface relative shear displacement.

Output files from the FISH data acquisition functions are presented as a plain text, which may be imported into a spreadsheet for post processing. Where data is acquired for a number of files and or throughout the construction sequence, macros can be used to aid repetitive data management.

4.6 Design toolbox document

As part of the research, a guidance document on the analysis of landfill lining systems has been produced, entitled “Design Toolbox: A guide to Landfill stability and integrity assessment” (Fowmes, 2007b). This document contains guidance on how to assess a design, select analysis methodologies and implement analysis tools. The document contains guidance of the use of limit equilibrium software (based on the code Slope/W), a detailed description of the FLAC modelling codes developed during this research, and guidance on their application. Figure 4.12 details the design flowchart from the Design Toolbox. This identifies a systematic approach for carrying out stability and integrity analyses for steep sided landfill lining systems.

This document has been applied commercially at Golder Associates (UK) Ltd, to guide both junior and experienced engineers in application of both limit equilibrium and advanced numerical modelling tools.

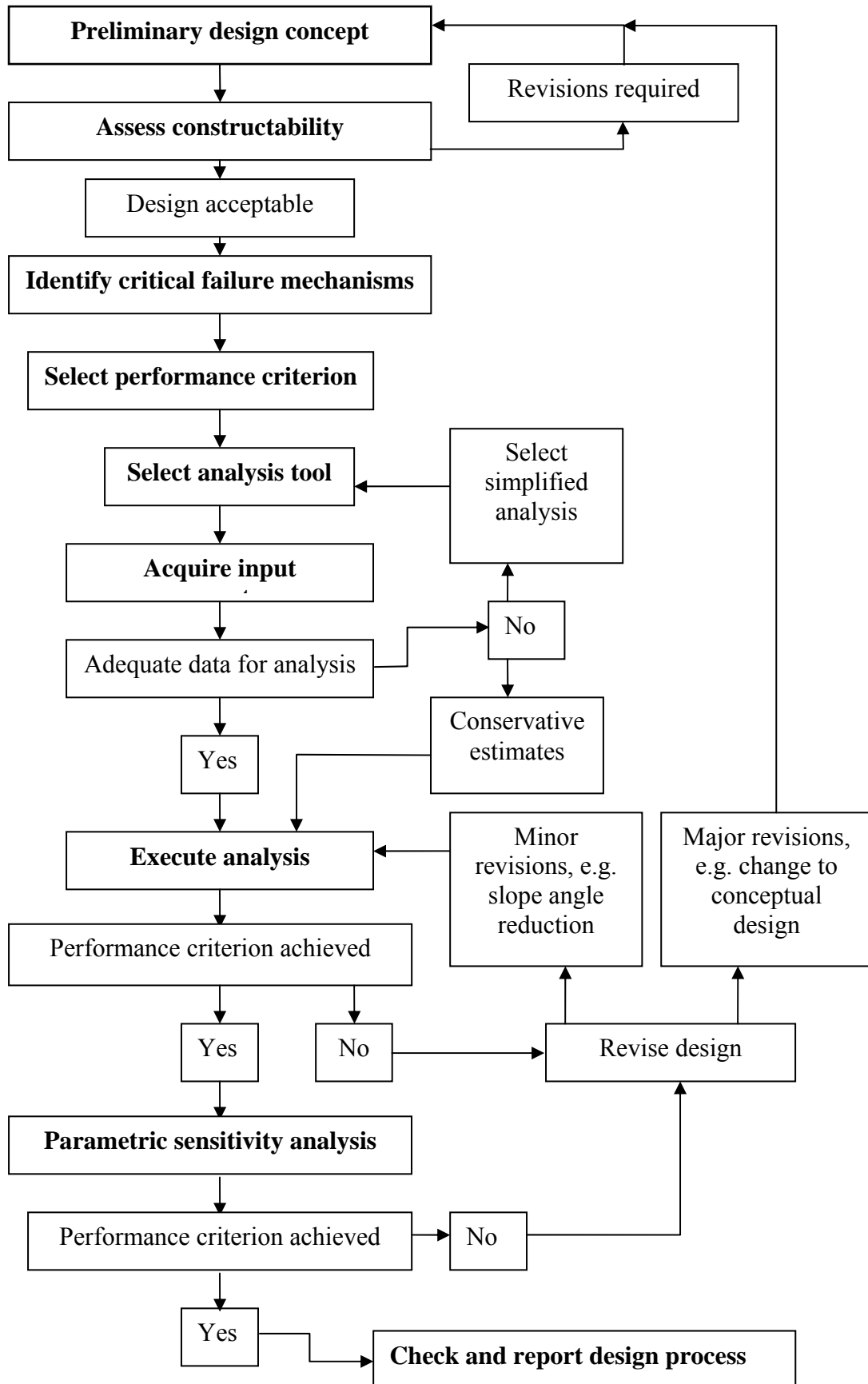


Figure 4.12 Design flowchart (after Fowmes, 2007b)

4.7 Validation of numerical analysis toolbox

Numerical modelling is increasingly being applied to landfill stability and integrity analysis for risk assessment and design. However, there is very limited validation of landfill lining system modelled performance against measured behaviour. It is not considered appropriate to develop a reliance on complex modelling software, if no validation of performance is given.

Jones and Dixon (2005) and Jones (1999) presented validation of shear box data using the single interface strain dependent code in FLAC. Similar shear box tests have also been used containing multiple beam elements (see section 4.4.3.1), to ensure the interface displacement, strength and shear strength behaviour (i.e. strain softening) is represented correctly. This section, along with Section 4.8, highlights some of the work that has been carried out to validate the numerical modelling approach, using FLAC (as discussed in section 4.4), and some of the additional validation required in the future.

4.7.1 Comparisons with Villard *et al.* (1999)

Villard *et al.* (1999) reported the results from instrumentation of an unconfined landfill lining system at the Compagnie Generale des Eaux experimental site at Montreuil sur Barse. The lining system consisted of (from the bottom up) clay layer, placed up a 1:2 (V:H), 9m long (measured parallel to slope) side slope, a HDPE geomembrane, a non-woven geotextile, and a 0.3 m thick granular drainage layer. Four stages of the landfill construction were considered:

Stage I: Placement of 300 mm thick gravel drainage layer in six 1m lifts on the side slope.

Stage II: Removal of drainage material at the toe of the slope to induce instability.

Stage III: Placement of two additional 1m lifts of granular drainage material on the side slope.

Stage IV: Placement of waste.

Villard *et al.* (1999) also produced comparative numerical models using a Finite Element Method (FEM) that can take into account large displacements that occur at interfaces using a formulation that insures non-interpenetration and equilibrium on either side of the interface (Villard 1996). The modelling carried out by Villard showed a good agreement during stage I (see Figure 4.14) however, during stage II the agreement was less satisfactory. This is thought to be due to observed tilting of the fastening posts onto which the geosynthetics were attached in the experiment, thus influencing the measured displacement and geomembrane tension. This explains the drop in geomembrane tension at the head of the geosynthetics observed between lift 5 and 6 of stage I (see Figure 4.14).

FLAC modelling has been conducted and compared with the results from the field instrumentation data and the finite difference analysis carried out by Villard *et al.* (1999). The modelling grid used in this investigation is shown in Figure 4.13. The results, in terms of geosynthetic tension are presented in Figure 4.14 and Table 4.1. The FLAC model included two beam elements to represent the geomembrane and geotextile. As with Villard's FEM model, separate tensile and compressive moduli were defined for each to account for wrinkling at the base of the geosynthetics. The interfaces use a Mohr Coulomb failure criterion, with no strain softening, but a mobilisation displacement required before peak strength is applied. The FLAC modelling shows good correlation with the measured and theoretical results as shown in Figure 4.14. In terms of geomembrane tension the FLAC model predicts greater tensile force, and gives a closer correlation with measured behaviour than the Villard *et al.* (1999) FEM model up to the drop in tension following lift 5.

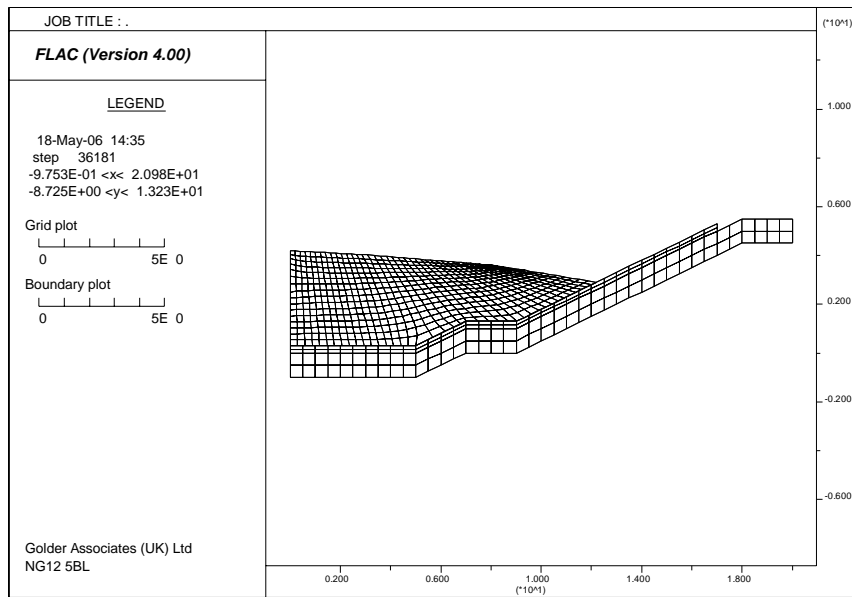


Figure 4.13 FLAC modelling grid for comparison with results from Villard *et al.* (1999)

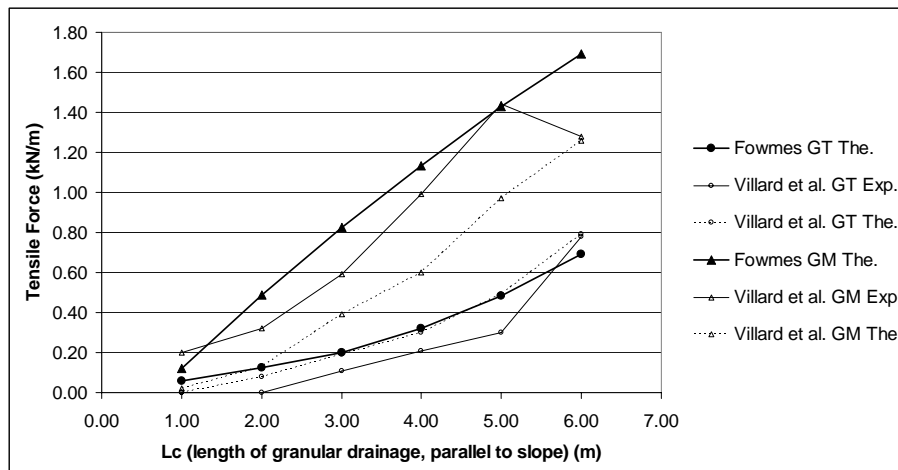


Figure 4.14 Comparison between FLAC model and Villard *et al.* (1999) for stage I (Lc 1 to 6m)

The drop in reported experimental geomembrane tension continues into stage II, which is believed to be due to failure of the fixing posts. As in stage I the FLAC model predicts very similar geotextile tension to the experimental model reported by Villard *et al.* (1999), and higher (1.96 compared to 1.40) geomembrane tension, although the trends indicated before the assumed instrument failure suggest that the higher value would be generated.

Table 4.1 Tensions at the head of geosynthetics in FLAC models compared to Villard *et al.* (1999)

	Geotextile tension (kN/m)			Geomembrane tension (kN/m)		
	Fowmes	Villard <i>et al.</i> (1999)		Fowmes	Villard <i>et al.</i> (1999)	
	Theoretical	Experimental	Theoretical	Theoretical	Experimental	Theoretical
Stage I	0.69	0.65	0.64	1.69	1.2	1.18
Stage II	3.82	2.68*	3.75	1.96	0.275*	1.40
Stage III	5.72	No data presented		2.43	No data presented	
Stage IV	5.80			2.65		

*Results believed to be affected by movement of fixing posts and therefore are not representative.

Stage III and Stage IV were also modelled using FLAC, although Villard did not present theoretical or experimental results beyond stage II due to the deviation between the predicted and measured values in stage I. Villard states that insufficient time for the system to reach equilibrium was allowed at the end of stage II thus influencing the results in stage III and IV, however, for completeness and comparison with future studies the results for this stage are included in Table 4.1.

This study has demonstrated that the multilayered interface model using FLAC gives a good correlation with measured results for an unconfined lining system. As data was not presented for the confined conditions (i.e. during and post waste placement) further validation of the model was required.

4.7.2 Back analysis of a steep wall geosynthetic integrity failure

In order to assess the performance of the FLAC model for confined conditions, back analysis of a failure at a Southeast Asia landfill was carried out. The modelling and results are discussed by Fowmes *et al.* (2006b). An integrity failure occurred in a smooth geomembrane just below a bench of a quarry landfill lining system (see Figure 4.16). The integrity failure involved the tensile failure of the geomembrane liner due to forces induced by waste loading, from weight and compression induced downdrag. The tensile failure observed on site occurred at a waste height of approximately 60 m above the waste reference level (see Figure 4.15) (Cowland, J., 2005, Pers. com.). The lining system comprised, from the bottom up, a geocomposite drainage layer, a 2mm smooth HDPE geomembrane, a non-woven protection geotextile, and a 500mm leachate drainage layer.

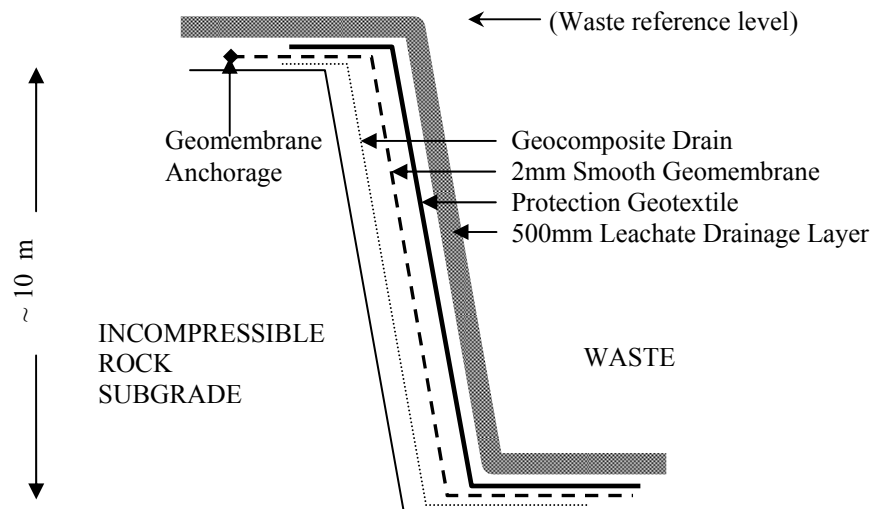


Figure 4.15 Schematic of the lining system at a Southeast Asian landfill (after Fowmes *et al.* 2006b)

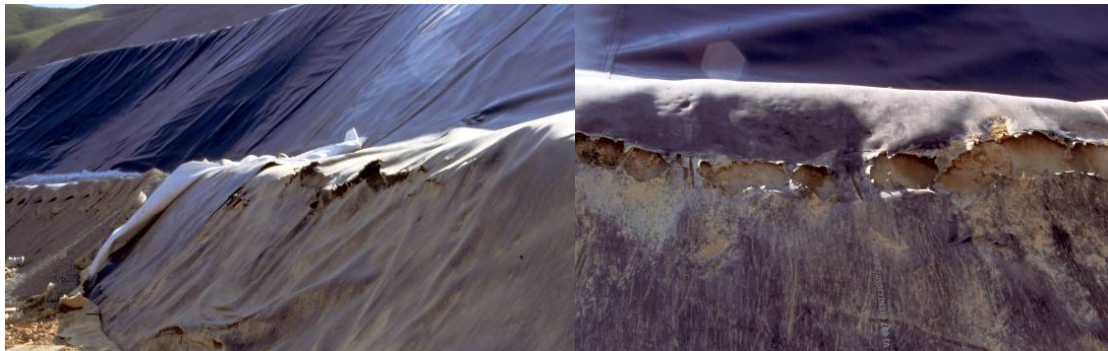


Figure 4.16 Landfill integrity failure (following exhumation)

Two models were used in the analysis; the first modelled a full height section of side slope to assess the waste and lining system behaviour on a benched quarry subgrade. The second model looked in more detail at a single section of the side slope in order to assess the behaviour of the lining system in more detail over a single bench height (Fowmes *et al.*, 2006b).

Individual beam elements were used to model each of the three geosynthetics, with strain dependent interfaces controlling interactions. Staged construction was considered with 2m waste lifts adjacent to the geosynthetics, followed by two 10m waste lifts to represent waste placement against subsequent benches, then loading increments of 140 kPa added to the upper waste surface to represent further 10 m

waste lifts. Waste was modelled with a volumetric hardening criterion, so that further compression, per unit stress increase, reduced as volumetric strain occurs.

The model was able to reproduce the failures in the geosynthetic elements as a function of tensile strain, with rapid increase in geomembrane stress following placement of waste at 60 m above the reference level (see Figure 4.15 and Table 4.2). The tensile strength of the geomembrane is approximately 28 kN/m, and the model predicts that this value is exceeded shortly following the waste loading increasing to the equivalent of 60 m above bench height. The rapid increase coincided with post peak strength reduction on the interface underlying the geomembrane as smooth geomembrane - geocomposite drainage interface has a peak interface friction angle of 13° and a large displacement friction angle of approximately 8°. Figure 4.17 shows the stress and strain distribution in the geomembrane following the application of 70 m of waste above the top of the bench.

Table 4.2 Axial strains and tensile forces in the geomembrane related to waste height. (after Fowmes *et al.* 2006b)

Waste height above bench.	Vertical pressure (kPa) at waste ref level	Maximum axial strain in geomembrane (%)	Maximum tensile stress in geomembrane (kN/m)	Location of max stress (m below top of bench)
0	0	0.14	0.42	3.2
10	140	0.17	0.51	1.2
20	280	0.20	0.60	1.2
30	420	0.20	0.59	1.2
40	560	0.37	1.32	2.4
50	700	0.40	1.43	4.8
60	840	8.37	25.1	3.6
70	980	14.7	44.9	1.2

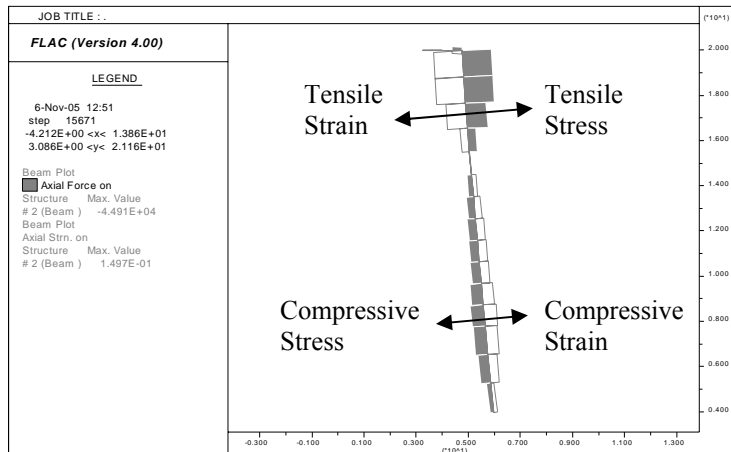


Figure 4.17 Geomembrane stress strain distribution (after Fowmes *et al.* 2006b)

The model was re-run with a mono-textured geomembrane (textured side down), with a textured geomembrane - geocomposite drainage interface having peak interface friction angle of 29° , and no integrity failure was predicted. This demonstrates the benefits of using mono-textured geomembrane on a steep side slope.

This study demonstrated that FLAC can be used to reproduce landfill lining system behaviour when confined and loaded by waste. Although no specific instrumentation was included on site to measure the tensile strength of the lining system, the timing, and hence loading conditions, associated with the lining tensile failure were identified due to increased leachate flow and this mechanism was reproduced using the FLAC code. Although giving confidence in the behaviour of the system, more rigorous validation of the FLAC predictions under confined conditions was required, with comparisons against a fully instrumented landfill lining system.

4.7.3 Field instrumentation

Although the field testing described by Villard *et al.* (1999) did not yield data for post waste placement, the information in stage I of the analysis showed good comparisons with both FEM and FLAC models, and highlighted the benefits of using field data for validation of numerical techniques.

Site instrumentation provides a means of validating numerical modelling with real world behaviour. In terms of landfill design it allows comparisons to be made

between the behaviour of multilayered lining systems subject to complex loading scenarios, and also allows the quantification of uncertain and assumed analysis input parameters, such as horizontal pressures applied by the waste and pore pressures generated in the lining system.

There are several key aspects of landfill lining system behaviour that can be measured through instrumentation. Table 4.3 shows the key areas of design that should be considered and potential instrumentation techniques identified during this project.

Table 4.3 Landfill instrumentation techniques

Physical characteristic to be monitored	Suggested Instrumentation
Horizontal and vertical waste pressure	Vibrating wire pressure cells
Deformations	Inclinometers and extensometers
Geosynthetic strains	Wire displacement gauge
	Fibre optic strain gauge
Pore pressures in mineral lining systems	Vibrating wire piezometers
Geosynthetic tension	Tensile load cells

4.7.3.1 Landfill “A” Instrumentation

Instrumentation of a landfill side slope was proposed at a landfill site in northern England, referred to as landfill “A”, and a design for the instrumentation was carried out as part of this project. The lining system consisted of a compacted clay lining system overlain by a geomembrane, which in turn was overlain by an innovative geotextile with integrated wick drains. The proposed instrumentation was to include wire displacement gauges at locations up the side slope on each of the geosynthetics and vibrating wire pressure cells at the base and up the side slope. Waste surveying and input data would be used to generate waste density and settlement behaviour. The project was a collaboration between Loughborough University, Golder Associates, *GEOFabrics* and Encia. However, the project was unable to proceed due to the landfill operator’s concerns surrounding the use of any data from the project and potential repercussions from the regulator (Environment Agency). Whilst it was stated that the project was to measure *in situ* performance to aid design and not to identify failures, approval to access the site was not granted.

4.7.3.2 Landfill “B” Instrumentation

As a continuation of this project instrumentation of a landfill lining system at landfill “B” has just begun at a landfill site in northern England. The lining system consisted of a compacted clay liner, against a hard rock quarry subgrade, supported by a reinforced soil buttress with polystyrene facing panels to provide a smooth flat face for placement of the geosynthetics lining system, similar to that assessed in Paper 3, (see Figure 4.18). The instrumentation involves the inclusion of inclinometer tubes behind the polystyrene facing material to measure deformations of the support system, vibrating wire pressure cells in the front face of the polystyrene to assess the normal force applied to the lining system from the waste mass and piezometers to measure pore pressures within the compacted clay lining system. Whilst this instrumentation allows assessment of waste support and associated deformations in the lining system, behaviour of the multilayered lining system performance will not be measured in the first stage of the instrumentation.

The instrumentation is initially planned during the construction sequence, and post construction to a total of two years, however, should movement greater than predicted tolerances be detected, the instrumentation monitoring will continue beyond this time. The steep sided lining system at Site B has been designed using the FLAC modelling code, and hence the results of the analysis can be used to validate the numerical model. Additionally, the results can be used to optimise both the design as a greater understanding of the waste support is gained.

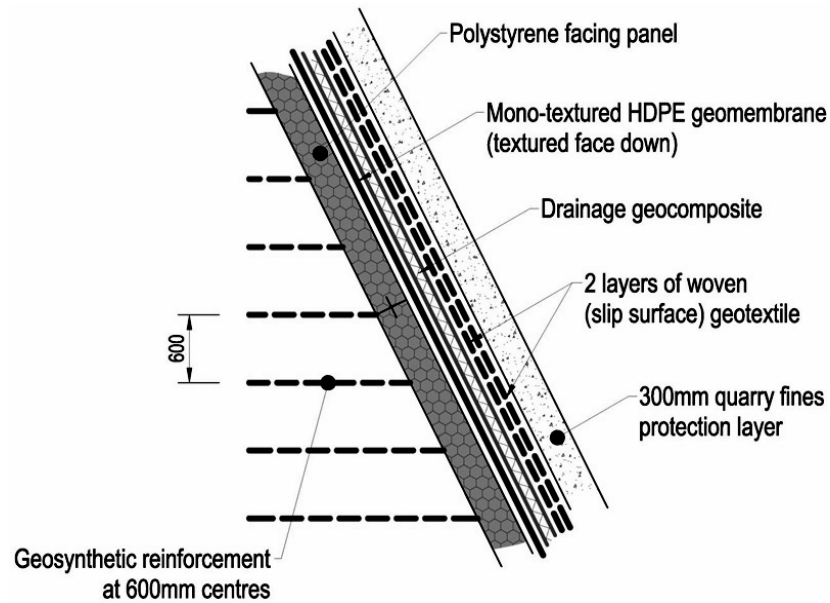


Figure 4.18 Steep sided lining system used at “Site B” landfill (after Golder Associates, 2006)

4.7.4 Fibre optic instrumentation

Whilst wire displacement gauges allow assessment of relative shear displacements between geosynthetic layers in the lining system, improvements in the measuring resolution would allow an assessment of strain in geosynthetic lining elements including the evolution of strain during and post construction. Fibre optic instrumentation has been considered as a potential method for assessment of strains in geosynthetic lining system elements, particularly geomembranes.

Fibre Bragg gratings allow precision measurement of strains over short gauge lengths, which can be of particular use in geosynthetic engineering. Fibre Bragg grating systems have been adopted in reinforcement geosynthetics. A patented system, Geodetect™, has been developed by TenCate (previously Polyfelt) where fibre optic cables are included as a fibre in the woven geotextile structure, allowing measurements of geotextile strain. A research project to include the application of FBG technology to strain measurement in multilayered geosynthetic landfill lining system is in preliminary stages as a collaboration between Loughborough University and Cranfield University. The plan for development of fibre optic instrumentation is as follows:

1. Develop methods for attaching the optic fibres;

2. Unconfined tensile testing of geosynthetics to assess fibre/geosynthetic interaction;
3. Confined tests to assess influence of surrounding soil/geosynthetics on fibre interaction and to develop protection systems;
4. Use of fibre optic instrumented geosynthetics in large scale 1G models; and
5. Use of fibre optic instrumentation in field scale trials.

Contributions during this project included an initial background research on the use of FBG fibre optics, and preliminary testing of FBG systems and fibre splicing techniques.

4.8 Validation of FLAC using a large scale laboratory model

As access to a landfill site for instrumentation could not be gained within the timeframe of this project, it was decided to carry out a comparison between the numerical analysis toolbox and a large scale laboratory model. This would allow downdrag forces, similar to those experienced by a steep sided landfill lining system, to be imposed on a geosynthetic lining system in a controlled manner. It also allowed the material to be carefully characterised and instrumented, and offers the opportunity for repeatability testing, which is not available in on site testing.

A large scale laboratory test chamber has been designed and constructed to represent a steep sided landfill lining system subject to downdrag forces from primary compression, and measured behaviour was then compared to predictive FLAC analyses of the same model. The details of this model are presented and discussed in Paper 5.

The research philosophy was to produce a geosynthetic multiple layer geosynthetic system, with a loading system and measured axial stress, displacement response. The design was controlled primarily by the attempt to reproduce a real world lining system, however, the decision to use a vertical face was taken to simplify the model and thus reduce the variables which may influence behaviour. This in turn allowed the numerical model to be more representative of the laboratory model behaviour.

The laboratory setting for the experiment allowed repeatability to be assessed, and testing and calibration of the measuring equipment in situ. In the field tests reported

by Villard *et al.* (1999), the measurement of tensile forces at the head of the geosynthetics was compromised by failure of the fixing posts, where as such problems could be avoided in controlled repeatable laboratory conditions.

4.8.1 Laboratory testing apparatus

Paper 5 describes the testing procedure, results and associated modelling. The laboratory testing chamber is discussed in detail in Paper 5 and is summarised in Figure 4.19 and Figure 4.20. An iterative experimental design procedure was applied to the development of the test chamber. Preliminary designs included an inclined side slope, however, this overcomplicated the model. Refinements were also made to the instrumentation. Prior to compression the chamber contained 1m^3 of rubber crumb, with an instrumented geosynthetic lining system to one side, comprising of a geomembrane overlain by a non-woven geotextile. A summary of laboratory tests carried out in the chamber is given in

Table 4.4. It not intended to directly represent behaviour of a landfill lining system; it was instead designed to represent the interaction of lining system components when exposed to downdrag forces, and hence generate post-peak shear strength interface displacements experienced in side slope landfill lining systems. It must be acknowledged that real world landfill lining systems generally have a drainage layer between the protection geotextile and the waste, potentially altering the stress transfer into the geomembrane.

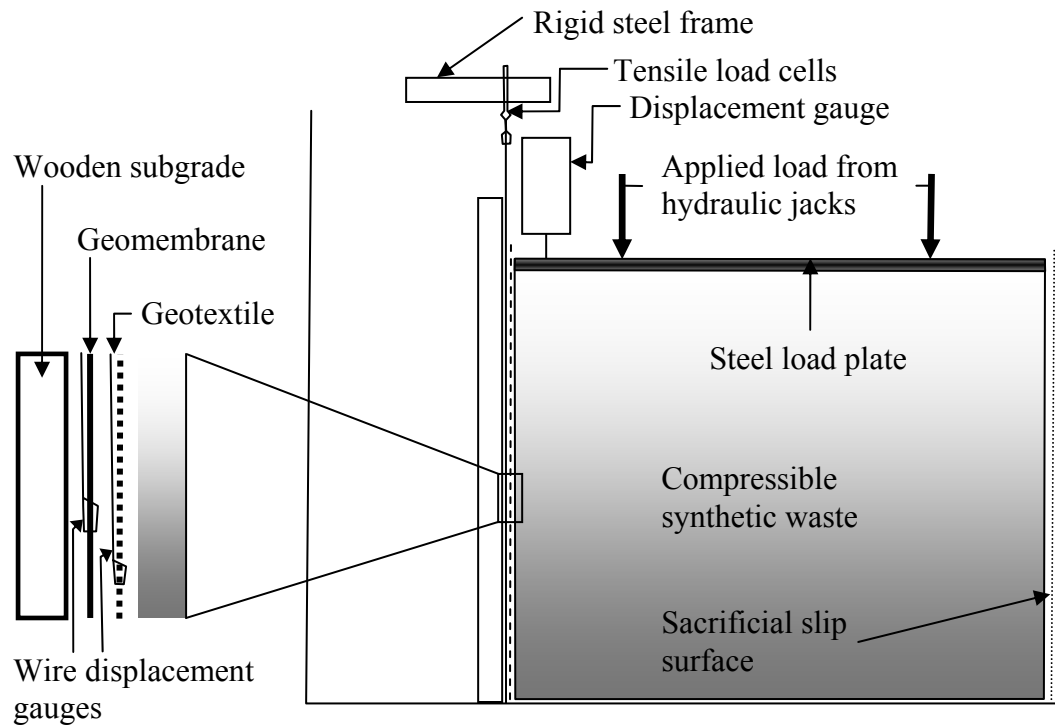


Figure 4.19 Schematic drawing of laboratory test chamber (reproduced from Paper 5)

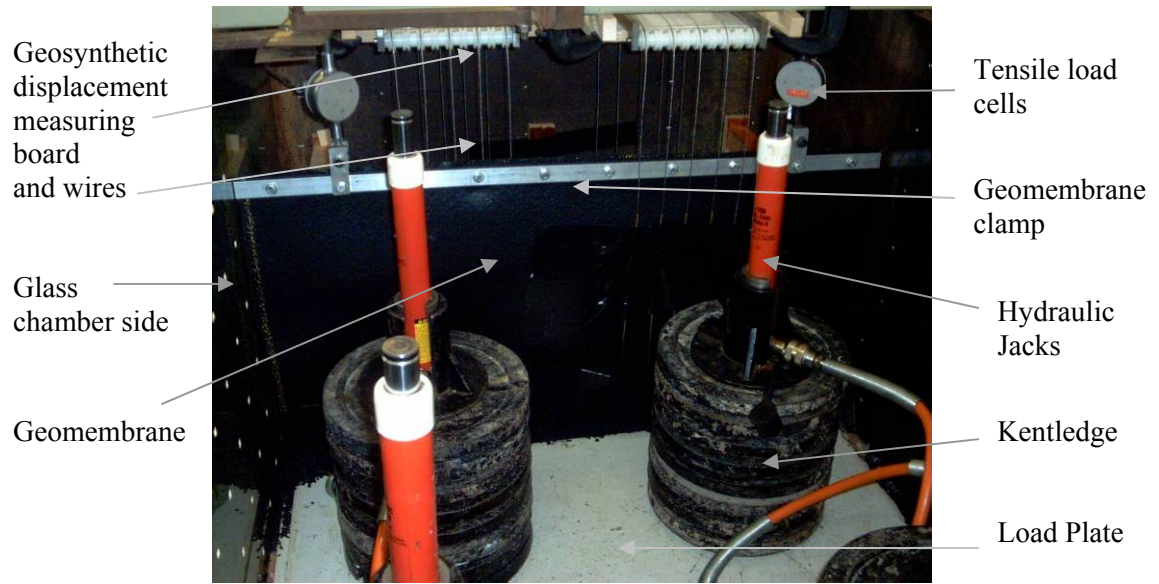


Figure 4.20 Photograph of laboratory test chamber

Table 4.4 1G model lining system laboratory testing programme

Test number	Geomembrane	Geotextile
Test 1	1 mm Smooth LLDPE	HPS7
Test 2	1 mm GSE Textured LLDPE	HPS3
Test 3	1 mm GSE mono textured HDPE (Textured side up)	HPS3
Test 4	1 mm GSE mono textured HDPE (smooth side up)	HPS3
Test 5	1 mm GSE Textured LLDPE	HPS3
Test 6	1 mm Solmax Textured LLDPE	HPS3
Test 7	1 mm Solmax Textured LLDPE (with fusion seam)	HPS3
Test 8	1 mm Solmax Textured LLDPE (with extruded seam)	HPS3
Test 9	1 mm Solmax Textured LLDPE	HPS3
Test 10	1 mm Solmax Textured LLDPE (with fusion seam)	HPS3

4.8.2 Instrumentation

4.8.2.1 Synthetic waste settlement

The vertical position of the load plate was measured using a linear MTS Temposonics position sensor. Preliminary tests were carried out with visible markers in the rubber crumb that could be monitored, through the glass window at the front of the test chamber, during compression to ensure that compression was uniform through the vertical profile of the sample. Following the preliminary tests, which showed greater compression in the upper section of the rubber crumb than near the base, the sheet steel and glass walls of the test chamber were lined with a 0.1 mm thick sacrificial plastic sheet to reduce boundary effects on the three sides without geosynthetics. This resulted in an observed compression in the lower 200 mm of the rubber crumb being equal to 91% of the settlement in the top 200 mm of the rubber crumb, which was considered to be satisfactory.

4.8.2.2 Geomembrane displacement

The displacement of both the geomembrane and geotextile were monitored at distances of 200, 400, 600, 800 and 1000 mm above the base of the geosynthetics (measurements are prior to deformation). Wires were attached to the geosynthetics at these points through small holes created in the geosynthetics. Whilst it is acknowledged that this would not be appropriate on site, it allowed the wires to be attached with only a small inclusion being formed. The wires were run to the surface then, via pulley wheels, over displacement measuring boards (see Figure 4.20) and each wire was tensioned using a 0.2 kg static weight. Along the length of the wires between attachment points with the geosynthetic and upper surface of the test, they

were isolated inside brass tubing to avoid interaction between the geosynthetic and wire gauges. As the geosynthetics displace, the wire attachments also move allowing the magnitude of the displacements to be measured.

4.8.2.3 Geomembrane tension

The geomembrane was anchored at the top using an aluminium flatbar clamp which was attached to a fixed steel frame via two tensile load cells (see Figure 4.20) allowing the axial force at the top of the geomembrane to be measured. The tensile load cells had a 12 kN limit and 1 N resolution.

4.8.3 Selection of synthetic waste

Selection of synthetic waste was based on a number of criteria:

- Compressibility $\sim 25\%$ under applied load;
- $K_o \sim 0.5$, therefore able to transfer stress horizontally onto the lining system; and
- Material needs to be reusable for repeated tests due to the large material requirements and limited storage space. No irreversible plastic deformation should occur.

The first materials assessed for use as synthetic waste were polystyrene sand mixtures that would allow vertical compression under load and still apply horizontal force to the lining system. Whilst the polystyrene particles selected could be small relative to the test chamber, there was potential for uneven mixtures of polystyrene and sand to be created. An additional drawback with the use of this material combination is that plastic deformations could occur in the polystyrene and hence, the polystyrene would have to be sieved out and removed following each test. Shredded tyres provided similar response to loading as MSW. However, preliminary tests showed that the large grain size and the potential for wire remaining in the tyre shreds could cause localised loading at the interface, and hence highly heterogeneous downdrag loads could have occurred, which were considered to be unacceptable. Hence, it was decided that rubber crumb would provide an acceptable synthetic waste material. Although the weight of the material ($\sim 5 \text{ kN/m}^3$) was lower than that of measured

waste of 6 to 16 kN/m³ (Dixon *et al.*, 2005), the lining system was vertical and the loading was imposed through applied horizontal force and not the self weight.

4.8.4 Preliminary testing

Table 4.5 and show details of the laboratory testing schedule for material characterisation.

Table 4.5 Preliminary laboratory tests for material characterisation

Purpose of test	Number of tests	Method Applied
Rubber crumb compression testing	6 tests in CBR mould 4 test in 0.125 m ³ test chamber	Strain controlled compression tests
	2 test in 1 m ³ test chamber	Stress controlled compression
Find K ₀ of synthetic waste	2 tests, two cells in each which are exchanged in the second test	Vibrating wire pressure cells in 1m ³ test chamber.
Interface shear tests	Minimum of 3 for each interface (total of 42 tests)	300 mm by 300 mm Direct shear apparatus
Rubber crumb shear strength	9 tests at 3 normal stresses	100 mm shear box

The shear strength parameters of the rubber crumb were measured in a 100 mm x 100 mm shear box. A series of 9 tests at 3 normal stresses were carried out. Tests were also carried out in the 305 x 305 mm direct shear apparatus used for interface testing, and comparable results were found. The mean shear strength of the rubber crumb under direct shear was a friction angle of 29.3° and cohesion of 3 kPa.

The compression behaviour of the rubber crumb was assessed at three scales. Preliminary tests were carried out in a CBR mould with strain controlled vertical load applied through a motorised compression rig (as used for triaxial testing). Repeated confined vertical compression tests were carried out on rubber crumb samples to assess the repeatability of the material response under loading. It was not permanently affected by the loading process, provided that the material was fully exhumed and replaced between tests thus removing the effects of particle rearrangement.

The second compression tests were carried out in the glass fronted test chamber measuring 500 mm x 500 mm x 500 mm (volume = 0.125m³). A motorised screw jack was used to apply a vertical load of 20 kN (80 kPa), via a load cell and a 25 mm thick wooden load plate, to the upper surface of the rubber crumb. The test showed very similar load compression curves to the preliminary CBR tests. The final

compression tests were conducted in the 1m³ test chamber, using a hydraulic loading system. The response again showed good correlation with the preliminary CBR tests.

Vibrating wire pressure cells were placed vertically and horizontally in the test chamber to determine the horizontal at rest pressure in the synthetic waste. The cells were swapped over for a repeat test and the results averaged to give a $K_0 = 0.55$.

Interface shear strength testing was carried out on each of the interfaces used in the laboratory model. The interfaces were categorised into 3 groups; synthetic waste – geotextile, geotextile – geomembrane, and geomembrane – wood subgrade. The interface shear strength tests were carried out in a direct shear apparatus, with a constant shearing area of 305 x 305 mm (using a 400 mm lower box). All tests were carried out in dry conditions, thus representative of the *in situ* conditions in the large scale laboratory test chamber. A shearing rate of 1mm/min was selected for the tests, allowing for stress dissipation in the rubber crumb. A slower shearing rate was not considered necessary due to the absence of fine grained soil. Ten minutes was allowed from application of normal stress to commencement of shearing to allow for compaction of the rubber crumb. The interface tests were carried out as performance tests, hence the materials above and below the interface were representative of the conditions in the large scale laboratory test chamber. The results of the interface shear strength tests are shown in Paper 5 Figures 4 to 7.

The tensile load cells were calibrated using applied weights up to 1.5 kN and using a tensile testing rig up to 10 kN. The loading system was applied to a loading reaction frame via a 100 kN vibrating wire load cell, thus calibrating the built-in pressure gauge. The vibrating wire pressure cells were tested in air, then placed horizontally in the rubber crumb and subjected to known vertical stress, whilst the vibrating wire readouts were taken. The tensile load cells were found to be to have a precision of $\pm 0.1 \%$ and the vibrating wire pressure cells had a precision of $\pm 1 \%$.

4.8.5 Numerical modelling of 1m³ laboratory model

The numerical modelling approach is considered in section 4 of Paper 5. Paper 5 Figure 8 shows the modelling grid, which consists of 3 zones of elements representing the wooden subgrade, the synthetic waste, and the test chamber side.

Beam elements are included in the model for the geomembrane and for the geotextile. The geomembrane has a fixed node at the top of the section and at the base to represent the boundary imposed by the base of the box. The geotextile is free to move at the top and has a fixed node to represent the boundary at the base. The beams were modelled with a 2% secant elastic modulus. A 2% secant modulus is often quoted by manufacturers for LLDPE geomembranes, and was considered more appropriate than a secant modulus at yield when compared to the expected deformations (see section 4.5.4). Giroud (1994) highlights the non-linear nature of the initial portion of the geomembrane tensile loading curve, and shows that the secant modulus at yield underestimates geomembrane modulus by a factor of approximately 3.5 times at 2%. The values of HDPE geomembrane secant modulus at yield were factored to account for this. Following this investigation it was decided to develop a non linear geosynthetic tensile response model, which is discussed in section 4.5.4.

A sign dependent code was applied so that the compressive modulus was one order of magnitude lower than the tensile modulus. This approach was adopted by Villard *et al.* (1999) to account for the fact that geosynthetics will fold and wrinkle instead of simply carrying compressive loading. Geosynthetic buckling under compressive loading is extremely complex (Villard, 1996), and beyond the scope of current landfill modelling practice. Figure 4.21 shows the influence of reducing the axial compressive modulus of the geosynthetic elements in the model. Where no reduction is applied, the model shows significantly less tension developed in the modelled geomembrane, 1.5 kPa compared to 3.9 kPa using one order of magnitude reduction. However, where a two orders of magnitude reduction was applied, only a smaller increase to 4.6 kN was observed.

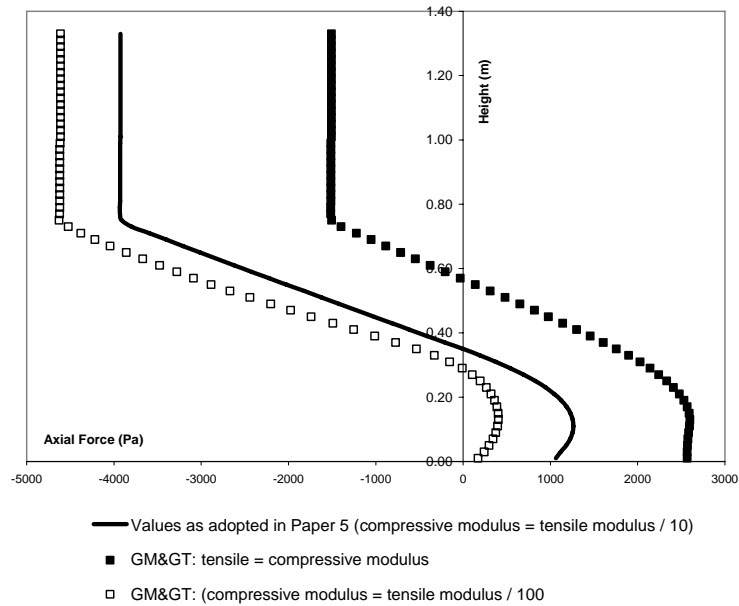


Figure 4.21 Axial force in the geomembrane as a function of compressive modulus reduction

A linear elastic model with a Mohr Coulomb failure criterion was adopted for the synthetic waste. Although volumetric hardening models were available, the Mohr Coulomb model is typically adopted in landfill engineering practice, hence validation using this model, would better serve industry requirements. This is discussed further in Paper 5 section 4.2.

The interface modelling adopted displacement dependent behaviour as described in 4.5.3, with three interfaces in the lining system. This is discussed in Paper 5 section 4.4. The inclusion of three interfaces allows the integrity of the geosynthetics, and stress transfer through the interfaces to be considered.

4.8.6 Laboratory and FLAC results

Comparisons between the numerical analysis and the laboratory measurements are presented in Paper 5 Section 5, and a summary is presented below.

For the Type G textured LLDPE geomembrane two laboratory tests were carried out, giving very similar responses (Paper 5 Figure 9), thus ensuring confidence in the repeatability of the investigation. The FLAC results show similar behaviour, with limited movement between the geotextile and the geomembrane, and large displacements between the geotextile and synthetic waste. The tensile force in the

geomembrane predicted by the FLAC model is a maximum of 3.93 kN/m, where the laboratory tests give values of 3.49 and 3.47 kN/m. This gives confidence in the assumptions regarding the compressive modulus adopted in the FLAC modelling.

The Type S textured LLDPE geomembrane was delivered directly from site and required cleaning prior to use in the test chamber. During the cleaning process of the first sample (in test T6), the texturing became damaged as abrasive cleaning techniques were adopted, thus reducing the friction angle between the geotextile and the geomembrane. Although this was noted prior to testing, it was decided to continue with the test to assess the effects of this and then to repeat the test. Paper 5 Figure 11 shows the increased movement between the geotextile and the geomembrane as a result of the damaged texturing in test T6. In test T9 the sample was cleaned using jetted water, which did not damage the texturing. The results from test T9 show good correlation with the FLAC numerical model (See Paper 5 Figure 11). Greater displacement is predicted by the FLAC model and observed in test T9 than between the geotextile and Type G geomembrane in tests T2 and T5. This is explained by comparing the shear box curves in Paper 5 Figures 4 and 5, as the peak friction angle between the type G geomembrane and the geotextile exceeds that of the synthetic waste – geotextile interface, whereas the same interface involving the type S geomembrane has lower peak strength than the synthetic waste – geotextile interface. These results not only act to validate the performance of the numerical model, they also highlight the influence of material damage on stability and the effect that installation damage and handling can have on stability and integrity. There is a significant difference in the measured geomembrane tension when using a Type G or a Type S LLDPE geomembrane (see paper 5, Figure 10). This is further evidence that interface shear strength is the primary controlling factor of geosynthetic integrity in landfills.

A mono-textured HDPE geomembrane was also tested, which has similar texturing to the Type G LLDPE geomembrane. When tested with the smooth side against the wooden subgrade, the interface displacements and predictions were similar to those in tests T2 and T5. The FLAC predictions also mirrored this result. The similarities in the shear strength relationships between these tests can be seen in Paper 5 Figures 4 and 6.

The mono-textured geomembrane was also tested with the textured side against the subgrade and the smooth side against the geotextile. The maximum displacement was between the geotextile and the geomembrane and reduced slippage occurred between the geotextile and the synthetic waste. Due to the large displacements of the geotextile, folding occurred that was beyond the scope of FLAC to model. This resulted in a less accurate prediction of the measured axial force in the geomembrane (see Paper 5 Figure 10).

Comparisons between the FLAC model and the same interface configuration when analysed using infinite slope limit equilibrium analysis show that for the Type G LLDPE geomembrane, at 50 kPa applied load, an axial tension of 16 kN/m develops, thus exceeding the tensile strength of the material. Both the measured and FLAC predictions show an axial tension in the order of 3 kN/m. This difference is due to the fact that in the lower portion of the interface the peak strength of the synthetic waste – geotextile interface has not been mobilised, and infinite slope analysis do not account for the reducing contact area as compression occurs.

A discrepancy is evident between the predicted and the measured relative shear displacement at the synthetic waste – geotextile interface. This is particularly evident in Paper 5 Figures 9a and 12a. This discrepancy is due to shearing in the synthetic waste, which is considered in the FLAC results, however, in the laboratory results, the relative shear displacements are based on an assumption of compression and thus shearing of the waste is omitted.

4.9 Reinforcement of mineral lining systems

4.9.1 Requirements for reinforcement

The numerical analysis carried out in Paper 3, in addition to commercial modelling of steep sided landfill lining systems, has identified that mineral barriers as part of steep sided landfill lining systems can be subjected to large strains. The magnitude of the strains is dependent on the clay strength and stiffness. The modelling in Paper 3 adopted relatively low shear modulus for the clay based on the internal database of parameters from FLAC. More appropriate parameters can be obtained from literature

sources including Cripps and Taylor (1986 and 1987). However, even with increased stiffness values, the mineral lining system was still susceptible to significant shear strains.

Planar soil reinforcements (e.g. geogrids, woven geotextiles) are typically applied in the construction of support buttresses used to maintain stability of artificially established mineral lining system. However, soil reinforcement elements are not continued through the compacted mineral liner as it is considered that continuous reinforcement would provide preferential flow paths for contaminants through the mineral barrier layer. This is considered as an area requiring further study, as designs could be made more stable if inclusions through the geological barrier are shown to be acceptable.

Randomly oriented fibre reinforcement was considered as a method for providing increased strength, and a more ductile stress strain response, whilst not facilitating increased fluid migration through the barrier. Random fibre reinforcement involved mixing of discrete fibres into the soil matrix prior to compaction.

Whilst the initial concept was to mix random fibre reinforcements into fine grained compacted clay lining systems, mixing proved difficult, and whilst studies (e.g. Miller and Rifai, 2004) show encouraging performance of fibre reinforced fine grained soil, commercially applicable mixing methodologies are yet to be found. Preliminary testing and the experience of other authors (e.g. Mayer and Ho, 1994) indicate that the only way to generate an even mix of fibres is to increase the moisture content of the soils beyond the liquid limit. Although feasible at laboratory scale, the waste and energy requirements and the fact that fibre reinforced fine grained soil would require drying prior to use would be unacceptable at site scale.

4.9.2 Randomly reinforced fibres theory

Randomly reinforced fibres apply force through tensile resistance to elongation. Michalowski and Zhao (1996) state that fibres only add strength through tension mobilised along the fibre and that reinforcement in the compressive regime is not considered to occur due to buckling and kinking. When a fibre is held by soil grains, it can act to resist movement of soil grains away from one another. Therefore, the fibres

become more effective as their orientation approaches the plane of the minimum principal stress. Figure 4.22 highlights a sample subject to triaxial compression and identifies the influence of fibres.

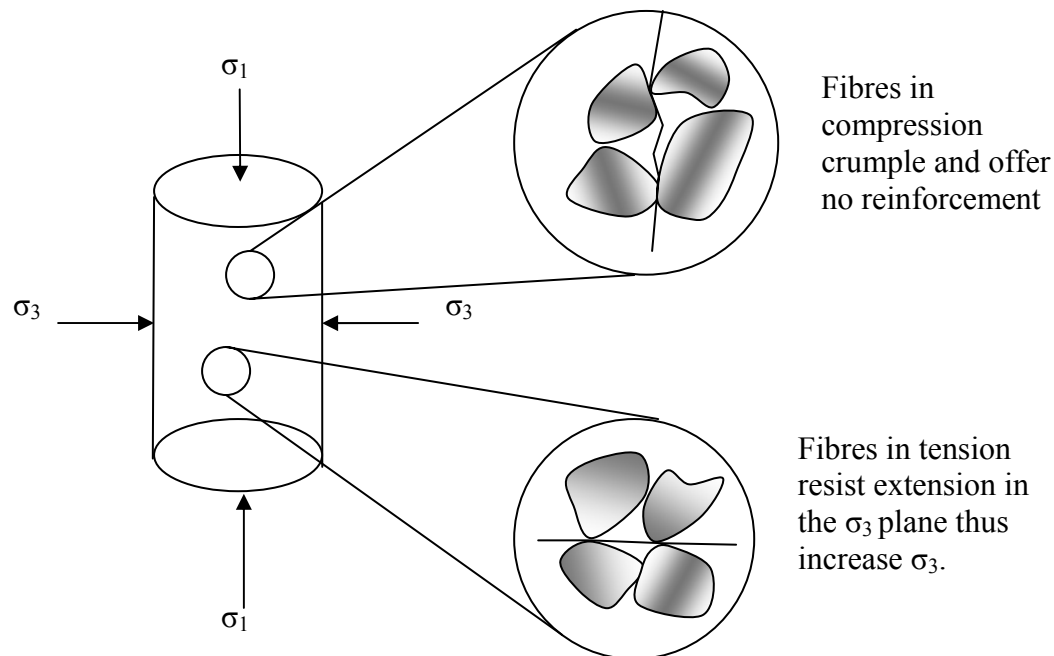


Figure 4.22 Fibre reinforcement theory

Michalowski and Cermak (2002) identify that strain hardening occurs in fibre reinforced soil. As compression occurs fibres will be preferentially aligned perpendicular to the principal stress axis, thus, the reinforcing effect will increase. However, Santoni and Webster. (2001) describe that strain hardening characteristics were seen at strains in excess of 25%, which is not considered acceptable if the hydraulic performance of the barrier is to be maintained. Whilst the fibre reinforcement will be randomly oriented at the time of mixing, compaction will cause preferential alignment.

One concern with the preferential alignment of the discrete fibres under strain is that preferential flow paths for fluid migration may form. Alignment of fibres in a landfill barrier is likely to be in a near horizontal plane due to the vertical compactive forces and overburden forces. Any preferential fluid migration would thus be allowed to pass through the barrier. Simple falling head permeability tests were carried out to assess the influence of fibre reinforcement, however, flow through the samples in the tests was parallel to the direction of compactive forces used in sample preparation. A

suggested area of further research would be to assess permeability, in a flexible walled permeameter, with samples acquired from vertical and horizontal orientations.

4.9.3 Bentonite enhanced soil reinforcement

Bentonite enhanced soil (BES) is used to form a mineral barrier, particularly where fine grained natural soils are not present in abundance. BES utilises the swelling properties of bentonite to fill the voids between sand particles (Jefferis, 1998). Mixing of fibres into the composite is easier than in fine grained plastic soil due to the dominant granular sand component and relatively dry mix. Mixing plant is already required on site hence the fibres could be added without the need for additional plant mobilisation.

Paper 4 details testing programmes and the results from compaction, triaxial and permeability tests that were carried out on randomly reinforced bentonite enhanced sand (RRBES) sample.

4.9.4 Results

Paper 4 Figure 2 shows that the optimum moisture content for both unreinforced and reinforced BES is very similar, however, the dry density is lower with reinforcement, believed to be due to the fibres acting to resist some of the compactive effort.

Significant strength increases were achieved when fibres were mixed and compacted at optimum moisture content (OMC). Increased fibre length, and, therefore, greater bond length, gave greater strength increases; Paper 4 Figure 3, indicating that fibre pullout was the controlling mechanism, as opposed to fibre tensile failure.

Samples were compacted with a moisture content of 2 x OMC to investigate the effect of increased moisture on fibre reinforcement. These showed much lower unreinforced strengths, however, reinforcement with 20 mm and 35 mm crimped fibres resulted in strength improvements, see Paper 4 Figure 5. Where samples were compacted at 2 x OMC samples reinforcement with 10 mm fibres produced lower strength parameters in triaxial tests than unreinforced samples, as the smooth fibres provided preferential slip planes along which sliding may occur.

Fibres must mobilise tensile stress to act as reinforcement, hence whilst the strength of the fibre reinforcements should not be ignored, the controlling factor in fine

grained soils with near optimum moisture content is likely to be the behaviour of the soils – fibre interface shear strength (i.e. bond resistance).

4.10 Construction considerations: Geomembrane seams

In landfill lining systems, sheets of geomembrane must be joined together by welded seams in order to provide a continuous barrier. In shallow slope lining systems the welded seams usually run from top to bottom of the slope. However, in steep sided lining systems, where construction occurs in lifts, it may be necessary to create horizontal geomembrane seams across the strike of the slope. In the UK, where the slope angle allows, fusion seams would generally be adopted, however, at near vertical slope angles it may be necessary to use extruded welding techniques.

There is very limited information on the influence of seams on interface behaviour available in the literature, primarily due to the difficulties in including seams in direct shear apparatus as the inclusion created by the seam would result in forced dilation of the overlying material around the inclusion, which would alter the shear stress at the interface, and the relative size of the inclusion would be large compared to the test area. The 1m³ laboratory test chamber was thus utilised to assess the affect of fusion and extrusion welded seams on interface displacement and tensile forces in geomembranes. Figure 4.23 shows a cross section through the seams.

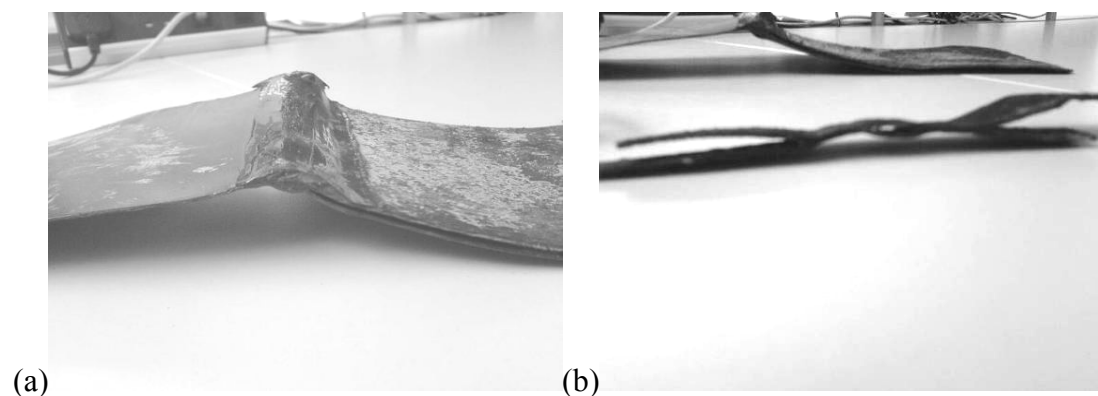


Figure 4.23 (a) a cross section through an extrusion welded seam and (b) a tab from fusion welded seam.

The test apparatus was described in 4.8.1. A single type of LLDPE geomembrane from the same roll was used for all of the tests reported in this section, and the properties are summarised in Table 4.6. Geomembranes were tested with:

- No seam;
- A horizontal extrusion welded seam; and
- A horizontal fusion welded seam.

Table 4.6 Geomembrane properties in seam influence investigation

Polymer Type	LLDPE
Resin Density	< 0.926 g/cc
Texturing	Double
Texturing Type	Blown film
Thickness	1 mm
Asperity Height (ave.)	0.25 mm
2% modulus	4×10^5 kPa
Break Strength	17.5 kN/m
Break Elongation	400 %

The axial force in the geomembranes as a function of applied vertical load is shown in Figure 4.24. For reference, the results are compared to a test with no seam present. Although this was not the only test carried out with no seam, it is believed to have the most representative properties and the same water jetted cleaning technique was used for this test and the geomembranes with seams.

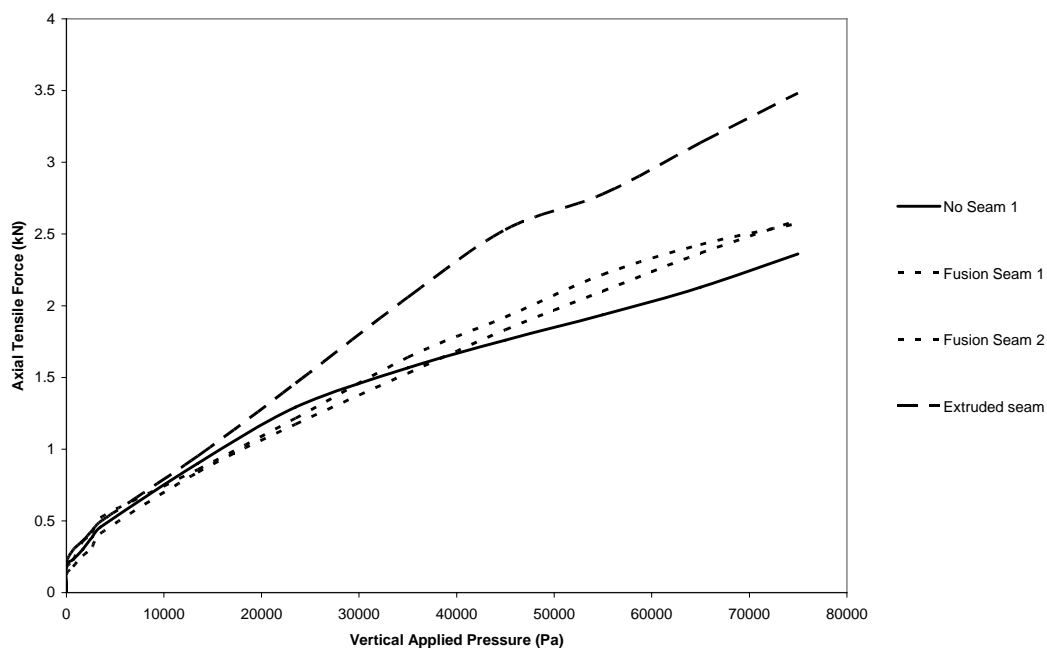


Figure 4.24 Tension in LLDPE geomembrane with and without welded seams

The fusion welded seams slightly increase the maximum developed axial force, from 2.36 kN/m with no seam, to 2.57 kN/m and 2.60 kN/m where the seams were present (increases of 9 and 10% respectively). The increase in geomembrane tension only

became apparent after 35 kN applied vertical load. This is believed to be a function of the increased horizontal stress causing the inclusion of the seam to have greater influence. When a horizontal extrusion welded seam was included in the test, a significant increase occurred in the measured axial force at the geomembrane anchorage. The maximum value recorded was 3.48 kN/m (an increase of 47 % compared to the sample without a welded seam). The influence of the extruded seam increases at applied vertical loads in excess of 15 kN. As the extrusion weld forms a greater physical inclusion, the effect is seen at lower stresses.

It was noted during exhumation of the rubber crumb that folding of the geotextile occurred around the location of both the extrusion and fusion welded seams, although this was more pronounced for the extruded seam than for the fusion welded seam. The inclusion of the welded seam is thought to provide a stress concentration, which induces the folding in the geotextile. Once fold formation has occurred, the fold also creates an inclusion that further increases the stress concentration.



Figure 4.25 Folding of the geotextile approximately 500 mm above the base of the sample in test with extrusion welded seam.



Figure 4.26 Folding of the geotextile approximately 500 mm above the base of the sample in test with fusion welded seam.

Interfaces are also influenced by the presence of the seams, particularly the extrusion welded seam which creates a rough inclusion inhibiting displacement at the geotextile – geomembrane interface, and inducing greater displacement in the geomembrane – subgrade interface.

5.0 Discussion

5.1 Interface shear strength variability

Bergado *et al.* (2006), Jones and Dixon (2003) and Koerner and Soong (2000) all identified interface sliding as a major cause of landfill failure. Jones and Dixon (1998), Criley and Saint John (1997), Koerner and Koerner (2001), Stoewahse *et al.* (2002), McCartney *et al.* (2004) showed that the variability associated with interface shear strength can be high. The purpose of this investigation was to assemble a comprehensive database of geosynthetic interface shear strength results which can be used as a reference tool, and to use the statistical data from the database in first order second moment reliability based limit equilibrium landfill stability analyses.

Typical practice for selection of parameters for interface shear strength was summarised by Koerner and Koerner (2007). This often simply consists of omitting any adhesion, and reducing the friction angle by one or two degrees, with little regard for the actual variability associated with the interface in question or the testing methods used. The database produced in this project allows users to quantify the variability associated with the interface in question, and select characteristic values accordingly.

The database shows large variability for literature results even when the interfaces involve geosynthetics on both sides of an interface. Geosynthetic – geosynthetic interfaces should give the minimum variability as they are manufactured in quality controlled processes. Whilst it was initially thought that variation in the global database is due to variability of basic material types, and hence site specific testing is required, inter-laboratory testing using materials from one source did, in some cases, yield equivalent variability, which is of concern as even with specified testing, significant variability may still exist. This demonstrates the importance of testing procedures, and well trained experienced testing personnel who understand the mechanisms associated with interface shearing and sensitivity to testing practice.

The interface shear testing database has provided a valuable commercial tool for selecting and verifying preliminary design values prior to site specific interface shear

strength testing, and then allowing greater confidence in selection of conservative estimates of the mean value affecting the limit state design, as required in BS EN 1997-1 : 2004 (Eurocode 7).

Probabilistic analysis has shown that use of the mean values from global datasets can produce very high probabilities of failure, due to the high variability and the sensitivity of the analyses to the selected parameters. Even with variability from repeatability testing, probability of failure is still significant, highlighting the importance of selecting appropriate, and conservative, design parameters. Probabilistic design methods allow the designer to gain an overall understanding of the combined sensitivity of the analysis to the combined variability of the input parameters and to undertake ‘what if’ assessments.

Schneider (1997) suggests the use of the subtraction half of a standard deviation to derive a characteristic value (see Equation 5.1), however, use of standard deviations from the global dataset would result in significantly more conservative designs than are currently produced. Whilst current design practice is not disputed as an acceptable methodology, failures do occur and designers should be aware of the variability associated with the parameters, and also the sensitivity of the calculation techniques to the chosen parameters. It should also be noted that designers should not always assume that a value lower than the mean is conservative (see section 4.1).

$$X_k = X_m - 0.5 \sigma_m$$

Equation 5.1

Where:

X_k = Characteristic value

X_m = Mean of test results

σ_m = Standard deviation of test results

Sia and Dixon (2007) and Criley and Saint John (1997), report interface shear strength from repeatability analyses, which would be representative of the variability of site specific testing carried out for commercial design. However, even when the variability from the Criley and Saint John (1997) was considered in Paper 2, the associated probabilities of failure were still higher than suggested limiting values of $P_f \leq 0.01\%$ (for serious), 0.05 (medium) and 0.3 (low risk events) (Koerner and Koerner,

2001). This indicates that either current design practice is unconservative, or that the target factors of safety are unreasonable. McCartney *et al.* (2004) also reported consistent findings, with the factor of safety corresponding to a $P_f=1$, showing that the factor of safety was in the range of 1.23 to 2.25.

Sia (2007) has further assessed the shear strength variability using statistical framework, and used the numerical models developed as part of this project along with the interface shear strength database to demonstrate the influence of shear strength variability. Sia (2007) has shown that combining the numerical analysis toolbox, with quantified variability, reliability analyses can be conducted, and whilst this is beyond the scope of current commercial design, it advances the understanding of the influence of variability.

For future design practice, particularly in view of the introduction of Eurocode 7, selection of appropriate parameters will require significant consideration. When interfaces are included in analyses involving factored input parameters, design values should be factored if target factors of safety of unity are to be applied. Application of partial factors for interface shear strength variability would be challenging as variability is different depending on which interfaces are selected. Generic partial factors could be defined for geosynthetic - geosynthetic, geosynthetic - fine soil, and geosynthetic - coarse soil interfaces. It is acknowledged that selection of interface shear strength parameters for design will require significant judgement on the part of the designer, who should derive characteristic values using site specific and consideration of the literature database.

The importance of interface shear strength testing behaviour was highlighted in the 1m³ laboratory model where the use of two different textured geomembranes was shown to significantly affect the behaviour of a three interface lining system. Additionally, cleaning of the textured geomembrane was shown to significantly increase interface relative shear displacement due to apparent reduction in peak strength (section 5.2 Paper 5), which demonstrated that even if designers have confidence in the interface design properties, the materials can be damaged easily, particularly during handling on site, thus lowering the strength properties. Installers of geomembranes and CQA engineers should be aware that the manner in which

geosynthetics are handled on site can have significant implications for stability and integrity. Careful placement of geosynthetic rolls using bars, and careful unrolling should ensure that the texturing is not damaged, however, the CQA engineer should visually inspect the geomembranes for damage to ensure that site specific testing is representative of on site conditions.

5.2 Numerical analysis toolbox

Prior to this investigation, analysis using FLAC had been limited to a single interface with strain dependent behaviour, Byrne (1994), Jones (1999), Connell (2002) and Chugh *et al.* (2007), or a single geosynthetic layer, with simple Mohr coulomb interface either side (Itasca, 2002). Whilst some specific modelling codes have been created, they have lacked the features to predict the behaviour of steep sided landfill lining systems under complex loading conditions.

The use of a commercially available and well established and tested numerical modelling package allows additional features of landfill lining systems, such as the geological barrier, and reinforcing elements in soil support structures to be incorporated. It also allows greater flexibility over the geometry that can be produced. This research has made significant progress since the work of Byrne (1994), Jones (1999), Connell (2002) and Chugh *et al.* (2007) due to the incorporation of geosynthetics into the model, therefore, the introduction of multiple layer parallel slip planes. This allows the integrity of the geosynthetic layers during and post construction to be assessed.

Non-linear strength laws have been applied to the interfaces, thus allowing the progressive failure mechanism identified by Filz *et al.* (2001) and Jones and Dixon (2003) to be assessed. The toolbox developed in this project allows analysis of progressive integrity failure in addition to the stability mechanisms considered by Filz *et al.* (2001) and Jones and Dixon (2003).

Giroud (1994) identified non-linear pre-yield response of geomembranes in tension, which is often overlooked in favour of a secant modulus at yield. This assumption can result in a significant overestimation of tensile strain at pre-yield stresses. This has been accounted for in the model allowing strain dependent axial properties for

geosynthetics to be defined. Guidance on obtaining input parameters strain dependent geosynthetics axial response is presented by Fowmes (2007b).

The model proposed by Villard (1996) allows analysis of multiple layer interfaces. This is considered by the author to be the most advance analysis model prior to this project, however, unlike FLAC, this is not a commercially available code and does not offer the wide range of modelling features which can be used to include the mineral lining system and engineered support systems. Another limitation in the application of the code by Villard *et al.* (1999) is that full strain dependent interfaces are not represented, although a mobilisation displacement for peak strength is included, implying that the code is capable of including fully strain dependent interfaces.

State of practice in commercial landfill design is still based on limit equilibrium analyses. Numerical techniques are only applied where problems have been identified or for complex and innovative designs. The numerical tools discussed in this report have been applied to commercial projects, however, as with any numerical analysis, application of the developed analysis techniques required a detailed understanding of the software, and also requires an understanding of the FISH programming language, as the codes developed in this project require modification to apply the code to the correct zone, interface or structural element. Application is time consuming, and whilst this project has developed tools capable of complex analysis, they cannot compete with the simplicity and ease of application of limit equilibrium techniques for conventional shallow slope designs.

Environment Agency concerns with regard to integrity tend to surround mineral liner performance, and whilst numerical modelling such as FLAC can be used to predict shear strains within the clay material, the stress strain response of the clay barrier is often highly simplified. Moreover, the allowable strains, in terms of increased permeability limits, in mineral lining systems are poorly understood, as are the relationships between soil plasticity and strain-permeability response. This area requires significant further research to allow designers to accurately quantify barrier performance.

Modelling of the non-linear stress strain behaviour of multilayered geosynthetic elements is generally beyond the requirement of stability risk assessments, however, it is considered as technological advances occur, regulator requirements will, in time advance. Golder Associates (UK) Ltd is, at the time of writing, producing and delivering an Advanced Landfill Engineering (ALFE) course for the Environment Agency. This course is intended to increase regulator understanding of the requirements of landfill design and construction practice, and it is envisaged that the requirements for detailed numerical analysis during the design process will be better appreciated following this course.

Whilst continuum modelling techniques have been applied to large strain waste containment situations, the continuum grid is often excessively deformed during flow around corners. Therefore, for benched landfill applications the grid must be carefully considered in order to allow movement to occur. The use of 'pseudo-stiffness' to represent waste settlement can be used to give vertical settlements. Where the grid is unrestricted by benches, vertical settlements of 20 – 30 % do not cause geometric problems, (i.e. the grid aspect ratio is such that zones are not significantly wider than they are tall prior to deformation). Where such settlements occur next to a benched subgrade, settlement of the geometry causes problems. A coarse grid will allow for greater deformations, however, at the expense of model resolution. Excessive deformations of the grid can cause calculation difficulties, preventing accurate strain predictions, and in extreme solutions will not be possible.

Theoretically, there is no limit to the number of geosynthetic liner components that can be included in a model, as zero thickness interfaces are used and the thickness of the geosynthetics is used in axial calculation, however, the thickness is not taken into account perpendicular to the beam element. Interface interpenetration occurs, particularly under high normal loads, as a function of the interface normal stiffness. The stiffness value cannot be set unrealistically high as numerical instability occurs, hence some interpenetration, albeit small, is unavoidable. The interpenetration is cumulative across the interfaces, hence if one beam is included between two grids (i.e. two interfaces) relative displacement of the two grids, normal to the interfaces, will be the sum of the interpenetration across the two interfaces (see Figure 4.7, section

4.4.3.2). From experience, a limiting number of three beam elements is acceptable with respect to interface interpenetration.

Interface stiffness has been highlighted as an area of importance during numerical modelling, as parameters are often assumed, or selection is made from shear strength data with a poor understanding of the influence of the initial stiffness on the behaviour of the numerical model. The selected stiffness value will create a straight line on the displacement – shear stress plot (see Figure 4.6) until a limiting value, controlled by the interface shear strength is reached. If the interface shear stiffness is set too low, displacement will occur without mobilisation of peak shear stress on the interface. This may result in overestimation of displacements and misrepresentation of integrity issues. Low interface shear stiffnesses are generally selected to avoid calculation difficulties and to reduce solution time, however, selection of the initial stiffness from direct shear data, at the highest expected normal stress, will prevent the loss of peak strength that can occur.

Complex waste constitutive models have been developed to include reinforcing elements (Machado *et al.*, 2002 and Krase and Dinkler, 2005) and with compressible particles (Zhang, 2007), however, due to their complexity, acquisition of meaningful input parameters and difficulty integrating them into commercially applicable software packages limits their use. Numerical models for commercial application must be simple and input parameters readily available. In this study the development of stress dependent non-linear waste models has allowed commercial application of such analyses where waste properties alter as a function of depth and volumetric compression. This combines improvements in accuracy of the model response whilst retaining ease and speed of application.

Following development of the analysis toolbox, this project focused on validation of the numerical modelling techniques. However, subsequent research by Sia (2007) implemented the numerical modelling approached for multiple layered landfill lining systems reported above with statistical variability analysis, allowing the influence of multi parameter variability to be assessed over a large number of repeated FLAC realisations in a Monte-Carlo analysis. Figure 5.1 shows the relationship between the concurrent research project by Sia (2007) and the research reported herein.

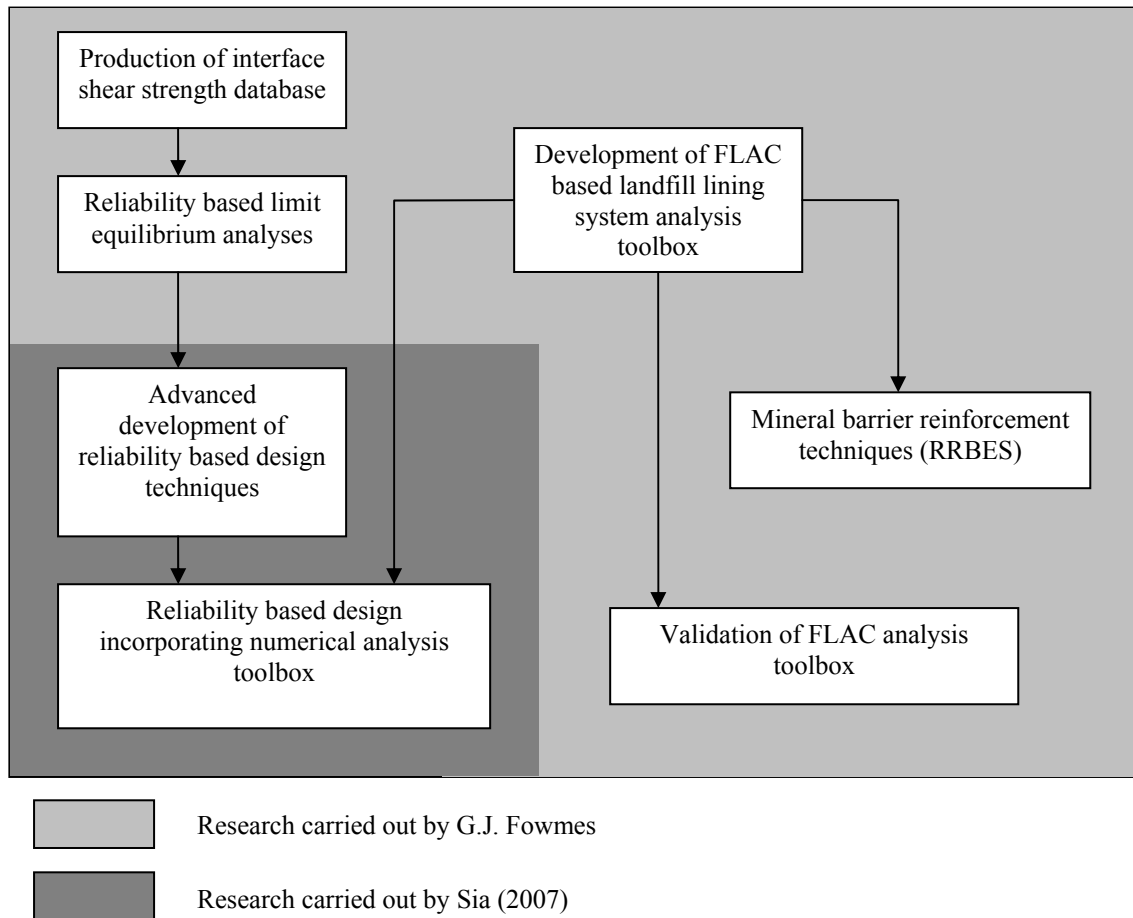


Figure 5.1 Interaction between this research and other Loughborough University landfill engineering research group activities

5.3 Validation of landfill lining analysis tools

Whilst several authors have carried out numerical modelling on landfill lining systems there is very limited evidence to validate the predicted behaviour against real world behaviour. Back analyses by Filz *et al.* (2001) and Byrne (1994) have suggested that numerical analysis of a progressive stability failure mechanism can reproduce an observed failure mechanism, although integrity assessment was beyond the scope of these analyses.

Villard *et al.* (1999) carried out numerical and experimental assessments of stability and integrity of a lining system in unconfined conditions, which showed an acceptable correlation, however, the experimental testing equipment failed so that confined measurements could not be made. Comparisons between the work by Villard *et al.*

(1999) and the FLAC numerical analysis toolbox show a good correlation with the measured behaviour up to the point of equipment failure.

Back analysis of a landfill failure in a steep sided quarry has shown that the failure mechanism could be reproduced at the correct waste depth (see section 4.7.2). The modelled failure mechanism closely reflected the observed mechanism and the position of the failure, and was attributed to post peak shear strength reduction beneath the geosynthetic layer. Whilst this showed that the correct behaviour could be predicted by a numerical model, comparisons to instrumented lining system behaviour would allow greater confidence in the numerical response.

As a site could not be accessed for a full scale field trial during the period of this research (see section 5.7.1) a large laboratory test was carried out, as discussed in Paper 5 and section 4.6. Numerical predictions of lining system behaviour based on input parameters from laboratory performance tests showed good correlation with the measured response. Whilst the exact values were not represented, given the simplification of the model, the correlation between the predicted behaviour and the measured results was considered to be good. The appropriate movements and mechanisms of post peak shear strength reduction were predicted by the FLAC model, in addition to the tension developed in the lining elements. This represents a significant development in the analysis of steep sided landfill lining systems, as it proves that the forces imposed on a lining system by a settling waste mass can be represented by a numerical model. It is envisaged in future that this will be further reinforced by validation using full scale field instrumented lining systems.

Thusyanthan *et al.* (2007) investigated tension in a scaled geomembrane on a shallow side slope in a centrifuge model. This modelling method allows a greater range of designs and materials to be investigated, although careful consideration of the scaling effects of each individual lining system component is required. Centrifuge tests may provide a useful tool in the assessment of downdrag force, particularly on shallower side slope, where hydraulic load application as applied in Paper 5 is not feasible.

5.4 Reinforcement of the geological barrier

In Paper 3 the potential for high strains in the geological barrier was identified. Whilst Miller and Rifai (2004) and Mayer and Ho (1994) discussed the use of fibres in fine grained soil, moisture content increases are required to facilitate mixing, which is not appropriate when constructing a mineral landfill barrier. The use of fibre reinforced bentonite enhanced sand was considered following the work by Michalowski and Cermak (2002) who showed that strength increases in sand could be achieved with the inclusion of fibres.

Addition of fibre reinforcements to BES showed significant strength increases when samples were mixed at optimum moisture content. However, when samples were mixed wet of optimum fibre pullout reduced the magnitude of the strength increase. Placement may be problematic as unlike with fine grained soils, unsupported steep slopes will not stand in the short term, thus limiting the application.

Two failure modes were identified by Al-Refeai (1991); fibre pullout and fibre tensile yield. Deconstruction of failed samples showed no evidence of plastic strain in the fibre reinforcements, indicating that failure by pullout was the preferential failure mechanism. This was likely to be due to the low frictional interface between hydrated bentonite and the smooth sided fibres. Longer fibres, with textured or fibrillated surfaces are suggested as methods of increasing pullout resistance, thus increasing fibre influence.

Consoli *et al.* (1998) demonstrated that the inclusion of fibres gave an increase in the peak strength of sand samples, but also observed an increase in residual strength, including a more ductile post peak response. Ductile post peak strength reduction is considered less likely to result in increased permeability than brittle behaviour.

The permeability of BES showed no increase with the inclusion of fibres, however, this was only assessed in falling head apparatus, conducted parallel to the direction of compaction. Due to the sub-horizontal orientation of fibres during compaction the permeability perpendicular to the direction of compaction is most likely to increase due to preferential flow path formation. Further investigation into the influence of

fibre reinforcement on permeability is required before this technology could be commercially implemented.

5.5 Steep sided landfill lining system design concepts

Gallagher *et al.* (2003) stated that the key design considerations for a steep sided lining system are:

1. Provision of an adequately smooth supporting surface as required;
2. Ensuring stability of the whole system; and
3. Selection of a system that will ensure the required support, integrity and compatibility of the barrier, and the protection layers in the context of the settling waste body.

Assessment (1) provision of an adequately smooth supporting surface, and (2) stability, were generally considered in design. The developed analysis toolbox allows an integrated approach to simultaneously assess (3) the required support, integrity and compatibility of lining system components. Whilst the design considerations presented by Gallagher *et al.* (2003) were written prior to the requirement for a geological barrier, point (3) above can be taken to refer to the support and integrity of both geosynthetic and mineral liner elements.

5.5.1 Lateral support

The waste will apply some lateral support to the lining system, which can be calculated using a coefficient of horizontal earth pressure K_0 (Dixon *et al.*, 2005). However, there is still some uncertainty surrounding the magnitude of applied support to the waste mass (Kavazanjian, 2006). The K_0 of waste governs the magnitude of support, and hence the requirements for the engineered support structure, however, due to waste variability this is difficult to quantify. Further data regarding the waste horizontal support should be generated from the instrumentation of landfill site “B” (see section 4.7.3.2).

In numerical analysis using FLAC, as with most commercially available numerical codes, the K_0 of the waste is not a direct input variable, and is calculated by the code

as a function of the constitutive model and input parameters. The designer must check that the applied lateral stresses in the model are representative of those believed to be present in the waste mass.

5.5.2 Three dimensional design considerations

Analyses of landfill lining systems are carried out in two dimensions with the assumption of plane strain conditions, however, in practice three dimensional geometry will exist and corners in the lining system and at the edges of caps and need to be considered. For inside corners where two inwards facing slopes meet, increased stability results, whereas outside corners will lead to reduced stability and potential for increased lining system stresses. Typically, side slope lining systems contain inside corners and capping lining systems contain outside corners, and at such locations the assumption of plane strain conditions may not be accurate. Three dimensional analyses is beyond the scope of this investigation, however, it should be considered as an area of further research. Additionally, variability in the geometry will exist due to construction tolerances and natural variations in angles of rock and soil slopes. When conducting numerical modelling, such features are treated as planar surfaces, therefore omit this irregularity.

5.5.3 Inclusion of the geological barrier

The requirement of the EC Landfill directive (1999) for inclusion of a geological barrier along the base and up the side slopes of landfill sites has greatly increased the complexity of constructing steep sided landfill lining systems. Large support systems are often required to maintain the stability of the geological barrier, and hence significant void is lost. In countries not influenced by the EC landfill directive, there is not a universal requirement for a geological barrier. Examples of alternatives are given by Cowland *et al.* (2006), who discusses a variety of conventional and innovative steep sided lining systems including concrete lining and sprayed elastomeric liners.

5.5.4 Drainage layer placement and integrity

Cowland *et al.* (2006) states that integrity of the draining layer is often considered as an afterthought in the design process, however, good drainage can greatly reduce the reliance on the barrier system. Geocomposite materials provide an alternative to

granular drainage layers, particularly on steep side slopes, when placement of granular material can be difficult. In steep side slopes, where effective drainage is provided, leachate and gas accumulation against the lining system will be prevented, hence, the drainage layer should be considered as a primary component of the barrier system. In near vertical lining systems the containment requirement of the lining system can be reduced by provision of effective drainage to avoid leachate accumulation against the lining system.

5.6 Steep sided landfill construction

5.6.1 Seam inclusions

Section 4.10 highlights the influence of horizontal seams on tension developed in a geomembrane exposed to down drag forces. When steep sided landfill lining systems are constructed in lifts, it is often necessary to have a horizontal welded seam at the top of each lift. The research presented in this document highlights the importance of using fusion welding techniques, rather than extruded welds, to create a smaller inclusion, and thus reduce stress transfer into the geomembrane.

As part of standard Construction Quality Assurance (CQA) procedures, samples of the geomembrane seams are removed for destructive tests, which are taken every 150 – 200 m. Where the samples are taken, an extrusion welded patch is included to repair the damage. In steep sided landfill lining systems it is recommended that such tests are not taken along the steep side slopes as the patch will act as a point of stress concentration. Agreement should be made with the regulator to take samples at the end of panels where excess geomembrane is present or on trial seams. In the CQA plan, consideration should be given to limiting the number of defect repairs by extrusion welding.

5.6.2 Placement of the geological barrier

Consideration of construction sequences and processes must be given during the numerical modelling process to ensure that the design can be constructed, and that the numerical model takes into account the important aspects of the construction process. Failures can occur in temporary compacted clay slopes prior to placement of support materials and waste.

The geological barrier requires compaction to achieve target permeability; however, the design may include a geological barrier on the side slope with a horizontal thickness of 0.5 m. Clearly this material cannot be placed and compacted as a 0.5 m thick column of material, hence a wider bench of mineral liner may be placed and cut back to the desired thickness. Where a reinforced soil, or engineered fill buttress is used in front of the barrier, it may be possible to place the fill, then excavate a trench at the rear of the fill to facilitate placement of the mineral barrier. This method allows placement of the mineral barrier with greater lateral confinement, thus improving compaction.

5.6.3 Construction plant

The design must consider both the influence of compaction plant on stability and integrity, and the access for construction and compaction equipment. Additional loading scenarios can be considered in both limit equilibrium models and the FLAC numerical model to account for plant loading. Where staged construction is carried out, it is the unconfined lining system that will be exposed to construction plant loading. The design of the landfill should take into account access for the construction plant to the steep sided lining system, which also facilitates material to be brought into place.

5.6.4 Health and safety

Any analysis of a design should consider constructability, not only in terms of what can be built, but also if the construction process may be dangerous. Steep sided landfill lining systems may introduce hazards to those working beneath the quarry wall, at or near limiting stability, working on or beneath constructed lining system as it is being constructed, and then as waste is placed. Such considerations should be taken into account from the conceptual design stage to allow appropriate lift heights and minimum bench widths to be derived. Under the Construction (Design and Management) Regulations (2007), which came into force 6th April 2007, it is the designer's responsibility to ensure that all potential hazards are eliminated or minimised. If health and safety and safe working practices are not considered at the preliminary design stage, significant redesign may be required later in the project.

5.7 Recommendations for future research

This project has identified several key areas requiring further research. The research suggested in this section is considered to be of significant interest to both academics and to design and construction practitioners.

5.7.1 Validation of the modelling

Whilst work has been carried out to validate the predictions of numerical modelling, full scale site instrumentation is required to compare the results of measured *in situ* liner performance to predictions using numerical modelling techniques. Ideally a variety of sites, with differing geometries, lining systems and slope angles should be instrumented to confirm the abilities of the numerical models to predict real world behaviour, and to refine the modelling with improved information on waste behaviour. This will allow increased confidence in numerical modelling and the use of numerical models for optimising designs.

Several attempts to instrument landfill sites have been made as part of this project, however, site operators have been concerned about potential repercussions if problems with the lining systems are found. Recent developments include the approval of a site operator for instrumentation of a steep sided lining system, at site B, to measure horizontal support provided by the waste, pore pressure development in the lining system, and installation of inclinometers to measure displacements of the facing blocks and the geomembrane. This lining system was designed using FLAC with application of the tools developed in this project and will provide a Class A, fully predictive, validation of modelled steep sided landfill support system behaviour. Further site instrumentation is required, including instrumentation of multiple layered geosynthetic lining systems to allow assessment of the relative displacements and geosynthetic tensile strains under loading.

5.7.2 Waste creep and degradation model

Current practice to represent waste settlement is to use waste with low stiffness, and hence represent downdrag due to waste compression and degradation behaviour. This is thought to give a falsely low representation of the horizontal support offered to steep sided lining systems by the waste and thus could lead to over-conservative designs.

The waste settlement model needs to include time dependent waste creep and degradation related volume loss with coupled changes in stiffness and strength, which will be dependent on the waste type. A key requirement of this model is ease of application in a commercial context, hence parameters should include the percentage volume loss due to degradation and the stiffness and strength changes in response to volume loss relationships.

5.7.3 Mineral barrier reinforcement

Randomly reinforced soil technology has been shown to have beneficial effects on the behaviour of BES, however, further work is required to quantify the effects of fibre reinforcement on hydraulic conductivity, particularly in the direction perpendicular to compaction forces. Additionally, the pullout of fibres in contact with hydrated bentonite is of concern and it is suggested that the use of fibres with fibrillated or textured sided may improve pullout resistance.

Continuous reinforcement and anchorage through geological barriers is typically omitted from designs due to concerns surrounding preferential fluid migration along the reinforcing elements. Whilst it is acknowledged that geotextiles would provide a preferential flow path due to their open structure, low permeability polymeric grids and solid anchors will only provide preferential flow at the soil - inclusion interface. For anchors the use of expansive clays may be used around the inclusion to reduce flow potential, but for geogrid reinforcements, placement of this layer would be challenging, costly, and provide a potential weak interface. There is also concern that mobilisation strains in the reinforcements may induce unacceptable strain in the barrier material. Further investigation into fluid migration through inclusions may allow their use in the future.

5.7.4 Mineral barrier permeability-deformation relationships

There is limited data available on the influence of strain on geological barrier layer permeability. Work by Arch *et al.* (1995) has been adopted to show that increased permeability is not observed with shear strain $< 10\%$. However, these investigations were carried out on kaolinite sand mixtures and are not necessarily representative of compacted clay barrier behaviour. Plasticity of the clay will control the strains that

may occur whilst still achieving required low permeability. This is considered an important area for future research into landfill liner performance and is of particular importance on steep side slopes where shear strains in excess of 5% have been predicted.

Over the duration of this project regulatory concerns regarding integrity have tended towards mineral barriers and not geosynthetic lining systems. Simple constitutive models are still applied to compacted clay landfill liners and a poor understanding of fluid pressures and suctions during construction still exists. The relationship between mineral barrier strain and permeability is poorly understood. Simple shear apparatus may be implemented to induce shear strains in material whilst permeability measurements should be considered in samples from vertical and horizontal orientations. An understanding of the response of permeability to strain as a function of plasticity would allow designers to set realistic limiting design criteria.

6.0 Implications for Industry

6.1 *The key findings of the research*

The key findings of the research are as follows:

1. The variability of interface shear strength can have significant implications on the results of limit state design analysis. Even analyses with apparently acceptable factors of safety can yield apparently unacceptable probabilities of failure when input parameter variability is considered.
2. FLAC can be used to assess the deformations of, and strains within, landfill lining system components, including multilayered geosynthetic landfill lining systems.
3. The FLAC model for landfill lining system performance has been validated against observed and measured landfill field behaviour in confined and unconfined conditions and against the behaviour of a lining system subject to downdrag forces at laboratory scale. Further field data is required to assess the behaviour of multilayered geosynthetic lining systems.
4. Random fibre reinforcement can be used to increase strength in fibre reinforced soils with no observed increase in permeability, but concern still surrounds the pullout of fibres in contact with hydrated bentonite.
5. The presence of horizontal welded seams, in particular extrusion welded seams, can significantly increase the tension developed in a geomembrane exposed to downdrag forces.

In addition, an analysis framework has been produced in line with the Environment Agency *de facto* guidance, giving advice on the selection and execution of landfill stability and integrity analyses. This includes analysis methods at a variety of technical levels including limit equilibrium and numerical modelling.

6.2 *Implications for Golder Associates*

The interface shear strength data is required for a large percentage of the landfill stability risk assessments carried out by Golder Associates, in the UK and internationally, and the interface shear strength database has allowed comparisons

between site specific interface tests to be compared to a global database, thus giving greater confidence in the selected design parameters.

The development of the multilayered lining system analysis techniques, in FLAC, has allowed Golder Associates to remain at the technical forefront of landfill stability and integrity risk assessments. Working commercially during the Engineering Doctorate programme has allowed the project to be directly tailored to the needs of industry, and the research reported in this thesis has been directly applied in a variety of landfill design projects. The design toolbox document has formed the basis of design practice and has been used by both junior and experienced engineers whilst conducting landfill stability and integrity analyses.

6.3 Implications for wider industry

The work on interface shear strength raises concerns surrounding the use of published data for interface shear strength without some site specific testing. Even with factors of safety well in excess of normally accepted levels, unacceptable probabilities of failure have been generated. The use of site specific interface testing should be used where possible, although there is still variability in the recorded measurements (Sia and Dixon 2007).

The quantification of the variability associated with literature reviews and presentation of summary plots for the literature database allows the designer to make an assessment of the data they have received and to make a conservative estimate of the mean. With the advent of the requirement to design under EN 1997-1:2004 (Eurocode 7) the need for documentation of the derivation of conservative estimates of the mean value will increase, and this database will assist with this process.

The use of numerical analysis in landfill design is becoming more commonplace, particularly using the finite difference modelling code, FLAC. Whilst reliance is being placed on such designs to assess the deformations and strains within lining system components, there is little evidence to validate the behaviour predicted by numerical modelling techniques compared to field performance. The numerical analysis techniques for assessing multilayered landfill lining systems, allows analysis

of complex lining systems and the validation of the displacements and generated stresses in the geosynthetic lining system elements, allows greater confidence in the application of these analysis.

6.4 Critical evaluation of the research

The validation of the numerical modelling was ultimately limited by the lack of full scale testing. Despite significant time and effort being expended in an attempt to instrument a steep sided landfill lining system, this has only become a possibility very recently. However, the information derived from the laboratory testing data allowed for high quality alternative data to be acquired.

Whilst randomly reinforced fibres showed that strength increases could be achieved when mixed with bentonite enhanced soil, it did not yield a solution to clay behaviour at strains which are associated with bench corners, due to concerns surrounding fibre pullout when coated with hydrated bentonite. The testing associated with this part of the project was limited to compaction, quick undrained triaxial tests, and falling head permeability tests. A more comprehensive suite of laboratory tests and field trials would be required before this technology could be properly assessed for commercial application.

With regulatory concerns tending to focus on the integrity of mineral lining systems and not that of the geosynthetic lining system, development of advanced analysis tools to focus on this part of the design may have better served commercial design practice. However, it is envisaged that as regulator awareness increases, the requirement for geosynthetic assessment will increase.

7.0 Conclusions

The published literature indicates that interface shear strength is a primary controlling factor in the behaviour of multilayered geosynthetic and soil landfill lining systems. The interfaces between geosynthetic and soil layers provide potential slip surfaces, and the magnitude of the shear stress is dependent on both the normal stress and the displacement. As municipal solid waste experiences settlement, displacement at interfaces is unavoidable, and engineered designs must account for this and ensure that the displacement occurs along specified planes.

A database of over 4200 data points, each representing a peak and residual shear strength at a given normal stress has been produced. The database shows variability greater than typically accounted for in the design process. At a typically adopted factor of safety of 1.3, reliability analysis yields probability of failure in excess of 1% (1 failure per 100 designs), which is not considered acceptable. The database can be used for preliminary design or comparison with measured results; however, this highlights and confirms the importance of site specific interface shear testing.

This project has developed a modelling technique, based on the FLAC numerical code that is applicable to commercial projects and research applications. The modelling toolbox developed as part of this analysis is capable of being used in assessment of multiple layer geosynthetic and soil lining systems, with the inclusion of strain dependent interface and geosynthetic properties. The model can include the complex loading induced by the waste mass, and has the ability to include varied subgrade and engineered support systems throughout a full staged construction process.

Validation of the modelling technique against published values, observed field behaviour and large scale laboratory tests has produced a satisfactory correlation between observed/measured and predicted behaviour. Further field scale landfill lining system instrumentation would allow validation of the model response during construction, post waste placement and in the long term, thus not only providing validation of the modelling presented here, but also for future models including time dependent degradation generated settlement.

Numerical modelling has identified that with correct selection of liner materials, interface stresses can be controlled and thus build up of tension in geosynthetics can be avoided. In steep sided landfills, changes of slope angle, particularly outside corners at benches, can cause stress concentration in the geological barrier.

The use of fibre reinforced bentonite enhanced sand has been considered to provide a high strength material for use in areas of high predicted strain, although concerns exist surrounding the low interface friction between the fibres and the bentonite post hydration. At present, design of a lining support system is deemed to be the appropriate method for reducing strains in the geological barrier.

Construction details have been considered including the presence of horizontal welded seams, which are often a necessity during staged construction of a lining system in lifts. Laboratory tests showed small increases in geomembrane tension due to the presence of the fusion welded seams. Extrusion welded seams were shown to increase the geomembrane tension by nearly 50%.

This project presents significant advances in the development of analysis tools for use in the design of landfill lining systems, particularly for steep side slopes. The modelling technique developed allows the simultaneous assessment of stability and integrity of complex multicomponent lining systems. Previous work towards validation of numerical models for landfill lining systems has been limited, and it is believed that the work carried out in this project provides significant progress and increased confidence in the application of these methods.

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Articles

Fowmes, G.J. (2005c). Innovative landfill lining systems. Innovation and Research Focus
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10.0 Appendices

Appendix 1	Paper 1	Fowmes, G.J., Dixon, N. and Jones, D.R.V. (2007b). Landfill stability and integrity: the UK design approach. <i>ICE J. of Waste and Resource Management</i> . May 2007, WR2, 51–61.
Appendix 2	Paper 2	Dixon, N., Jones, D.R.V. and Fowmes, G.J. (2006). Interface shear strength variability and its use in reliability-based landfill stability analysis. <i>Geosynthetics International</i> . Vol. 13, No1. p 1.
Appendix 3	Paper 3	Fowmes, G.J., Jones, D.R.V. and Dixon, N. (2005c). Analysis of a landfill directive compliant steepwall lining system. <i>Proceedings 10th International Waste Management and Landfill Symposium</i> , Sardinia. pp369.
Appendix 4	Paper 4	Fowmes, G.J., Dixon, N., Jones, D.R.V. (2006a). Use of randomly reinforced soils in barrier systems. <i>Proc. 5th International Congress on Environmental Geotechnics</i> , Cardiff, UK, June 2006. 709 – 716.
Appendix 5	Paper 5	Fowmes, G.J., Dixon, N. and Jones, D.R.V. (2007c). Validation of a numerical modelling technique for multilayered geosynthetic landfill lining systems. <i>Geotextiles and Geomembranes</i> 26 (2008) 109–121.

Paper 1: Landfill Stability and Integrity: The UK design approach

G.J. Fowmes, N. Dixon, and D.R.V. Jones

Full Ref:

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.

Abstract

This paper highlights the design considerations, in terms of stability and integrity, for EC Landfill Directive compliant sites. The paper details a design chart based on research and development reports produced for the Environment Agency (England and Wales) intended to guide the designer and highlight the areas for consideration in each of six aspects of landfill construction: subgrade, basal lining system, shallow slope lining system, steep slope lining system, waste slopes, and capping lining systems. The paper is not intended to offer design methodologies in terms of which calculation methods should be adopted, but to provide designers with a framework in which to apply engineering skill and judgement and to highlight challenges.

Keywords

Landfill Stability, Integrity, Design, Lining systems,

1.0 Introduction

A report conducted on UK landfills¹ identified over 85 failures. Although the majority of these failures were recorded during landfill construction, and hence could be easily remediated, there were still significant cost implications. The failures were primarily attributed to inadequate site investigation, uncontrolled groundwater and inappropriate design. This highlights the need for guidance on the design of landfill lining systems.

A literature review, R&D technical report P1-385/TR1², and a guidance document, R&D technical report P1-385/TR2³, have been produced for the Environment Agency for the assessment of the stability and integrity of landfill lining systems. Application of the guidance provides a framework for the assessment of six aspects of landfill stability and integrity: subgrade, basal lining systems, shallow slope lining systems, steep slope lining systems, waste slope stability, and capping lining stability.

In England and Wales landfills are required to obtain a Pollution Prevention and Control (PPC) permit. The stability risk assessment requirements for the permit application are based on P1-385/TR2. This paper presents the design considerations for each of the six cases and discusses the key considerations. To date, over 600 copies of Report TR2 have been distributed by the Environment Agency in response to orders from designers, operators and researchers both in the UK and overseas. This paper considers the key aspects of landfill design and provides an updated summary of current design approaches and highlights areas for future consideration.

1.1 Typical lining system

A typical lining system comprises of barrier, protection and drainage layers formed from geological (e.g. clay and gravel) and geosynthetic (e.g. geomembrane and geotextile) materials. Figure 1 shows a typical lining system. A barrier layer is required to limit the leakage of fluids (both liquid and gas) from the waste mass into the surrounding environment. Examples of barrier layers include compacted clay, bentonite enriched soil (BES), colliery spoil, polymeric geomembranes and geosynthetic clay liners (GCL).

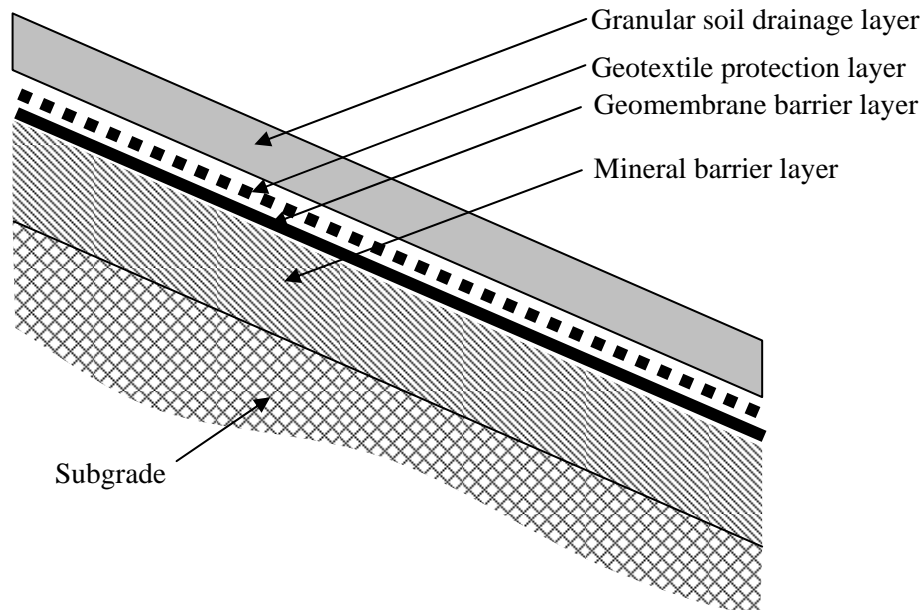


Figure 1. A typical landfill lining system.

For non-hazardous landfills (e.g. those taking municipal solid waste), the EC Landfill Directive (1999)⁴, enforced in the UK through the Landfill Regulations 2002⁵, requires a geological barrier to aid attenuation of contaminants. This should have properties equivalent to a thickness of 1m and hydraulic conductivity of $1.0 \times 10^{-9} \text{ ms}^{-1}$, and is required along the base and up the sides of landfill sites. The thickness can be reduced to a minimum 500 mm if the permeability of the barrier layer is decreased. In the UK the geological barrier is typically formed by compacted clay or BES (artificially established geological barriers) and may also include low permeability in situ materials (natural geological barriers)

In order to further reduce the leakage from the landfill, composite liners can be used where a combination of barrier materials provide a greatly reduced hydraulic conductivity. The most common composite lining system involves a compacted mineral liner overlain by a polymeric geomembrane, where the geomembrane is in close contact with the mineral liner; hence any fluids migrating through defects in the geomembrane must still pass through the mineral liner whilst the hydraulic head across the mineral liner is significantly reduced by the presence of the geomembrane.

It is important to protect the geomembrane from damage both during and following installation. A geotextile protection layer is typically placed above a geomembrane to protect the membrane from overlying materials. The performance of a lining system is improved by the control of leachate head acting on it, and it is common practice to include a drainage layer above the lining system. The drainage layer can be a granular mineral layer or a polymeric drainage composite.

2.0 Design Issues

2.1 General

Site investigation should provide designers with confidence in parameters selected for use in analyses. Inadequate site investigation may not only lead to inappropriate design parameters being selected but can also lead to critical failure mechanisms being overlooked. For landfill design, knowledge of the subgrade, groundwater regime and material properties of in-situ and engineered materials is required. In an ideal scenario the waste mechanical properties would be known but due to the waste heterogeneity, composition and particle size this is difficult, although ranges of likely waste parameters must be obtained as discussed in section 3.4.2.

When analysing barrier layers the allowable strains must be considered. In a mineral barrier this will depend on the soil plasticity. High plasticity fine grained soil may be subjected to greater deformations than low plasticity soil before the permeability of the soil rises. For polymeric geosynthetic liners, the polymer type, material thickness and design life will all affect the allowable strains. The designer must justify the selected materials and the design to show that the allowable deformations do not adversely affect the barrier performance.

A very important design consideration is how the system will be constructed. Safe working must be ensured during construction as detailed in The Construction (Design and Management) Regulations (2007)⁶. It is the designer's responsibility to ensure that all potential hazards are eliminated or minimised. Working at heights and at the base of high slopes must be kept to a minimum; this is of particular importance in steep sided landfill lining systems.

The design life of a geomembrane will depend on the polymer type, additive package applied to the polymer and environmental conditions, including the temperature generated within the landfill and the leachate chemistry. It is widely accepted that the geomembrane will have a finite period of functionality and the design life of the system must be sufficient to allow the waste mass to become stable and all contaminated leachate to have been treated. In the long term, geosynthetic lining system components will degrade⁷. A designer must firstly consider the impact of this on the containment function of a lining system. The hydrogeological and landfill gas risk assessments should take into account the loss of functionality of polymeric lining system components at a given number of years into the life of the system.

2.2 Interface shear strength considerations

The interfaces between lining system components, in particular where planar geosynthetic materials are present, may provide preferential slip planes. The strength of these interfaces can be highly variable, and large variability in shear strength can result from the testing laboratory used, as well as material variations⁸. Literature data should be used with caution as they may not be representative of the onsite conditions. Site specific testing should be specified to verify or enable detailed design to be carried out, and should involve performance, rather than index, testing with representative material above and below the interface. In addition to interface shear strength variability, if composite materials or Geosynthetic Clay Liners (GCL) are used, the internal shear strength will also need to be considered. GCL internal shear strength variability is high when comparing different manufacturing lot specimens, the natural variability of the bentonite material is also shown to play a part in this⁹. It is important to specify site specific testing in order to reduce the uncertainty associated with interface shear strength variability.

In Eurocode 7 (1997)¹⁰, the characteristic value of a soil property is defined as: ‘A *cautious estimate of the value affecting the occurrence of the limit state*’. With mineral, polymeric and interface properties, an understanding of the associated variability of the parameter will allow the designer to select a cautious estimate. Increased site specific testing will increase the confidence in the selected parameters.

Leachate and gas pressures may be present at an interface. Designers must consider if fluid pressures could act on an interface. Interfaces separated from the waste mass by a barrier layer are less likely to have elevated leachate and gas pressures acting on them than those not separated from the waste. In limit equilibrium analyses the use of peak or residual values in design can greatly influence the calculated factors of safety for a given design^{11,12}. The use of peak strength may be unconservative if sufficient displacement occurs to induce post peak shear strength reduction¹³. A landfill failure was reported where a compacted clay barrier layer failed along an interface with an underlying polymeric geocomposite drain. Back analysis of the failure showed that post peak interface conditions were required to generate a factor of safety of 1. It is believed that repeated plant loading from heavy dump trucks using a diagonal haul road in the vicinity of the failure caused post peak shear strength to be mobilised. Numerical modelling techniques can be used to represent strain dependent interface shear strength, although concerns still surround the accuracy of the input data associated with such analyses¹⁴.

3.0 Design Cases

Figure 2 shows the six main elements of a landfill containment system, and Figure 3 presents the individual design considerations for each of the design cases, including key controlling factors, that should be considered when assessing landfill stability and integrity. The fundamental aspect of a safe design is to select the potential critical failure mechanisms, either in terms of stability or integrity, then to assess these using relevant analysis methods using site specific parameters. Although each aspect of design should be considered, this does not imply that a calculation must be done in every case. A logical argument can be put forward as to why a particular failure mechanism is not considered to be likely.

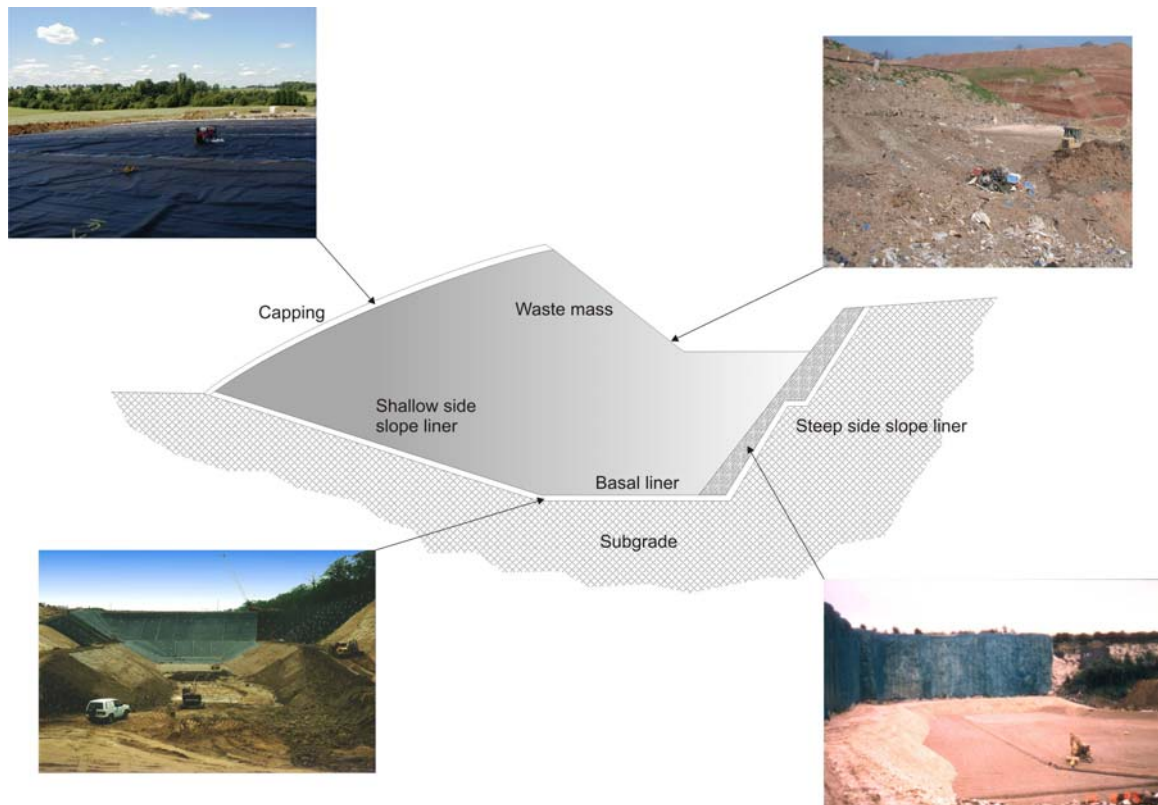


Figure 2. Landfill lining system design cases.

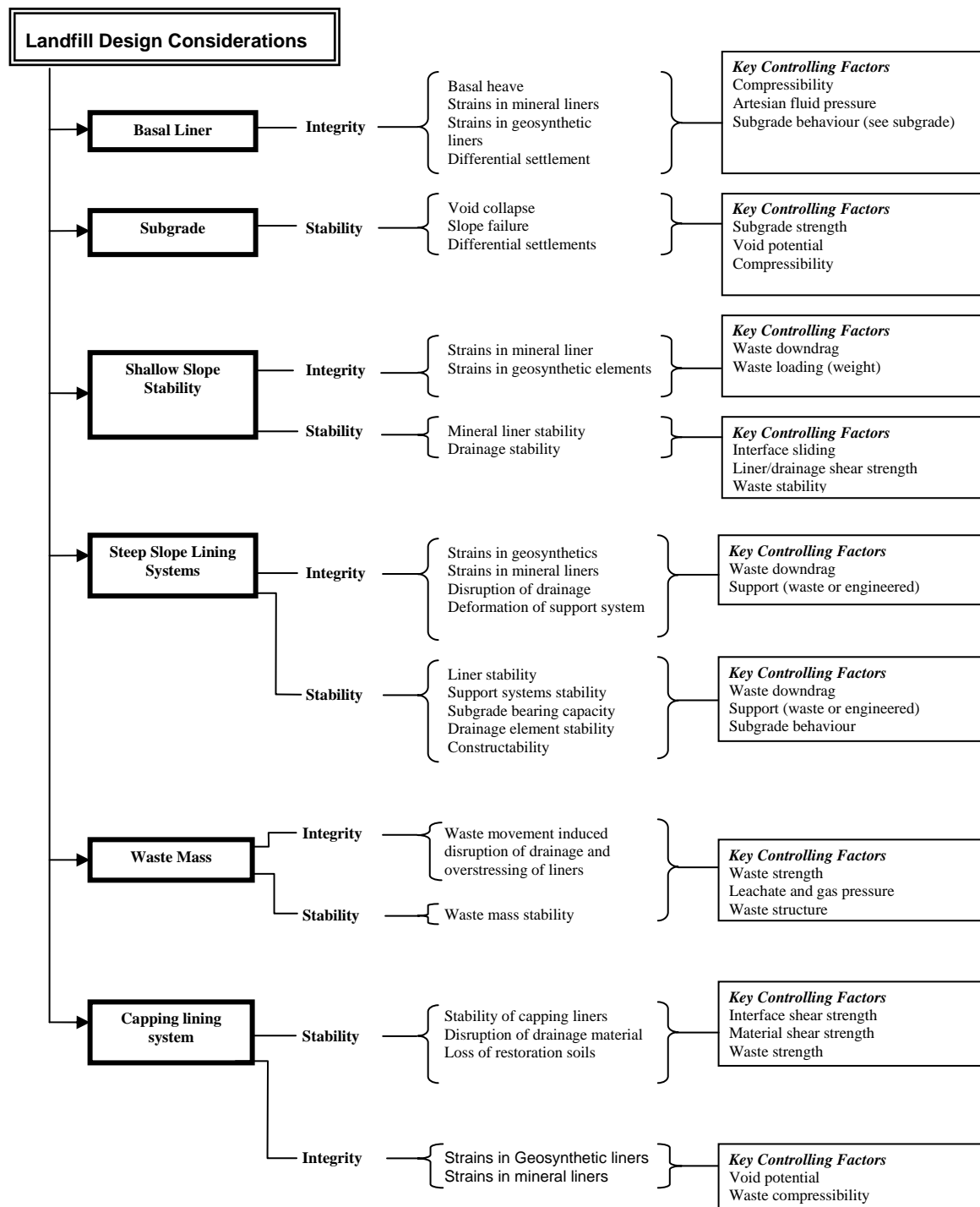


Figure 3. Landfill lining system design considerations.

The lining system and subgrade should be considered prior to, during, and post construction. Unconfined conditions occur in the subgrade prior to construction and in the lining system prior to waste placement. In side slope lining systems, particularly those on steep gradients, the absence of horizontal support at this stage means that

stability is dependent on the internal strength of the subgrade and the lining system. During the transient construction stage, the subgrade will become confined, then following waste placement the lining system will also become confined, however, the additional weight of the overlying material and downdrag associated with waste settlement means that consideration of the stability and integrity post waste placement is still required. The following sections address in turn each of the six overall design cases.

3.1 Sub Grade

When assessing the subgrade, the potential failure scenarios that should be considered include: basal heave, slope instability, potential void formation, and subgrade compressibility (see Figure 4). The considerations will be different depending on whether the base is a natural or engineered subgrade or if it overlies existing waste. To assess cavities in a non-waste subgrade, adequate site investigation is required to gain an understanding of the potential for soil collapse and settlement. For a waste subgrade, the waste stream, method of placement and compaction and age of the waste will need to be considered in order to assess the likelihood of void formation and the expected magnitude and distribution of settlements, both total and differential.

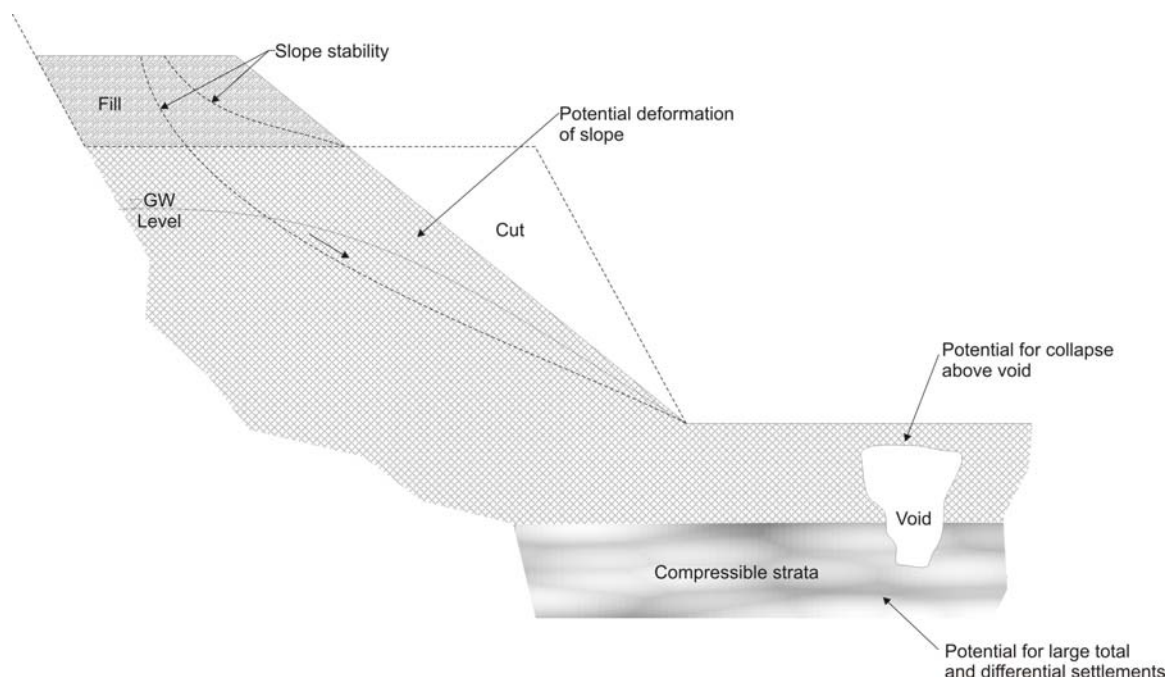


Figure 4. Subgrade failure mechanisms.

The subgrade should be stable, both during and post construction, to prevent movements that may damage the overlying lining system. Stability must be assessed for cut, fill and natural slopes (see Figure 4). For sloping subgrade the design issues will include stability, deformations and void potential. For slopes in fine grained soils, time dependent failure mechanisms should also be considered, as short term stability does not guarantee long term stability even following waste placement¹⁵. Rock mass stability assessments should consider the nature of the rock mass and jointing patterns in order to ensure that the slope will remain stable during and post waste construction. In quarry landfills the stability of rock slopes must be considered as many old quarry slopes are marginally stable following extraction, and re-profiling of such slopes may be required for safe working.

3.2 Basal lining system

For a basal lining system, potential deformations in the subgrade and lining elements leading to overstressing of barrier layers and loss of function, must be considered. Basal stability must either be assured (Section 3.1) or the likely deformations during the life of the facility must be accounted for in design of the lining system.

3.2.1 Influence of the subgrade on the lining system

Settlement of the subgrade may occur due to loadings from the overlying waste. The lining system must be designed so as to retain its integrity if subjected to differential settlements (see Figure 5a). The potential for cavities in the subgrade may require the inclusion of a geogrid to provide support to the lining materials and waste mass over the void¹⁶. Geogrids can be used, where significant total and differential settlements are likely to occur, as a means of averaging settlement profiles across the site. The allowable deformations in a clay liner due to the settlement vary depending on the soil composition, stress state and plasticity¹⁷.

3.2.2 Subgrade fluid pressure and basal heave

Pore fluid in the subgrade can generate hydraulic gradients in a mineral barrier leading to softening and potential shear failure. A landfill failure was reported resulting from softening of the compacted clay barrier layer due to uncontrolled

groundwater in the mudstone subgrade . This occurred during periods of heavy rainfall producing a high hydraulic gradient of upward flow through the liner together with the build up of surface water. This resulted in saturation of the clay liner and the toe of the side slope and slumping of the clay. Under-drainage was installed and pumping carried out to relieve the pore pressure in the subgrade. In order to ensure barrier integrity is maintained, fluid pressure in the subgrade should be considered for the design life of the landfill site, and under drainage installed if necessary.

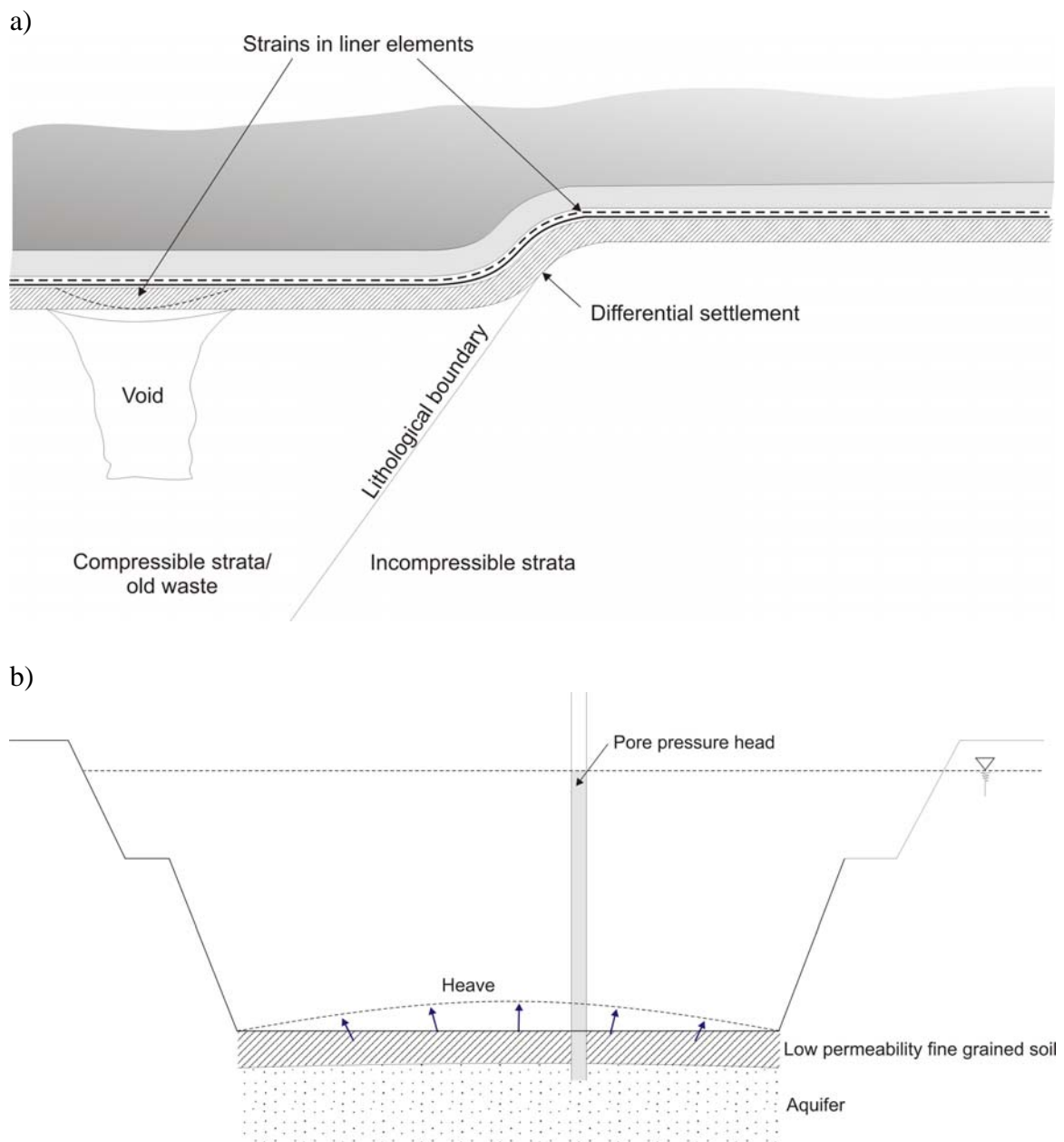


Figure 5. Basal lining system failure mechanisms: a) differential settlement and void collapse, and b) basal heave.

Basal heave can occur if the pore water pressure at a given depth in the sub-grade is greater than the total stress from the overlying strata (see Figure 5b). The presence of a natural low permeability layer or the placement of a low permeability fine grained barrier layer will prevent release of this fluid pressure into the landfill void and hence upward movement of the landfill base may occur. Basal heave is predominantly an issue prior to waste placement, however, may occur following waste placement if the waste mass exhibits a low unit weight.

3.3 Side slope lining systems

Design considerations highlighted in this section are for both shallow and steep sided slopes. Stability and integrity of the liner elements, as well as the stability of the subgrade, should be considered both pre (Figure 6a) and post (Figure 6b) waste placement, as waste placement introduces additional forces into the lining system. As with a basal lining system, the behaviour of the subgrade can also control the integrity of the side slope lining system. Time dependent waste degradation and creep can occur, introducing additional forces in liner elements post closure.

3.3.1 Waste settlement induced strains in lining elements

Waste downdrag must be considered in both waste supported and self supporting lining systems. The behaviour of such a system can be likened to negative skin friction in a pile foundation. As the waste settles due to both the weight of successive waste lifts and subsequent settlement due to degradation and creep, the downwards movement of the waste mass will induce downdrag forces on the lining system. These forces must be dissipated to prevent overstressing of geosynthetic barrier layers or loss of function of protection layers (Figure 6b and 7). Two design philosophies can be adopted in order to preserve the integrity of the underlying lining system. Firstly the system can be made sufficiently robust so that it is unaffected by these additional forces, or alternatively the system can be designed to dissipate the stresses through the use of preferential slip planes or sacrificial materials in order to prevent stress transfer into the lining system. Pre-compacted waste or inert fill can be placed adjacent to the waste barrier interface to buffer the lining system from the settling waste mass and also improve lateral support conditions.

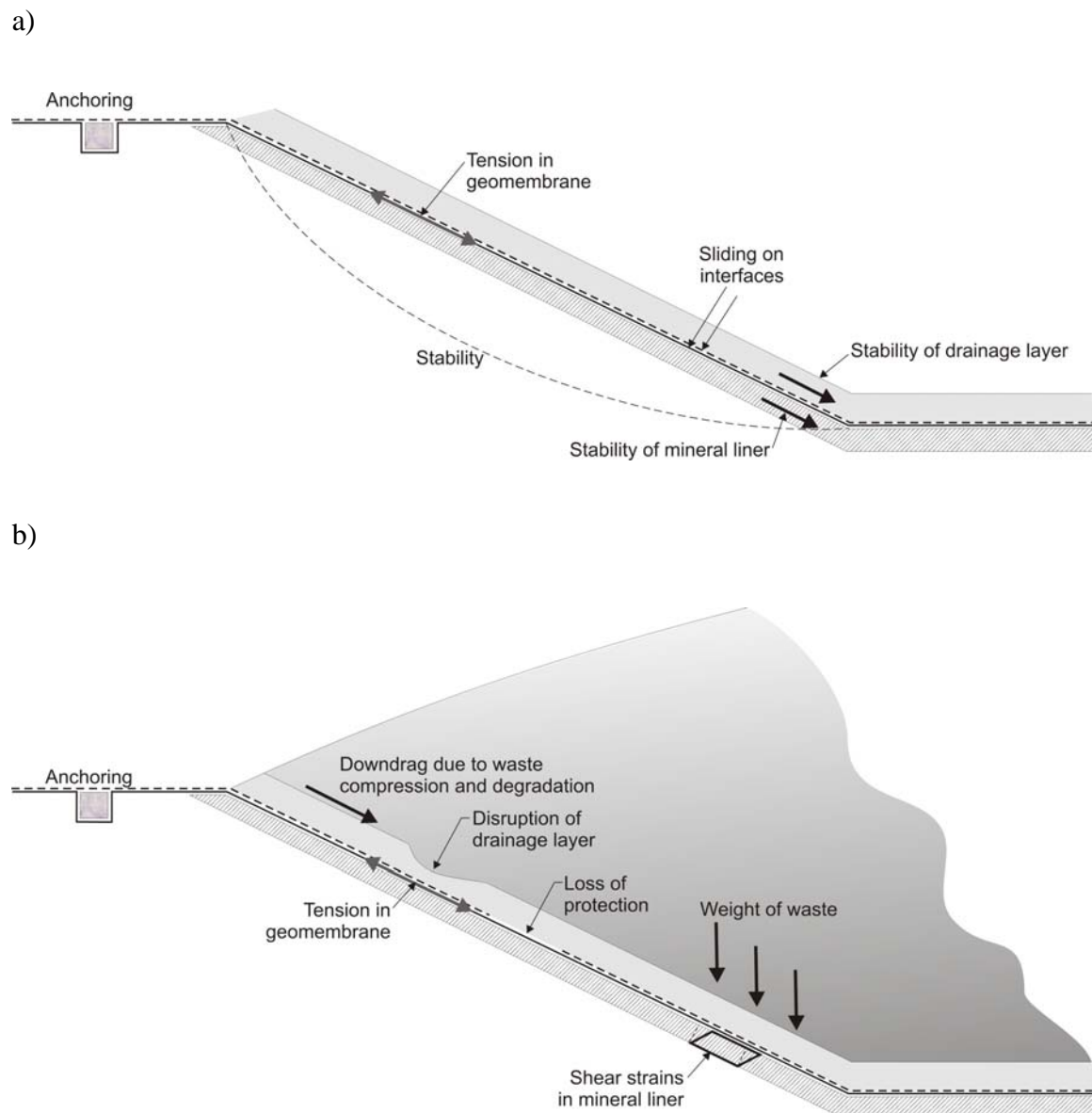


Figure 6. Side slope lining system failure mechanisms: a) unconfined, and b) confined.

Drainage layers can become distorted or localised failures can occur due to insufficient horizontal support and waste downdrag (Figure 6b). The drainage layer is essential to prevent the accumulation of leachate against the lining system and to relieve gas pressures.

3.3.2 Geosynthetic anchorage

As part of the assessment of geosynthetic interface sliding, and geosynthetic barrier layer integrity, the anchorage of a geosynthetic system must be considered (see

Figures 6a and 6b). If geosynthetics are used in the lining system, these can be anchored to prevent uncontrolled sliding of the geosynthetics. If one end of a geosynthetic is fixed then stresses transferred into the lining system will result in tensile stresses and associated strains within the geosynthetics.

During construction, temporary anchoring of geosynthetic elements may be considered in order to restrict relative movement. Where waste is constructed in lifts, the waste will be placed prior to a final permanent anchorage being established. In this case the effect of the self weight of the waste and settlement induced downdrag will already be acting on the material before the permanent anchorage is complete. During the final stages of construction, the geosynthetic may be permanently anchored to resist slippage occurring during final stages of waste placement, compaction and post placement settlement.

Consideration must be made as to whether geosynthetic anchorage is required. For example, the upper layer of a two layer slip surface geotextile system may not be anchored at all to allow for movement on the lower interface. However, the lower layer should be anchored as typically the requirement would be that this layer does not slip. An argument can be put forward that geosynthetic anchorage should not be required as it only acts to concentrate stresses and only comes into effect if slippage has occurred in the interfaces: Interface engineering should be the primary method of slippage prevention, not anchorage of the geosynthetics.

3.4 Steep side slope lining systems

Steep side slope lining systems present additional technical challenges highlighted below. Figures 3 and 7 demonstrate design considerations for a liner construction that must be satisfied both with and without waste support. With waste support the lining system is constructed ahead of waste placement but, unlike self supporting lining systems, it relies on the waste to apply a horizontal force resisting movement of the lining system. Waste is considered to be part of the liner support system. The key design considerations for a steep sided lining system have been described as follows¹⁸:

- Provision of an adequately smooth supporting surface as required;

- Ensuring stability of the whole system; and
- Selection of a system that will ensure the required support, integrity and compatibility of the barrier, and the protection layers in the context of the settling waste body.

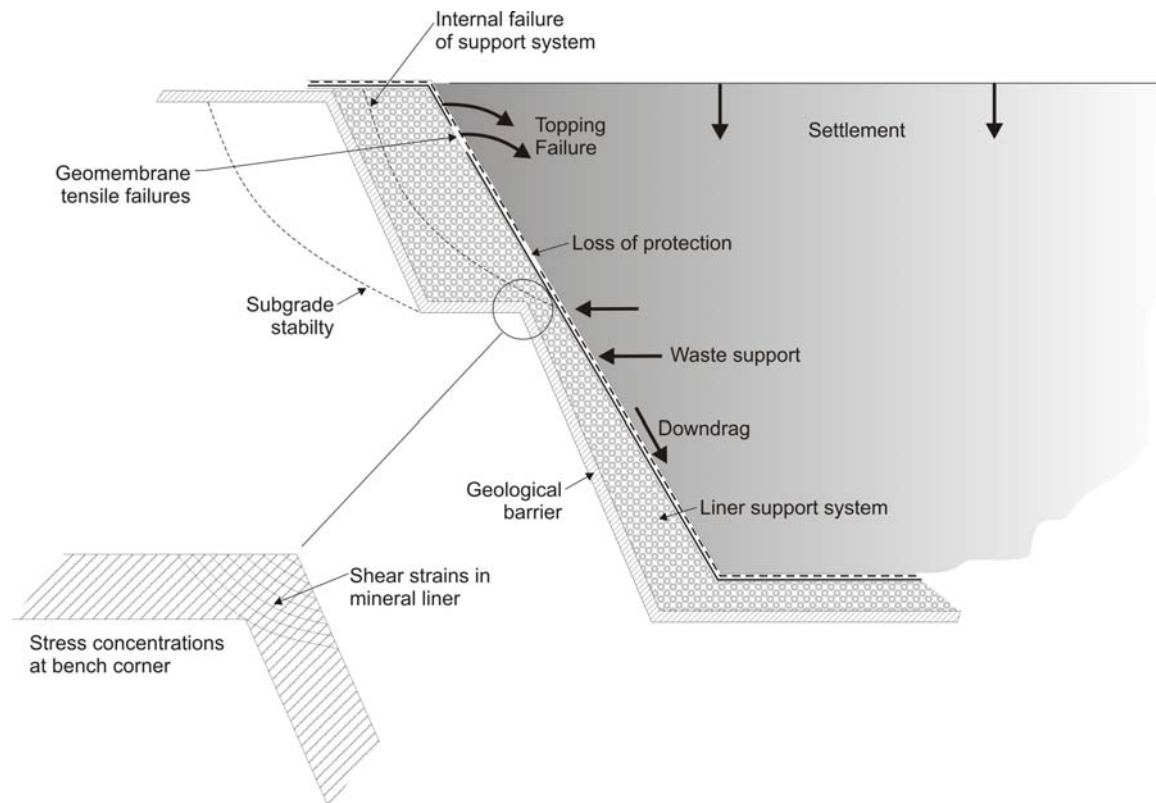


Figure 7. Steep slope lining system failure mechanisms.

The leachate and landfill gas containment requirements must still be retained in the steep slope lining system¹⁹. In addition, mineral and artificial components of the lining system should be considered from both a stability and integrity point of view and it is important to integrate these aspects to produce an effective yet viable design. Possible stability failure mechanisms include shear failure of the lining system and toppling of the lining system, whilst integrity failure can occur as a result of geosynthetic element overstressing and straining of mineral barrier layers (Figure 7)

3.4.1 Inclusion of a geological barrier

The EC Landfill Directive (1999) requires the inclusion a geological barrier along the base and up the side slopes of a landfill, regardless of the angle of the lining side

slopes. Clay barriers can be placed on slopes of up to 1(vertical):2(horizontal) for relatively small heights²⁰. Greater slope angles are required for use in steep wall lining systems hence a compacted clay barrier layer would need some lateral support to maintain stability. Strains in the lining system can occur due to the waste overburden stresses or due to waste settlement (downdrag). The dominant shear force is dependent on waste material and side slope angle. Assessment of shear stresses in lining elements due to waste weight and downdrag may be estimated by the use of numerical methods, although there is still a requirement for validation of such analysis techniques.

A reported field trial to investigate the interaction of a steep (80°) side slope compacted clay barrier system, supported by a gabion wall and waste, gave the following findings²¹:

- The barrier experienced significant vertical and horizontal strains, with the magnitude dependent on the stiffness of the waste body;
- The method of construction, including the phasing of barrier construction and waste lifts, has an influence on the magnitude and distribution of barrier deformations;
- Differential vertical strains were found in the barrier components; and
- A number of failure mechanisms were predicted resulting from the magnitude of deformations required for equilibrium between the barrier and waste body. These included shear failure, bulging failure, toppling failure and bearing failure.

Failure of a compacted clay lining system in the UK has been reported²², where the liner suffered a toppling failure and moved away from the quarry wall due to the lack of support from the waste. This study showed that the findings listed above are relevant to UK practice, and that the current UK waste stream placed using typical compaction practice is not suitable for supporting a clay only barrier system on steep slopes.

The geological barrier may be stable in the short term due to its high undrained shear strength. However, in the long term, drained conditions will occur and the barrier will require support. Internal support has been considered for strengthening the geological barrier; however, any form of continuous reinforcement may provide a preferential flow path through the materials. The use of discrete fibre reinforcements has been considered²³. Inclusion of such reinforcement in stiff clays is reported to be unviable, and strength increases in bentonite enhanced soils are only reliable if the moisture content of the material remains low enough as not to lubricate the soil fibre interface. Due to these concerns associated with internal support approaches, it is preferable for the clay barrier to be externally supported, either by the waste or by an engineered supporting structure. This could consist of an engineered fill wedge or a reinforced soil structure. Care must be taken if designing a benched quarry as the support for the geological barrier may have to be placed on top of the geological barrier in subsequent lifts. There is potential for increased strains in this scenario (Figure 7), particularly at the corners of the benches. Numerical analysis is often necessary in order to assess the deformations that will occur in the geological barrier.

3.4.2 Waste Support

A self supporting lining system can be constructed to its full height in the absence of waste support, however, two financial factors restrict the use of such systems: construction cost and, if the system is wide, loss of void space (loss of revenue). When considering a waste supported lining system, it should be noted that two distinct aspects of support will act on the lining system. Firstly, the waste will apply some lateral support, which can be calculated using a coefficient of horizontal earth pressure K_0 ²⁴. Secondly, should the lining system begin to deform then the waste stiffness will control the magnitude of deformation and may prevent stability failure. It should be appreciated that movements to mobilise the waste resistance can be large due to its low stiffness and hence integrity failures may still occur. If waste support is insufficient then shear or toppling failures of the lining system may occur (Figure 7).

In addition to waste composition, lateral support is also a function of compaction practice adjacent to the lining system. There is a tendency on site to avoid compaction directly adjacent to the lining system due to a concern that this may result in mechanical damage to the liner, however, this practice may have the adverse effect

and leave the lining system more vulnerable to deformation and hence loss of integrity. In order to fully understand the support from waste, the mechanical behaviour of the waste must be understood. Reviews of the current understanding of landfill engineering and waste mechanics^{25,26} highlight areas requiring further research. Important characteristics such as the coefficient of earth pressure at rest, and stiffness are still poorly understood with only a limited number of studies available.

3.5 Waste mass stability

Two types of waste slope failures should be considered: those involving waste mass alone and those involving the lining system and/or subgrade (Figure 8). Due to the highly heterogeneous nature of the waste mass it is unlikely that the actual strength characteristics will, or in fact could, be known. Therefore, conservative parameters should be selected that are appropriate to the waste stream, waste placement techniques and compaction practice for a particular site. Failures in the waste mass can occur due to exposed waste slope angles exceeding the shear strength of the waste body. Instability can also be induced by relic weak layers such as temporary soil cover layers, weak waste layers or leachate pressures within the waste body. Failure of a temporary waste slope was reported, due partially to sliding of the waste mass on an old cover soil layer. As the cover soil did not contain reinforcing elements (i.e. plastic) that were present in the waste body it formed a weak plane along which preferential shearing could occur.

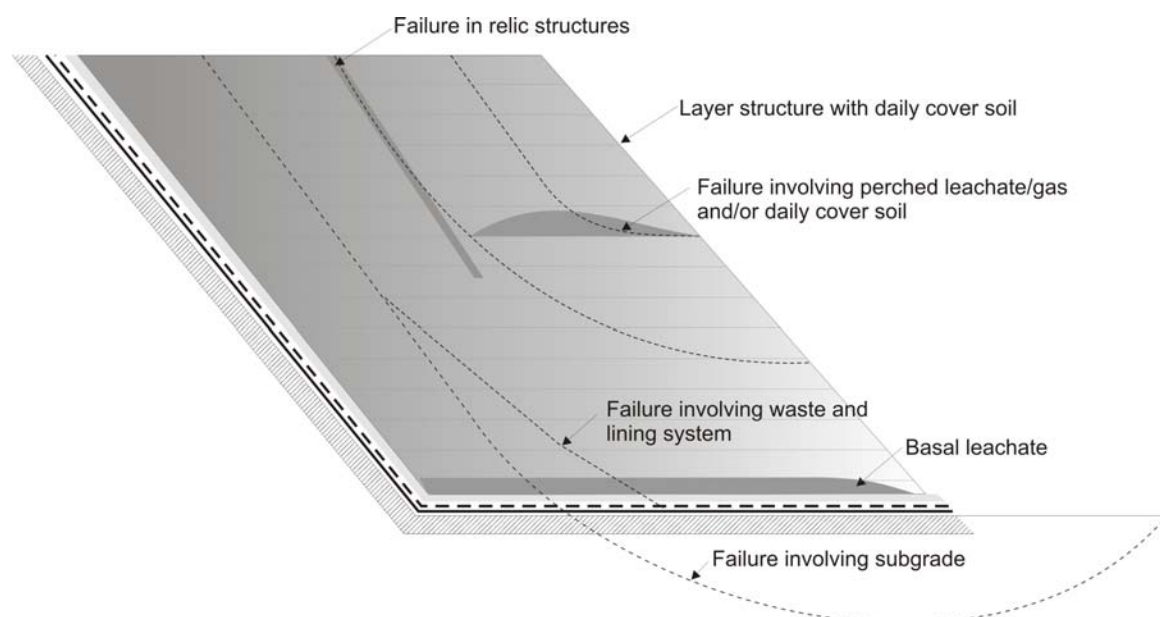


Figure 8. Waste slope failure mechanisms.

Leachate can be collected in the base, perched within the waste or can be present through the waste body (Figure 8) especially if recirculation is active. An 800,000 m³ slide of waste occurred, mainly due to injection of leachate as part of recirculation strategy in conjunction with inadequate leachate and gas collection systems²⁷. Gas and leachate pressures in the landfill may result in failures²⁸.

The designer must consider failures where the critical surface incorporates the lining system. Sliding can occur due to shearing within mineral layers, on geosynthetic interfaces, or internal failure of geosynthetic composites. Increased pore fluid pressure in liner components and along interfaces increases the likelihood of such failures, hence the leachate and groundwater conditions in the subgrade should be taken into account and controlled where required.

Translational failures of the whole waste mass along interfaces and composite failures involving both shearing of waste and interfaces must be considered. Movement along interfaces can mobilise post peak strengths and hence increase the possibility of failure. Post peak strength can be mobilised during construction and by subsequent loading as the waste is placed and settles. A landfill failure occurred due to sliding along weak planes provided by interfaces²⁹. Mobilisation of post peak interface shear strengths occurred at small displacements which, it was suggested, were likely to have been exceeded during the construction and filling phases. A translational slide was reported, where a sacrificial layer of soft clay was left in place to limit damage and desiccation and was subsequently covered by the geomembrane. This layer provided a low shear strength layer along which preferential shearing occurred.

Subgrade related failures may still occur following placement of the waste. These can be driven by the increased loading from the waste mass, particularly if the underlying subgrade experiences undrained loading. Groundwater rebound following void infill or cessation of pumping can also result in subgrade instability. A landfill failure was reported involving a low strength native soil underlying the waste³⁰. Adequate site investigation must be carried out to characterise the subgrade and likely areas of low strength leading to potential instability. While many infill (i.e. quarry type) landfills do not have steep waste slope profiles as part of the final design, it is common to form

temporary waste slopes during cell construction and staged filling, with such slopes typically have gradients up to 1 vertical: 2 horizontal. There have been a number of failures of temporary waste slopes in the UK in recent years and, therefore, it is important for designers to check the stability of all waste slopes that are formed during the filling process.

3.5 Capping lining systems

The landfill cap is exposed to the environment and the designer must consider the potential for degradation or damage to this lining system. Due to the heterogeneity of the underlying waste material differential settlements are likely under the capping system. All capping elements should be assessed in terms of overall stability and integrity (Figure 9).

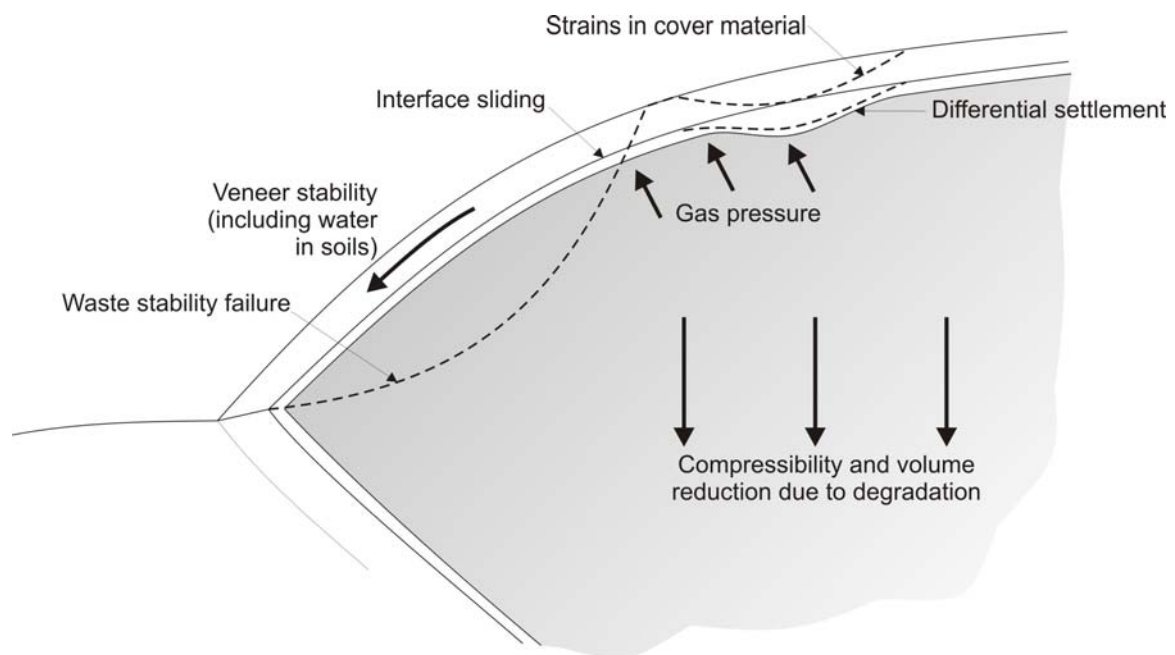


Figure 9. Capping lining system failure mechanisms.

The stability of the capping system must take into account potential failures: between and within liner elements and involving the underlying waste, restoration soils and drainage materials. Material and interface shear strengths are required for analysis as well as an understanding of the likely pore fluid conditions in the capping system. Even with inclusion of a drainage layer the cap may be susceptible to saturation in extreme rainfall conditions. Gas pressure acting on the underside of the barrier may

reduce stability beneath the barrier layers and, therefore, such permeable layers could be included below the barrier layer allowing migration of landfill gas³¹.

Figure 3 identifies the integrity mechanisms to be considered. These include slope deformations and differential deformation of the cap resulting from cavities in the underlying waste and from waste settlement. Due to the likelihood of differential settlements, materials may be selected that can accommodate greater deformations without loss of function such as LLDPE geomembranes, and high plasticity fine grained soils.

4.0 Summary

A design framework has been put forward for assessing stability and integrity of landfill lining systems. The framework covers the general design cases for each of the six elements: subgrade, basal lining system, shallow side slope lining systems, steep side slope lining system, waste slope and capping lining systems. The guidance is not intended to be prescriptive and direct engineers how to design each aspect of the lining system, but it is to highlight relevant issues. It should be noted that the factors presented here do not form an exhaustive list and all site specific issues should be considered.

Although design is very important, a good design alone will not ensure the adequate performance of a landfill lining system. Construction quality assurance (CQA) should be carried out during the construction to ensure that the construction of the lining system meets the design specification. Material properties should all be verified as deficiencies can invalidate the functionality of the entire design.

Design using generic properties derived from literature values, previous experience or index testing, can lead to potentially high risk designs, especially where the properties exhibit large inherent variability. Site specific performance testing on subgrade and lining system components reduces this risk, allowing greater confidence in designs and can result in cost savings.

In order to fully understand and assess the structural performance of landfill lining systems, they should be instrumented in order to monitor structural behaviour during

construction, waste placement and post completion. This will improve confidence in long term behaviour, lead to optimised designs and reduce risk. Integrity and deformation analysis using numerical analysis is becoming more common. However, insufficient validation of such numerical models currently exists and field instrumentation is required to address this.

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Paper 2: Interface shear strength variability and its use in reliability-based landfill stability analysis

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Full Ref:

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Abstract

Failure of modern landfills by slippage of lining materials and waste bodies is not uncommon. The majority of failures are controlled by slippage at interfaces between lining components. Information on variability of interface shear strength is required to carry out both limit equilibrium stability analysis using characteristic shear strengths and probability of failure analysis. Current practice is to carry out a limited number of site specific tests and this provides insufficient information on the variability of interface strength for design. A summary of measured strengths and an assessment of variability have been presented for seven generic interfaces common in landfill lining systems. This combines values from the international literature, an internal databases and results of repeatability testing programmes. The implications of variable shear strength are examined through probability of failure analysis of two common design cases: veneer and waste body slippage, and this adds to the small number of studies published previously. The reliability analyses show that relatively high probabilities of failure are obtained when using variability values from the literature and an internal database even when factors of safety ≥ 1.5 . The use of repeatability data produces lower probabilities for typically used factors of safety, although they are still higher than recommended target P_f values.

Notation

FS_k	Factor of safety using characteristic shear strengths
FS_{MLV}	Most likely (or traditional) value of the factor of safety
FS_i^+	Factor of safety calculated with the specific variable (i.e. shear strength) increased by one standard deviation
FS_i^-	Factor of safety calculated with the specific variable (i.e. shear strength) decreased by one standard deviation
P_f	Probability of failure
PSR	Parallel submergence ratio
V	Coefficient of variation
X_k	Characteristic value
X_m	Mean value
α	Apparent adhesion defining Coulomb failure envelope for interface shear strength (Pa)
β	Slope angle (°)
δ	Slope angle defining Coulomb failure envelope for interface shear strength (°)
σ_m	Standard deviation of measured value
σ_{MLV}	Standard deviation of FS_{MLV}
ΔFS_i	$FS_i^+ - FS_i^-$ for each variable
Subscripts	
k	Characteristic value
p	Peak
r	residual
+, -	Plus and minus one standard deviation

1. Introduction

A survey of UK failures in lined landfills reported by Jones & Dixon (2003a) showed that a significant number of slippages have occurred in the past decade. UK experience is consistent with the incidence of failures in other parts of the world that have similar landfill design and construction practices (e.g. Brink *et al.* 1999, Koerner & Soong 2000 and Mazzucato *et al.* 1999). Failures result in additional costs and at worst they can cause significant environmental damage and even loss of life.

Landfill lining systems are comprised of multiple geosynthetic and mineral layers. The interfaces between these materials can form preferential slip surfaces. The majority of failures reported in the literature are controlled by slippage at interfaces between lining components. Koerner & Soong (2000) back-analysed 10 large landfill failures and demonstrated that assessment of stability was most sensitive to shear strength parameters defined for the critical slip surface. There is growing evidence that measured values of interface shear strength show considerable variability (Criley & Saint John 1997, Koerner & Koerner 2001, Stoewahse *et al.* 2002, McCartney *et al.* 2004). This makes selection of appropriate shear strength values for use in design problematic. The relatively high rate of landfill failures has led some researchers to propose that risk assessment using probability of failure analysis can be used to quantify uncertainty in selection of appropriate interface shear strengths (Koerner & Koerner 2001, Sabatini *et al.* 2002, McCartney *et al.* 2004).

However, before design engineers can use reliability based stability analysis, guidance is required on quantifying variability of interface shear strength and on use of outputs from such analyses, in conjunction with traditional factors of safety, in the decision making process leading to design of stable slopes. This paper presents information on the variability of measured strengths obtained from a large data set for interfaces commonly encountered in landfill lining systems. Interfaces involving Geosynthetic Clay Liners (GCL) are excluded from this paper as these have been considered in detail by McCartney *et al.* (2004) using a similar approach. The use of reliability assessment in landfill stability is demonstrated through consideration of two common landfill design cases: Veneer and waste slope stability. Veneer stability has previously been used by Koerner & Koerner (2001) and McCartney *et al.* (2004) and waste slope stability by Sabatini *et al.* (2002), to demonstrate the sensitivity of landfill design to interface variability. These two design cases were selected for use in this

study in order to add to the existing published information on relationships between probability of failure and traditional factors of safety. The aim is to produce a body of information that can be used by engineers to carry out and interpret reliability based landfill designs.

1.1 Interface Shear Strength Database

The data presented in this paper has been obtained from 76 sources including journal papers, conference proceedings and internal shear testing reports. Shear strength data for seven interfaces commonly found in landfills is reported. These include both geosynthetic/geosynthetic and geosynthetic/soil interfaces. The combined database consists of 2559 shear strength values, each representing either a peak or large displacement value. The data sets for each interface have been sorted into the following three categories: values from the general literature (i.e. usually from papers reporting a small number of results for each interface), the Authors' internal database which comprises tests carried out for both design and research using common design of direct shear device and test specification, and values from repeatability studies each carried out in a single laboratory using one device and operator. While a significant proportion of this data is available in the international literature, considerable effort is required to process it into a useable format. The data is presented in this paper to aid those wishing to utilise this resource.

2. Statistical Analysis Of Interface Strength Variability

Although this paper focuses on the use of probabilistic stability assessment methods it is worth noting that information on variability of parameters required for such analyses are also needed to carry out traditional limit equilibrium stability calculations. In Eurocode 7 (1997), the characteristic value of a soil property is defined as: '*A cautious estimate of the value affecting the occurrence of the limit state*'. The characteristic value should be a cautious estimate of the mean value over the governing zone of soil (Orr & Farrell, 1999), or in this case over the area of the interface. Schneider (1997) has proposed a statistical approach for determining the characteristic value (X_k) using the mean value of the test results (X_m) and the standard deviation of the test results (σ_m):

$$X_k = X_m - 0.5\sigma_m \quad (1)$$

The approach aims to ensure in the order of 95% confidence that the real statistical mean of the parameter is superior to the selected characteristic value (X_k). In this application, it is the mean and standard deviation of interface shear strengths that are required. This is the same information that is required to undertake probability of failure analyses as discussed below.

2.1. Derived interface shear strength parameters

Interface shear strength parameters are obtained by plotting peak and large displacement, assumed to be close to residual values in most cases (Dixon & Jones 2003b), shear strengths measured in direct shear apparatus on a shear stress vs. normal stress graph. Coulomb failure criteria are defined by linear best-fit lines through sets of peak and residual data measured at normal stresses relevant to the design problem. Although linear regression provided the best fit for the interfaces reported, some geosynthetic interfaces display non-linear or bilinear strength envelopes. From the Authors' experience, it is rare for duplicate tests to be carried out at each normal stress, and hence failure envelopes are typically taken as the best fit straight line through one point at each of three or four normal stresses. This approach provides insufficient information to enable variability of measured shear strengths to be quantified. Shear strength envelopes are defined by pairs of apparent adhesion (α) and interface friction angle (δ) parameters. While it is common practice in many applications involving soil to ignore apparent cohesion values in design, this approach is not recommended for geosynthetic interfaces. Apparent adhesion values can be considered in design of structures that incorporate interfaces with a true strength at zero normal stress (e.g. *Velcro*TM type affect between non-woven needle punched geotextile and textured geomembranes). Apparent adhesion can also be used to define a failure envelope over a range of normal stresses (i.e. assuming a linear failure envelope) or to define a best fit straight line through limited variable test data. In these specific cases it would be over conservative to assume $\alpha = 0$, especially for design cases with low normal stresses (e.g. design of cap systems). Negative α can also be produced by best fit lines through limited test data. If negative α are ignored this will result in an over estimate of shear strength and hence potentially unsafe designs. Negative α values are produced by best fit lines through a number of the data sets included in this paper, and these demonstrate limitation of data sets in terms of number of points and their distribution.

As the quantification of interface shear strength requires two parameters (α and δ), variability of measured shear strengths requires consideration of linked pairs of these parameters. Dixon

et al. (2002) proposed an approach based on calculating the variability of measured shear strengths for each normal stress and using this data to derive the appropriate shear strength parameters for use in design. For example, Figure 1 shows how characteristic values can be obtained for use in a limit equilibrium analysis.

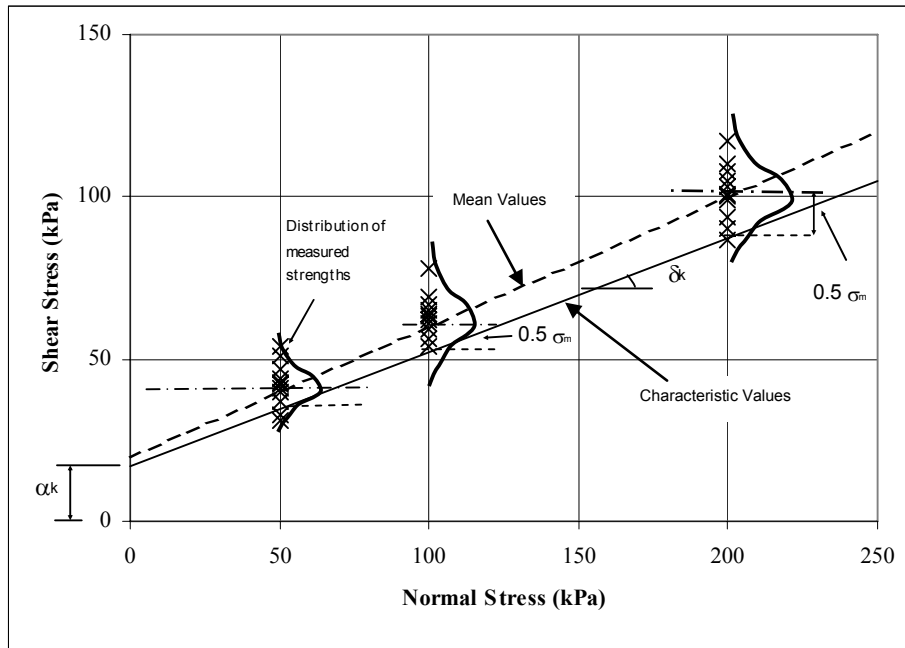


Figure 1. Derivation of interface shear strength parameters from measured shear strengths

2.2. Statistical data for measured interface shear strengths

Two approaches are available for obtaining information on the variability of interface shear strength for use in assessment of stability. The preferred approach is to undertake a sufficient number of site specific tests at each normal stress to enable statistical analysis of the measured strengths. This will allow the mean (X_m) and standard deviation (σ_m) of measured strengths to be calculated for each stress level. As discussed above, this approach is based on assessing the variability of measured shear strengths and not the derived shear strength parameters. It is believed that at present this approach is considered too costly (both in time and money) by the majority of designers.

A second approach is to carry out a limited number of tests to obtain site specific strength values and to obtain information from the literature on possible variability for that specific type of interface. However, a limitation of this approach is that there is no information available to indicate whether the measured site specific strengths are representative of mean

values. If in comparison to the estimated mean values (i.e. using data from previous tests on similar materials) the measured strengths are considered to be high, or there is limited experience of testing the interface, then further tests should be conducted and the first approach described above must be used. Design based wholly on literature values should not be attempted

Where there is limited data available, an alternative approach is to calculate standard deviation using the three-sigma rule, which uses the fact that 99.73% of all values of a normally distributed parameter fall within three standard deviations of the average (Duncan 2000). The three-sigma rule has been used by Sabatini *et al.* (2002) to quantify the variability of geosynthetic/soil interface strength. In this paper, variability of interface strengths have been expressed as a function of the mean using coefficient of variation (V) defined as:

$$V = \sigma_m / X_m \quad (2)$$

3. Variability of Measured Interface Shear Strength

Measured interface properties are influenced by inherent variability of soil and geosynthetics, and measurement errors. Measurement errors are the sum of systematic bias in average property measurements and random errors. It is not possible to measure random errors because repeatability tests use disturbed/modified or new materials and hence also include material variability. Systematic testing bias can be estimated by carrying out series of repeatability tests in different laboratories (i.e. using different equipment and personnel) on materials from the same source. This can only identify gross bias because material variability, although minimised, is still present because new samples are used in each test. A detailed discussion of factors causing variability of measured interface shear strength is provided by Stoewahse *et al.* (2002).

3.1. Published information on variability of interface shear strength

The international literature contains many papers that report measured shear strengths for geosynthetic/geosynthetic and geosynthetic/soil interfaces. The best controlled studies are those in which materials from one source (i.e. roll of geosynthetic and bulk sample of soil) have been used in direct shear tests repeated on one device, using the same test standard and carried out by the same personnel. Such studies have been reported by Dixon *et al.* (2000) for

both smooth and textured geomembranes vs. non-woven geotextile tested at low normal stresses (i.e. appropriate for cap design) and Criley & Saint John (1997) for both fine and coarse soils vs. textured geomembrane.

A number of studies are reported in which materials from one source have been tested in direct shear tests conducted at different laboratories using a common test procedure. These include the following inter-laboratory test programmes: 1995 and 1996 German tests carried out to support development of a general direct shear test standard (Blümel & Stoewahse 1998); tests carried out in seven laboratories across Europe (Gourc and Lalarakotoson, 1997) to support developments of the EC direct shear interface test standard BS EN ISO 12957-1 (2005); and North American inter-laboratory comparison tests reported by the Geosynthetics Research Institute (Koerner & Koerner 2001). Data sets for common interfaces have previously been published based on a summary of values reported in the literature. Jones & Dixon (1998) presented data in the form of summary plots of measured peak and large displacement shear strength vs. normal stress for 15 interfaces. It was proposed that these plots could be used to obtain parameters for use in preliminary design and to help designers assess site specific test results. However, there is evidence that some designers are using mean values from the Jones & Dixon (1998) literature summary in lieu of site specific tests.

The Jones & Dixon (1998) data sets based on the international literature have been updated and combined with the other data sources listed above (excluding the Koerner & Koerner 2001 data) and also with an internal databases compiled by the Authors. Table 1 provides a summary of the 7 interfaces for which data is presented, the number of test results in each data set, the range of normal stresses and the type of data set. It was not appropriate to subdivide the interfaces further (i.e. into different types of texturing or soil types) as this would have produced data sets too small to allow meaningful statistical analysis. This may become possible in the future as additional interface strengths are published.

Table 1 Summary of geosynthetic/soil interfaces and datasets on measured shear strengths

Interface type	Data set	No. of points (peak, residual)	Combined data mean best fit line, peak (α_p , δ_p)	Combined data mean best fit line, large displacement (α_r , δ_r)	Combined data standard deviation best fit line peak (slope, y intercept)	Combined data standard deviation best fit line large disp. (slope, y intercept)	Range of normal stress (kPa)
Smooth HDPE GM ¹ vs. NW GT ²	Internal Database Literature	52, 52 45, 30	-0.7, 10.0 (Figure 2a)	0.8, 6.1 (Figure 2b)	0.038(σ_n), -0.1	0.026(σ_n), 0.2	3 - 525
Textured HDPE GM vs. NW GT	Internal Database Literature Inter-laboratory Comparison	116, 130 16, 14 206, 0	8.1, 25.9 (Figure 3a)	6.0, 12.4 (Figure 3b)	0.0653(σ_n), 4.2	0.033(σ_n), 3.0	12 - 383
Smooth HDPE GM vs. Coarse Soil	Internal Database Literature	15, 15 133, 45	-7.3, 25.2 (Figure 4a)	0.8, 17.8 (Figure 4b)	0.069(σ_n), 1.9	0.074(σ_n), 1.0	10 - 1794
Textured HDPE GM vs. Coarse Soil	Internal Database Literature Criley & Saint John (1997)	30, 29 27, 15 122, 122	8.4, 33.1 (Figure 5a)	9.8, 30.5 (Figure 5b)	0.033(σ_n), 6.4	0.074(σ_n), 2.8	5 - 720
NW GT vs. Coarse Soil	Internal Database Literature Inter-laboratory Comparison	36, 36 206, 78 286, 0	3.6, 35.0 (Figure 6a)	4.2, 34.2 (Figure 6b)	0.155(σ_n), 5.6	0.136(σ_n), -0.7	5 - 575
Smooth HDPE GM vs. Fine Soil	Internal Database Literature	9, 9 143, 187	- ³ (Figure 7a)	- (Figure 7b)	-	-	5 - 718
Textured HDPE GM vs. Fine Soil	Internal Database Literature Criley & Saint John (1997)	41, 41 53, 38 91, 91	- (Figure 8a)	- (Figure 8b)	-	-	7 - 958

¹ GM = geomembrane; ² NW GT = non-woven geotextile; ³ Summary data not given due to range of test conditions used

Smooth and textured geomembrane samples are made from high density polyethylene (HDPE) or linear low density polyethylene (LLDPE). Texturing type varies, with impinged and blown film methods of texturing being the most common. Geotextile samples are all non-woven needle punched polypropylene. Soils used in tests have been categorised as either fine (primarily silt and clay) or coarse grained (primarily sand and gravel) materials. It is not possible to use a more rigorous classification system because of the lack of information on soil materials given in the literature.

Figures 2 to 8 are summary plots of measured peak (a) and large displacement (b) shear strengths for the selected interfaces. Main data sets are identified on each plot. Best fit trend lines are shown based on all the data points and the equations for these lines are summarised in Table 1. All the data points are shown in Figures 2 to 8 to allow the reader to independently assess the groupings/coverage of data with respect to normal stress. This information can not be obtained from Table 1. It is important that any potential users of the best fit trend lines fully appreciate the quality of the data sets from which they are derived. For example, in Figure 5 best fit lines are provided through all the data and also the literature data excluding the Criley and Saint John (1997) as it controls the location of the best fit line. Figures 9 to 13 provide information on the variability of measured strengths for the selected interfaces via plots of coefficient of variation vs. normal stress (a) and standard deviation vs. normal stress (b). Information on peak and large displacement best fit linear trend lines through the combined data are also shown and these are summarised in Table 1.

3.2. Distribution of measured shear strengths with normal stress

The data presented in Figures 2 to 8 shows a large variability in the number of tests and their distribution across the range of normal stresses. This is to be expected as the data sets are, in the main, compilations of tests conducted for different and specific purposes. However, despite this, there is sufficient data to demonstrate general trends. It was anticipated that data sets would show ranges of variability dependent upon the number of variables involved in testing (e.g. test equipment, personnel, test specification and material). For example, literature data sets would be expected to show greater variability than inter-laboratory data sets for material from one source. However, the data does not show this trend (Figure 3 and 6). Apart from the repeatability results, the other data sets for a given interface (both peak and residual) define comparable ranges of shear strength with respect to normal stress. This is surprising

because it indicates that differences in measured strengths resulting from material variability (e.g. from type of texturing, type of soil/conditions etc) represented in the literature and internal data bases are of the same order as that resulting from carrying out tests on the same materials at different laboratories. It can be concluded that for a given generic type of interface, test conditions have the most significant influence on observed variability of measured shear strengths.

The only interfaces that are not consistent with this trend are those involving fine grained soils. Figures 7 and 8 show large ranges of measured peak and residual shear strengths for a given normal stress. This is due to the poor control and reporting of test conditions with respect to the fine soil materials. The summary plots include drained, undrained and partially drained shear tests due to a range of consolidation conditions and shear rates being employed. Test conditions are seldom reported with sufficient detail to allow interpretation of the pore water pressure conditions at the interface. The data is only included in this paper to demonstrate the wide range of values and hence to highlight the inappropriateness of using literature values for such interfaces in design. Note that no trend lines are shown.

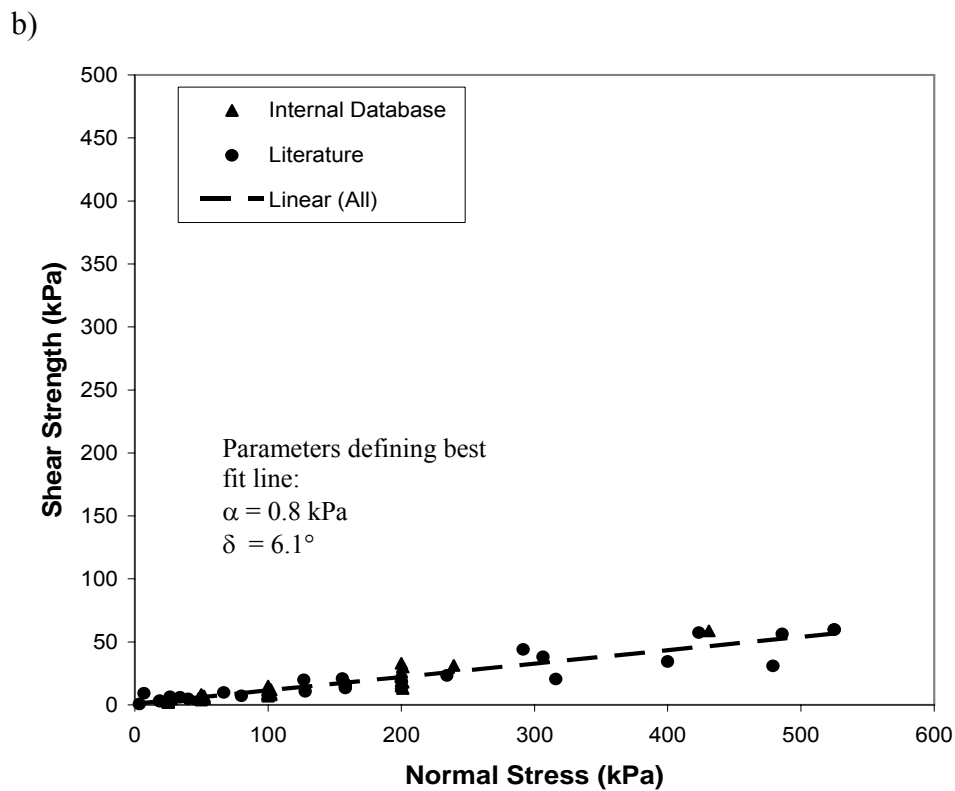
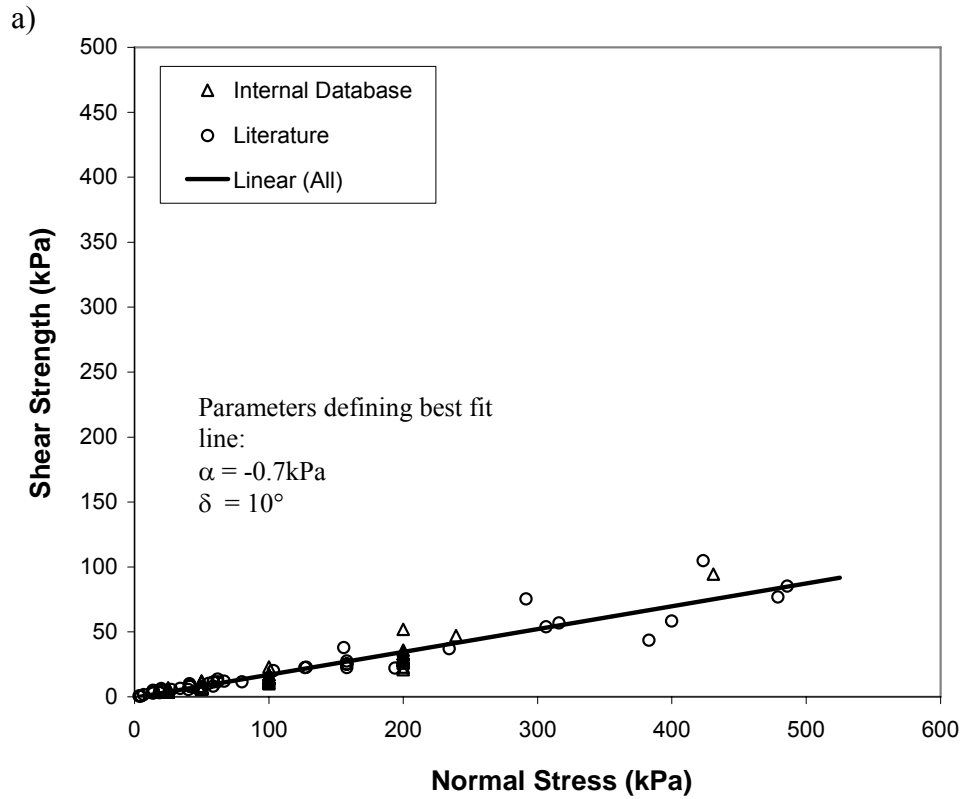
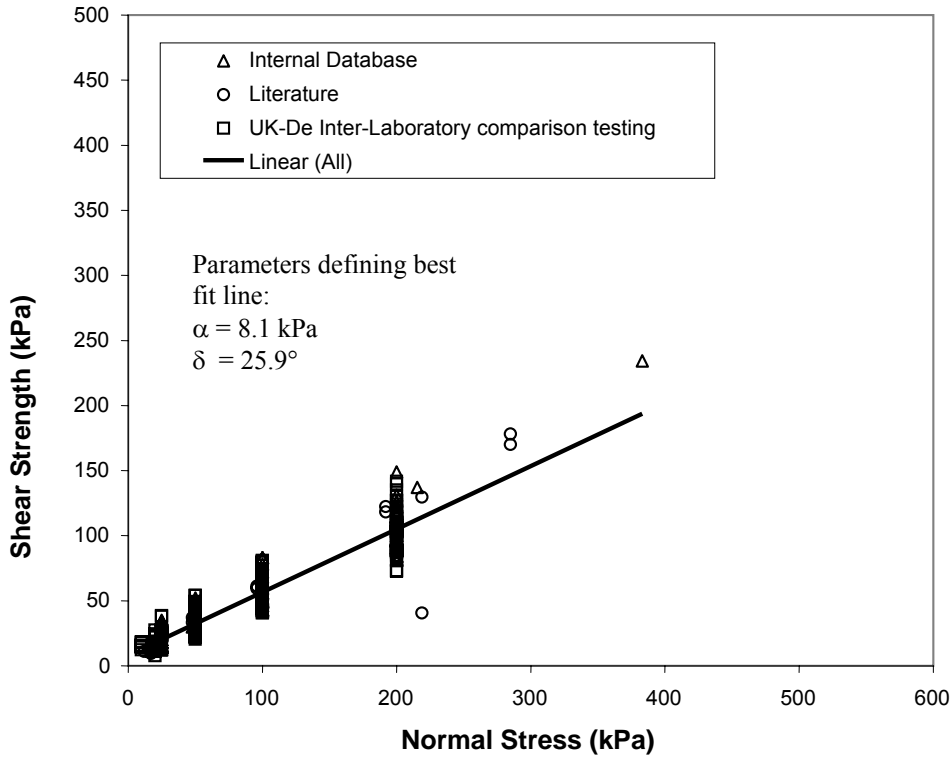


Figure 2. Shear strength vs. normal stress for smooth HDPE geomembrane vs. non-woven geotextile from internal database and literature a) peak, and b) large displacement.

a)



b)

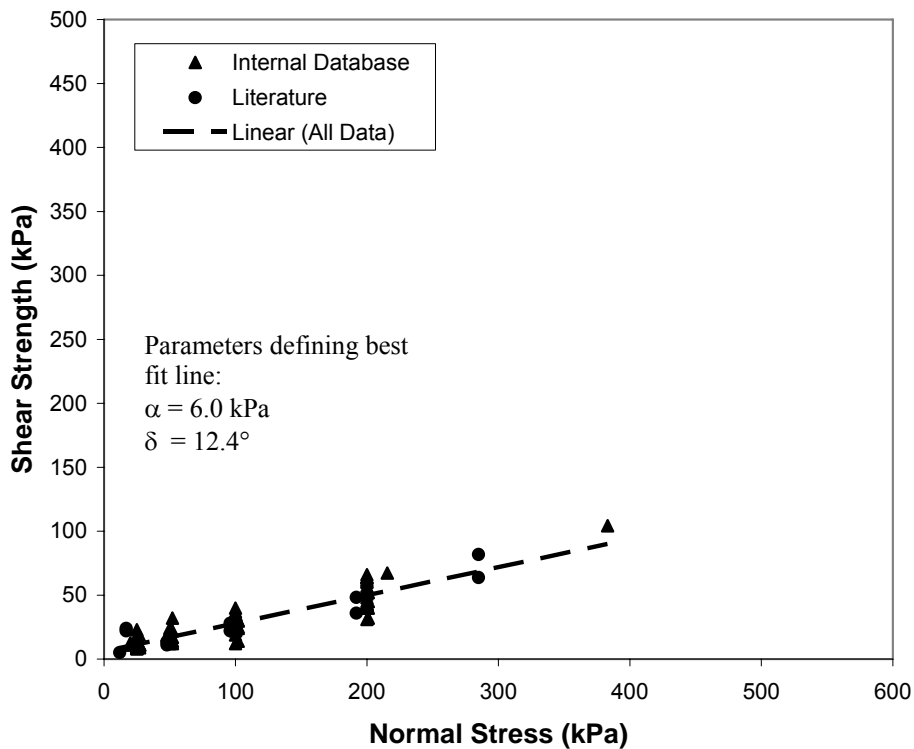
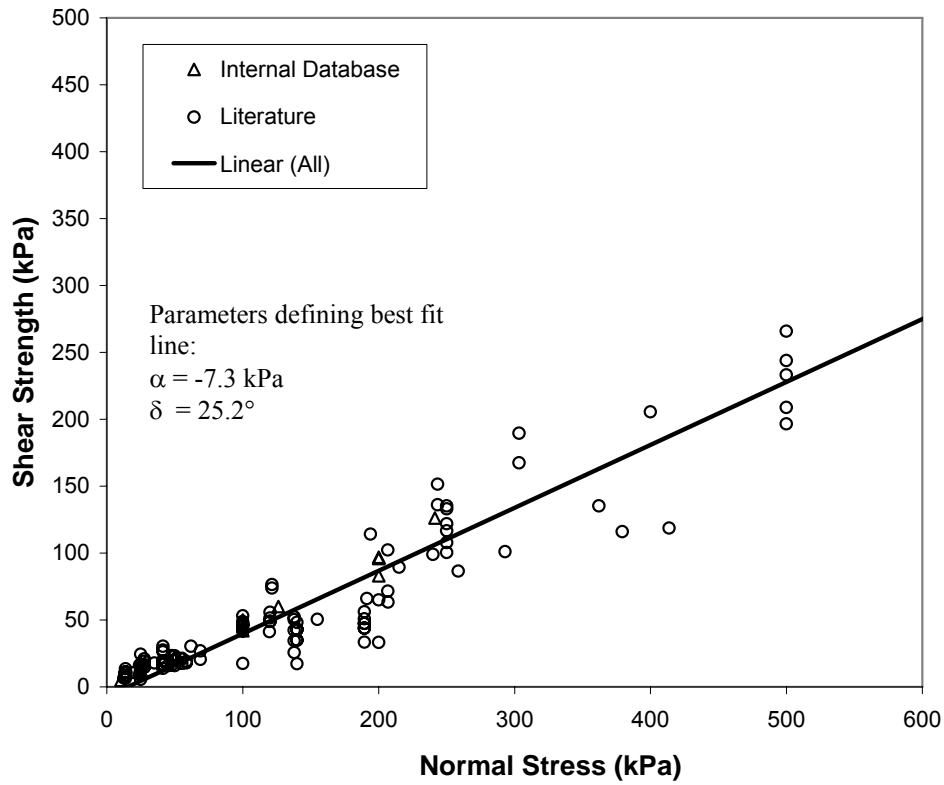


Figure 3. Shear strength vs. normal stress for textured HDPE geomembrane vs. non-woven geotextile from internal database, inter-laboratory comparison testing and literature a) peak, and b) large displacement.

a)



b)

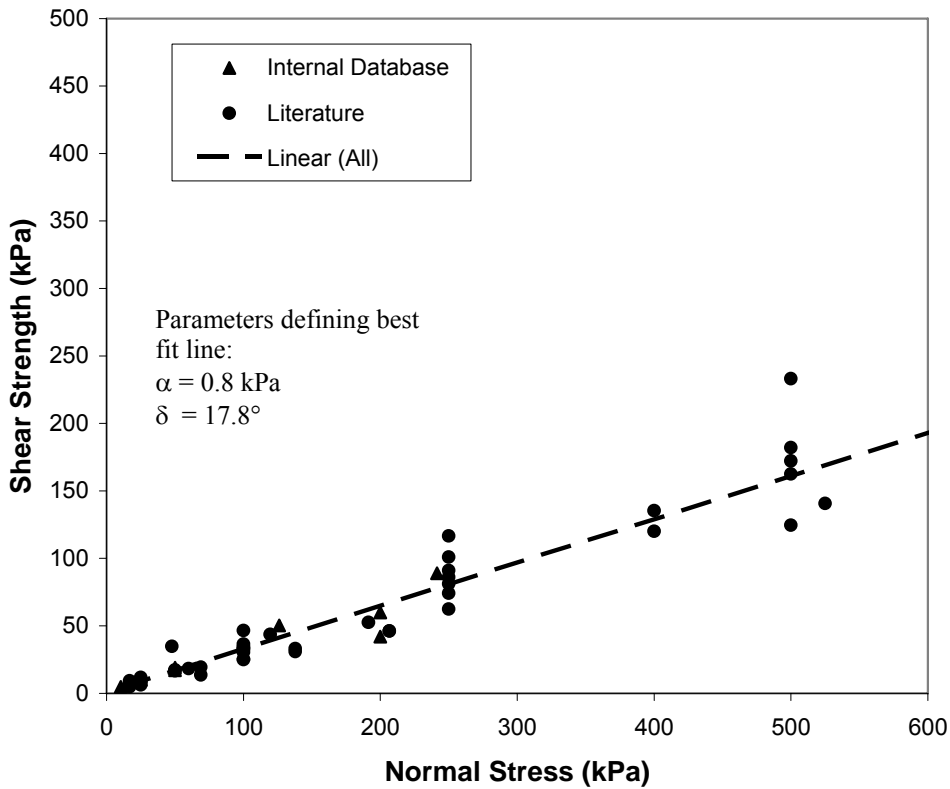


Figure 4. Shear strength vs. normal stress for smooth HDPE geomembrane vs. coarse soil from literature and internal database a) peak, and b) large displacement.

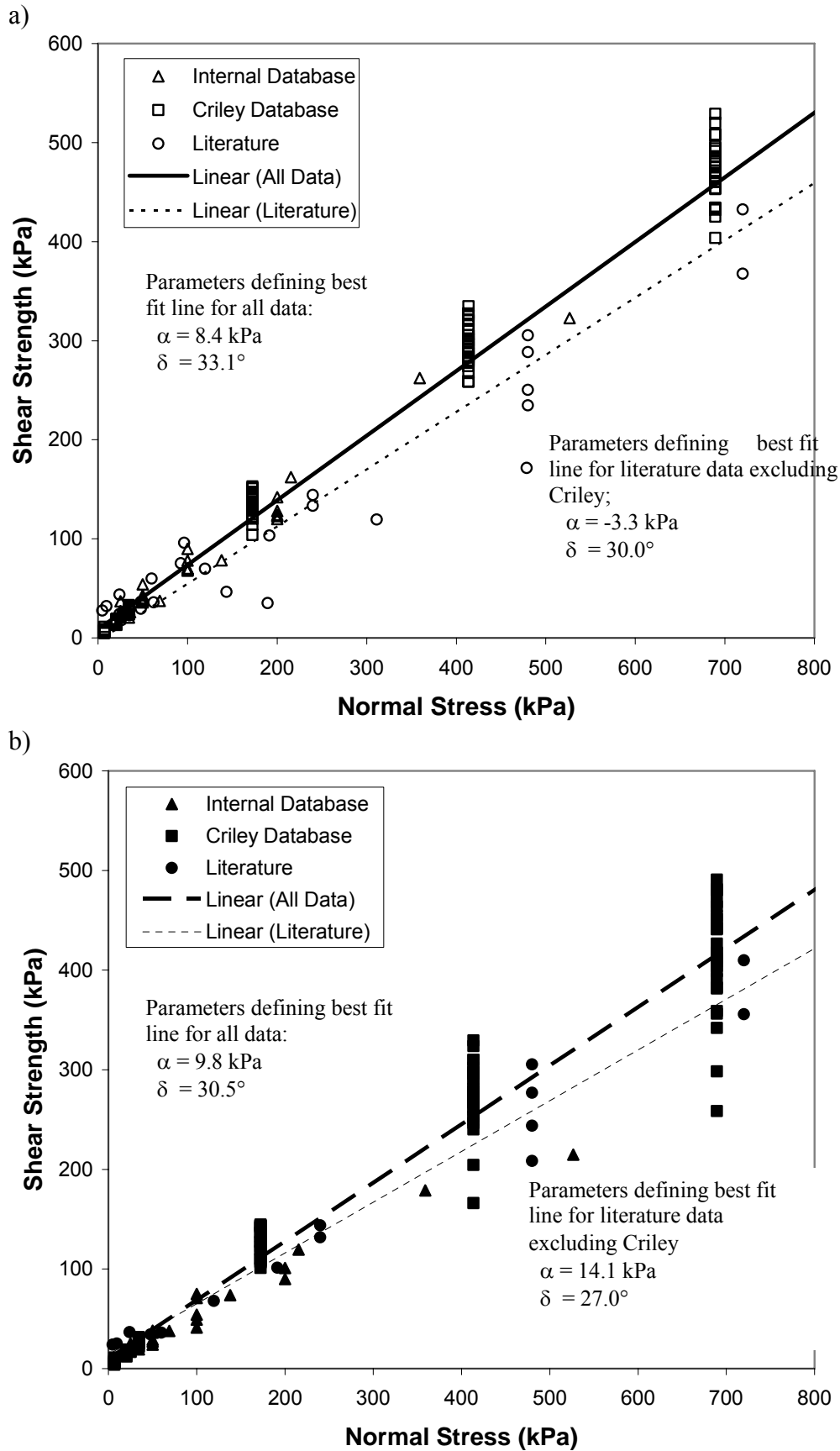
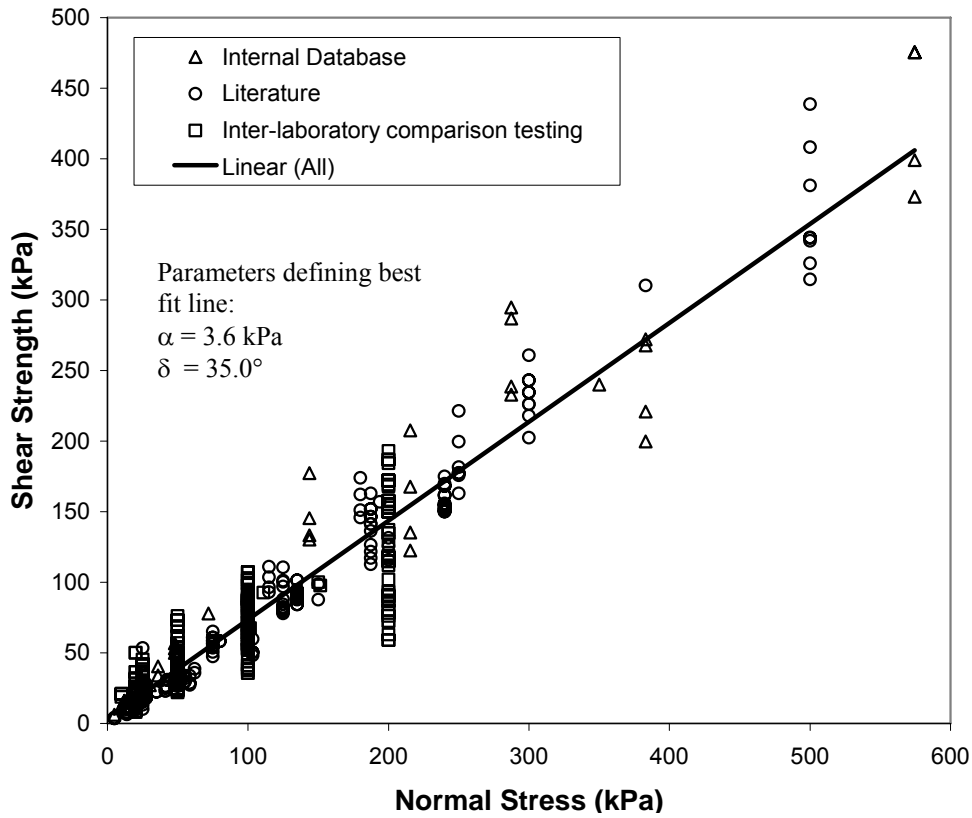


Figure 5. Shear strength vs. normal stress for textured HDPE geomembrane vs. coarse soil from literature, internal database, and Criley & Saint John (1997) repeatability results a) peak, and b) large displacement.

a)



b)

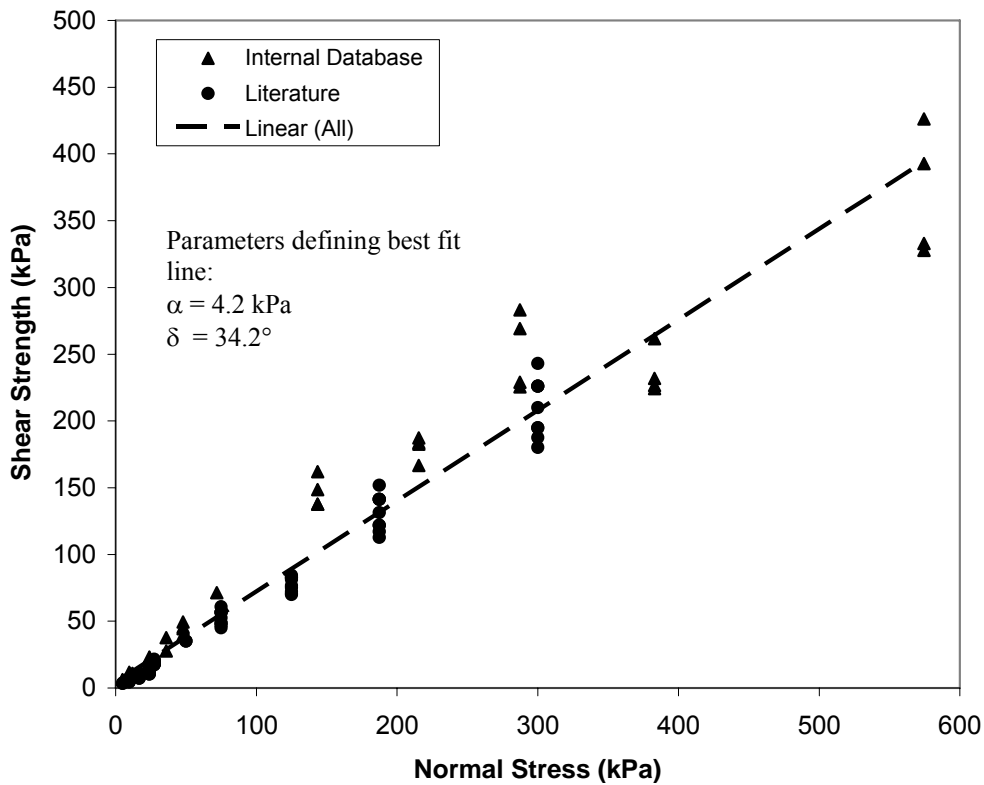


Figure 6. Shear strength vs. normal stress for non-woven geotextile vs. coarse soil from literature, internal database and inter-laboratory comparison testing a) peak, and b) large displacement.

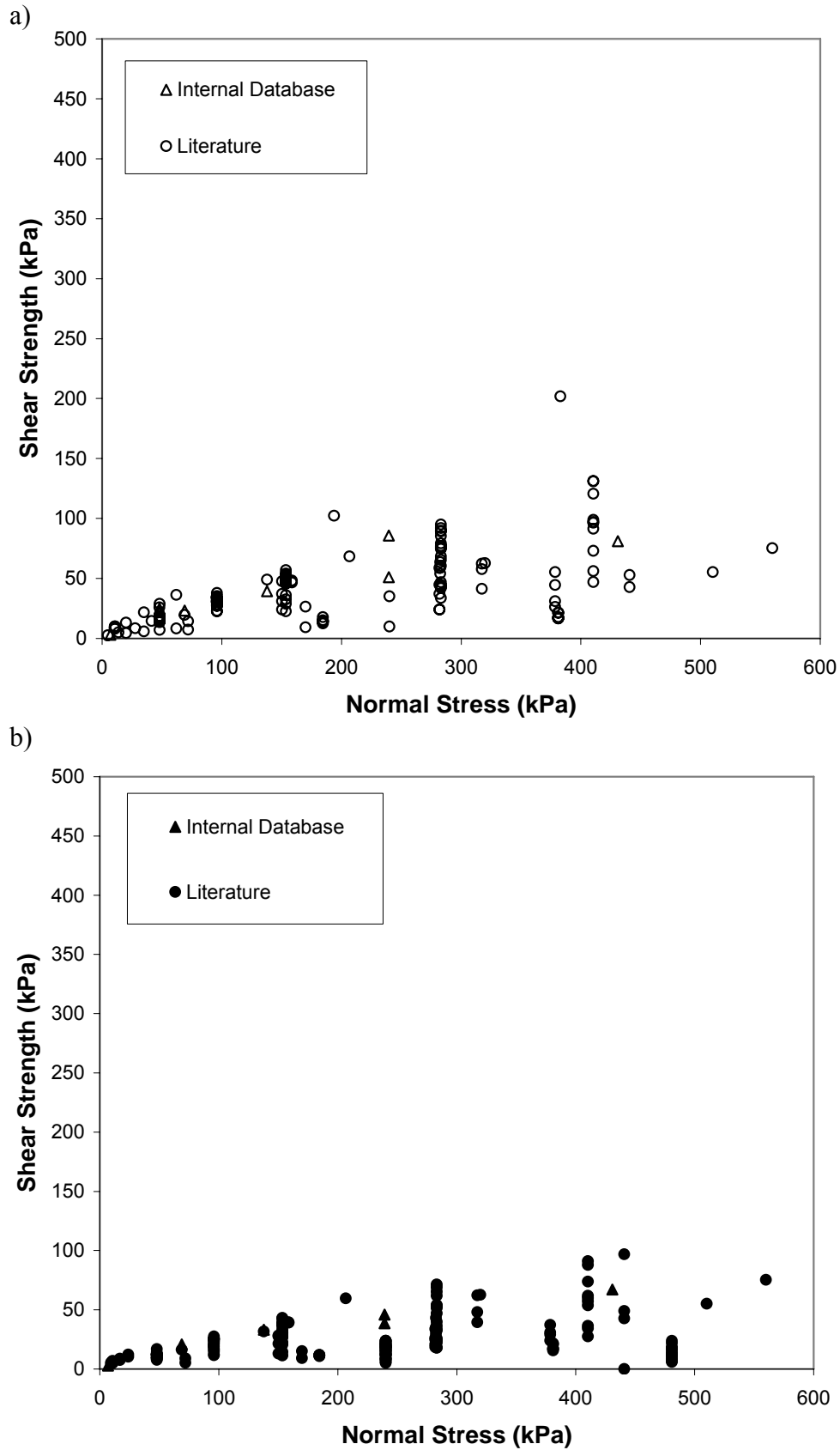
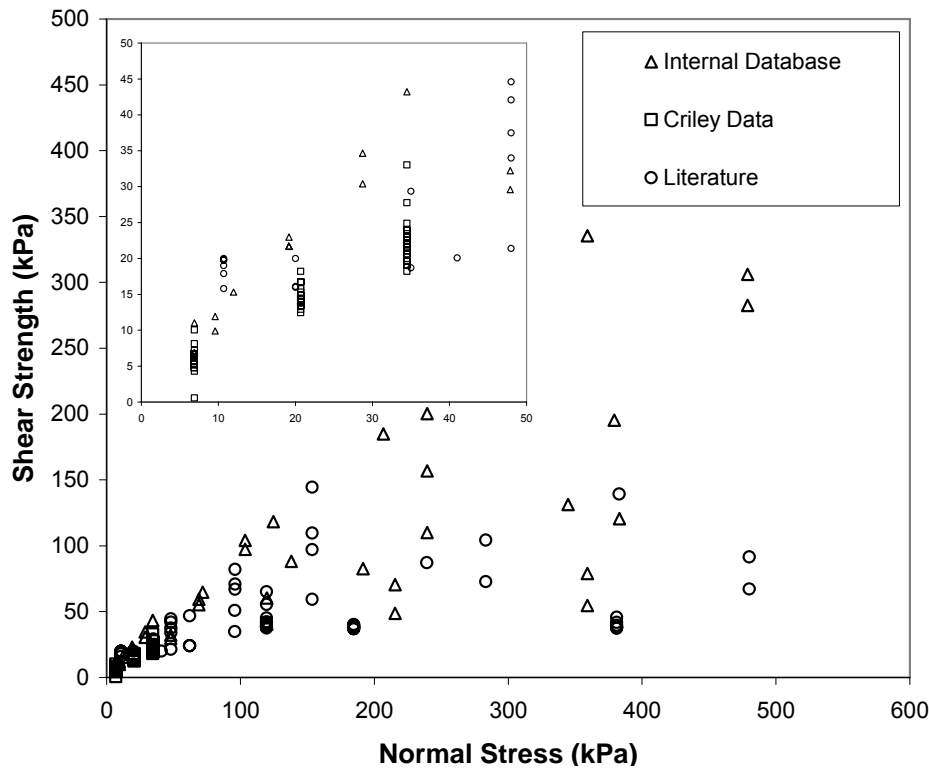


Figure 7. Shear strength vs. normal stress for smooth HDPE geomembrane vs. fine soil from literature, and internal database a) peak, and b) large displacement.

a)



b)

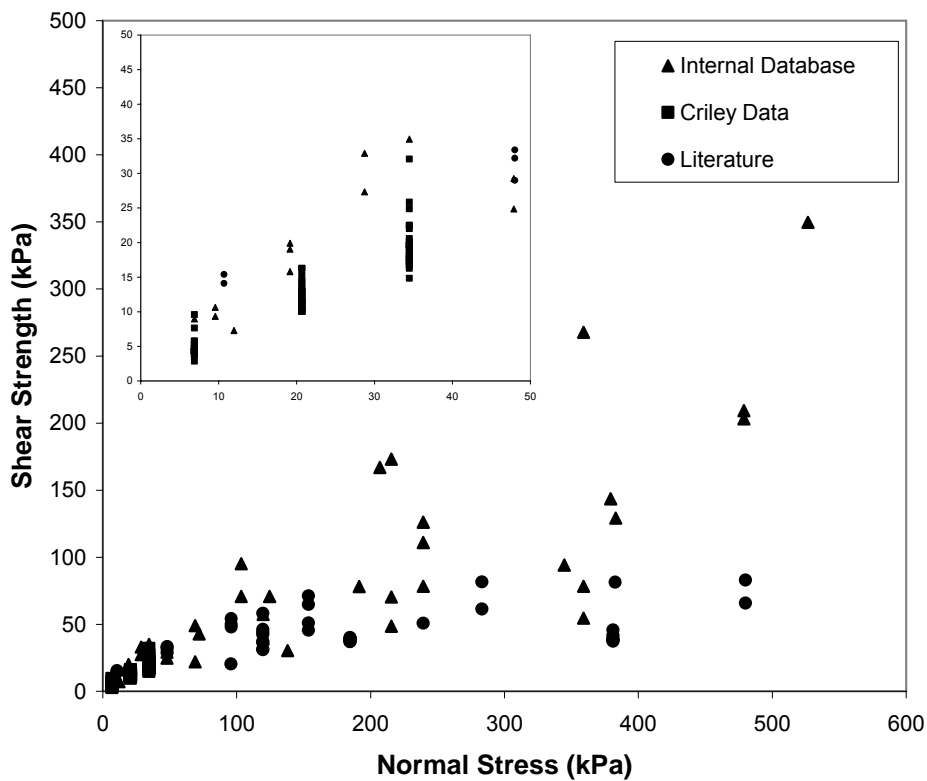


Figure 8. Shear strength vs. normal stress for textured HDPE geomembrane vs. fine soil from literature, internal database and Criley & Saint John (1997) repeatability results a) peak, and b) large displacement.

3.3. Trends in variability of interface strength

Figures 9 to 13 confirm that the variability of the different data sets (literature, internal and inter-laboratory) for a given generic interface is essentially the same, although there are differences between some data sets as shown by Figures 11 and 13. The reason for this is currently unclear but may be a function of the small size of some data sets. Best fit lines for combined data sets can be used to define the relationship between standard deviation and normal stress for each interface type. A linear trend has been found to best fit the presented data. The parameters defining the relationship between standard variation and normal stress for each interface can then be used to calculate shear strength parameters using equation (1) as shown in Figure 1.

The summary standard deviations are conservative values because they include different materials, test equipment and test specifications and hence would be expected to give upper bound values. The small number of repeatability test data sets, for example the Criley & Saint John (1997) data, give smaller variability as shown in Figure 12. These values of variability are more likely to be representative of those that would be achieved in site specific repeatability tests. Unfortunately, there are only a small number of such investigations reported in the literature, for a few interfaces, and therefore currently there is insufficient information to allow guidance values to be given.

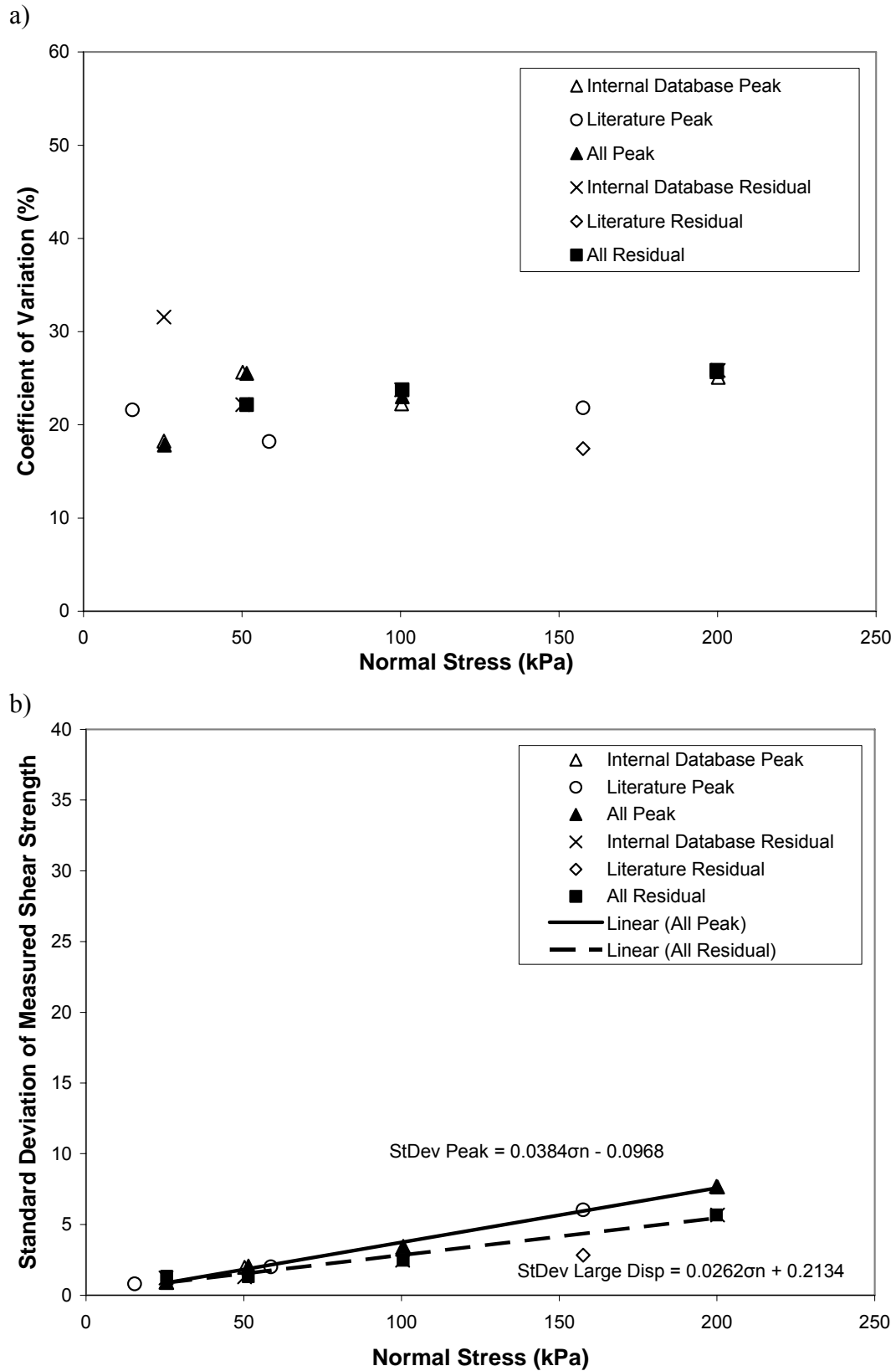


Figure 9. Smooth HDPE geomembrane vs. non-woven geotextile from internal database, literature, and combined for peak and large displacement a) coefficient of variation vs. normal stress, and b) standard deviation of measured shear strength vs. normal stress.

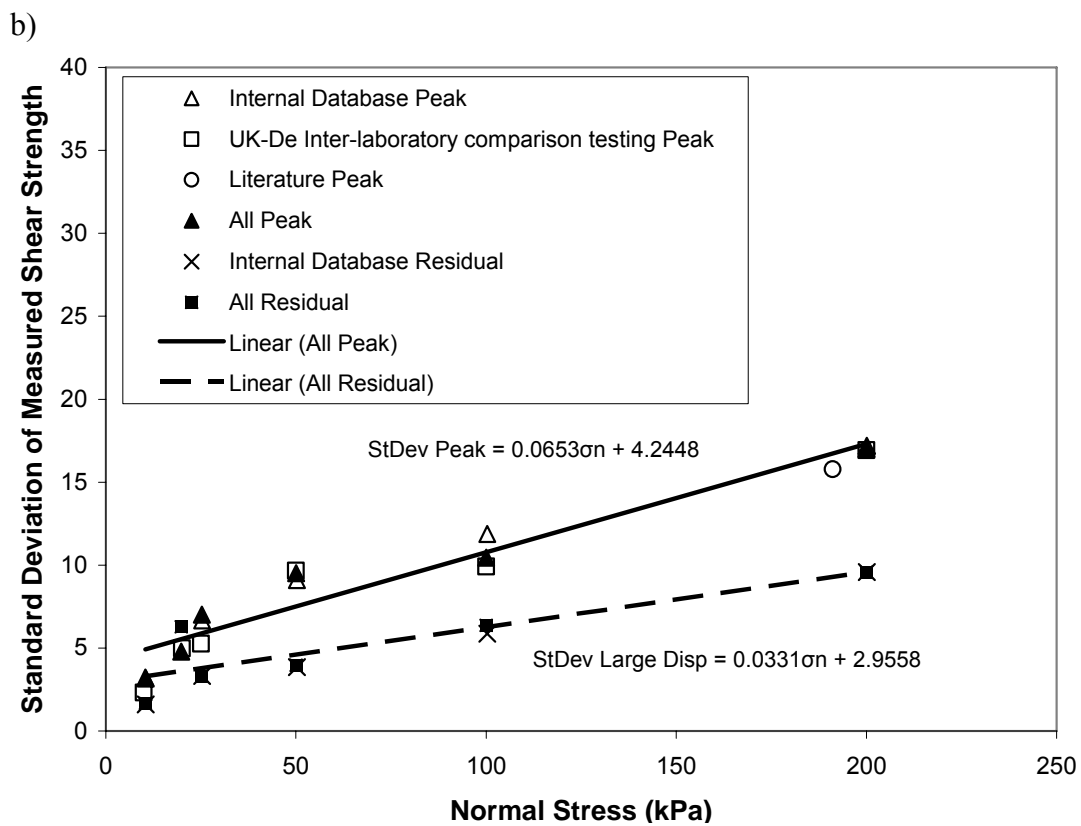
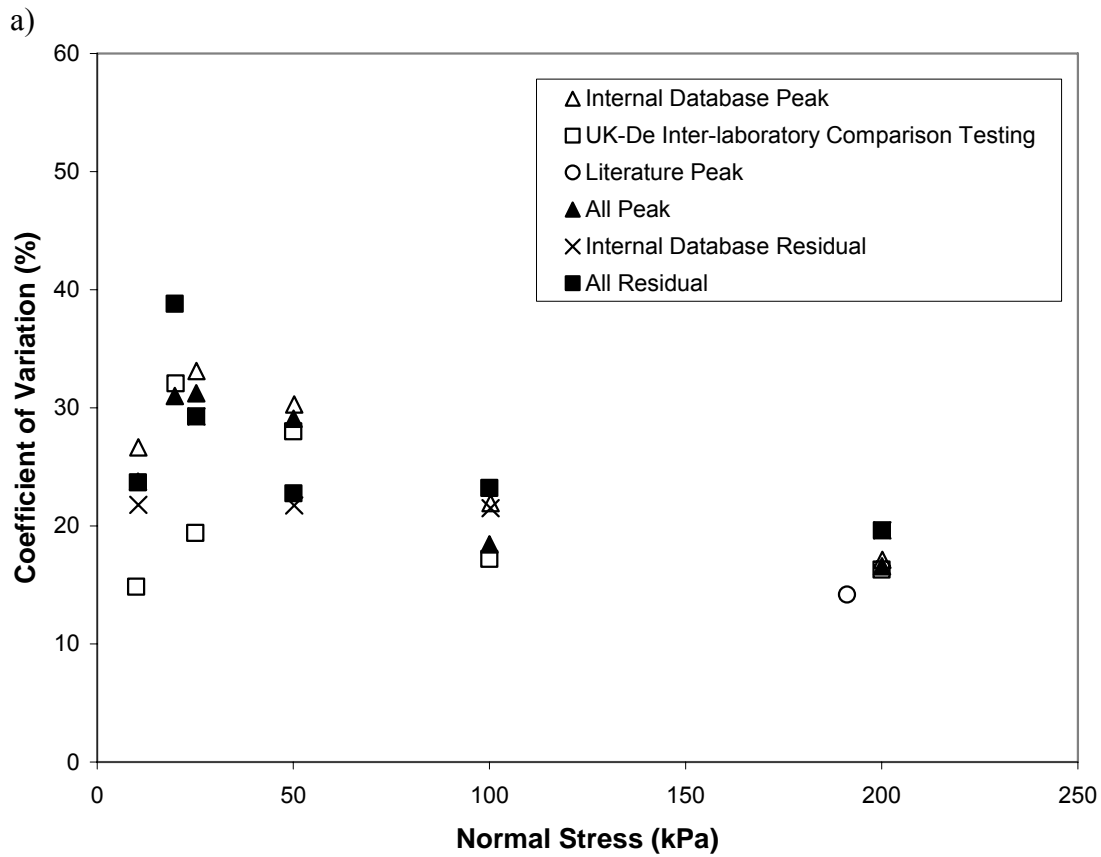


Figure 10. Textured HDPE geomembrane vs. non-woven geotextile from internal database, literature, inter-laboratory comparison tests and combined for peak and large displacement a) coefficient of variation vs. normal stress, and b) standard deviation of measured shear strength vs. normal stress.

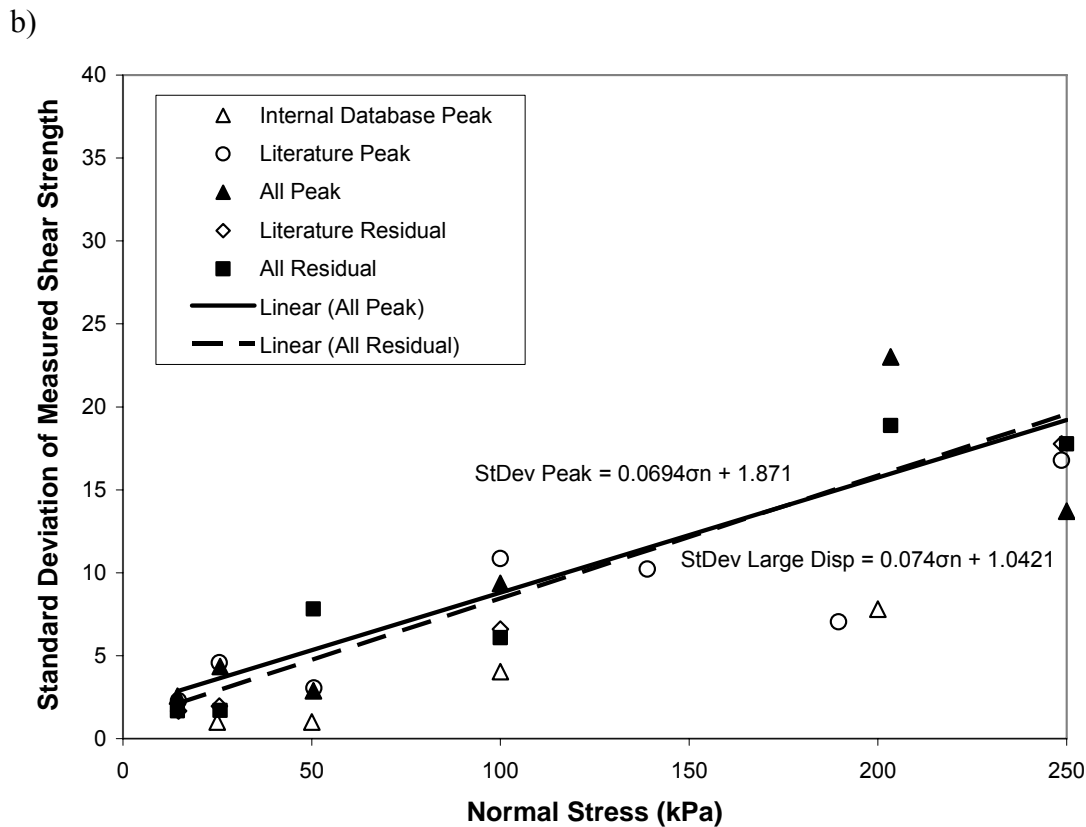
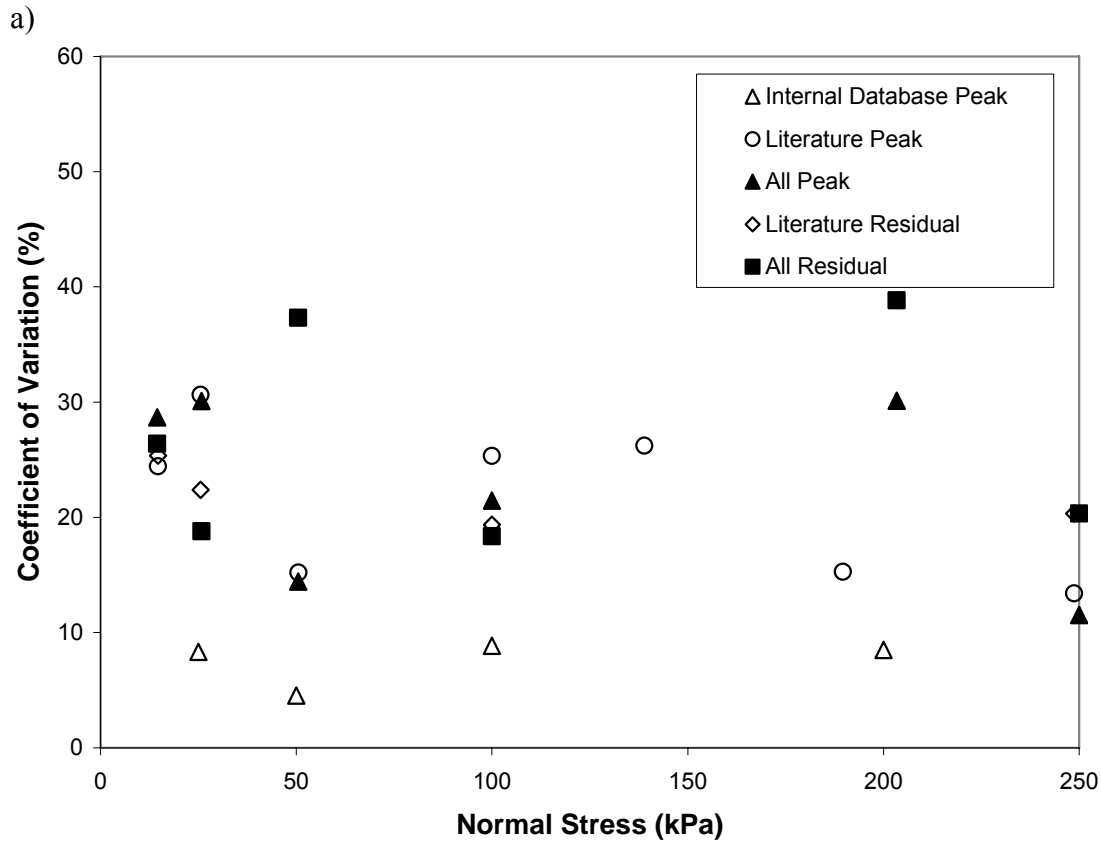


Figure 11 Smooth HDPE geomembrane vs. coarse soil from internal data base, literature and combined for peak and large displacement a) coefficient of variation vs. normal stress, and b) standard deviation of measured shear strength vs. normal stress.

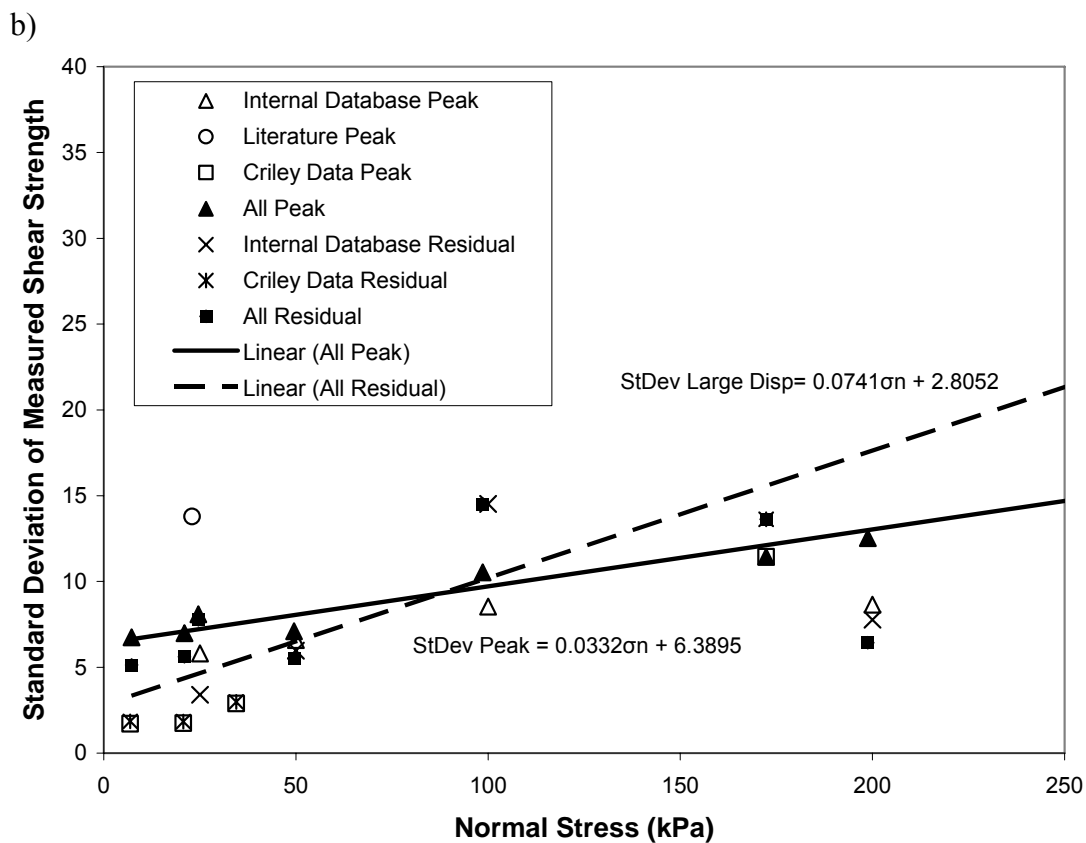
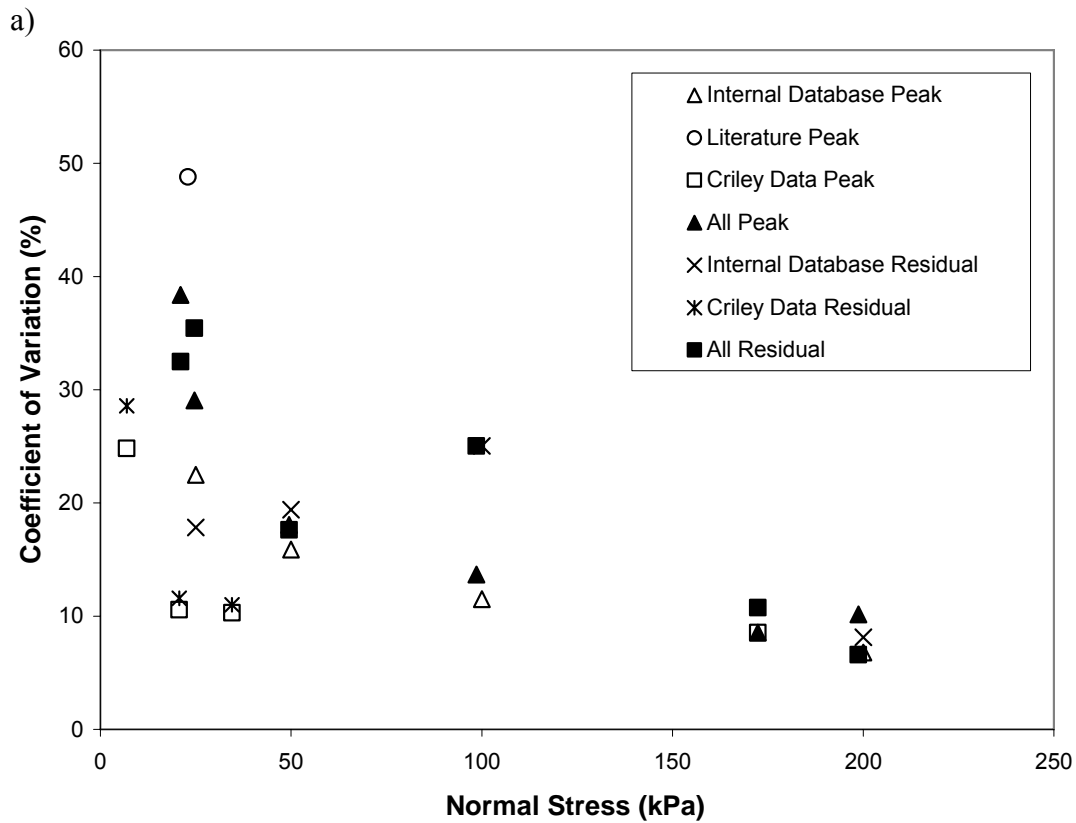


Figure 12 Textured HDPE geomembrane vs. coarse soil from internal database, literature, Criley & Saint John (1997) repeatability results and combined for peak and large displacement a) coefficient of variation vs. normal stress, and b) standard deviation of measured shear strength vs. normal stress.

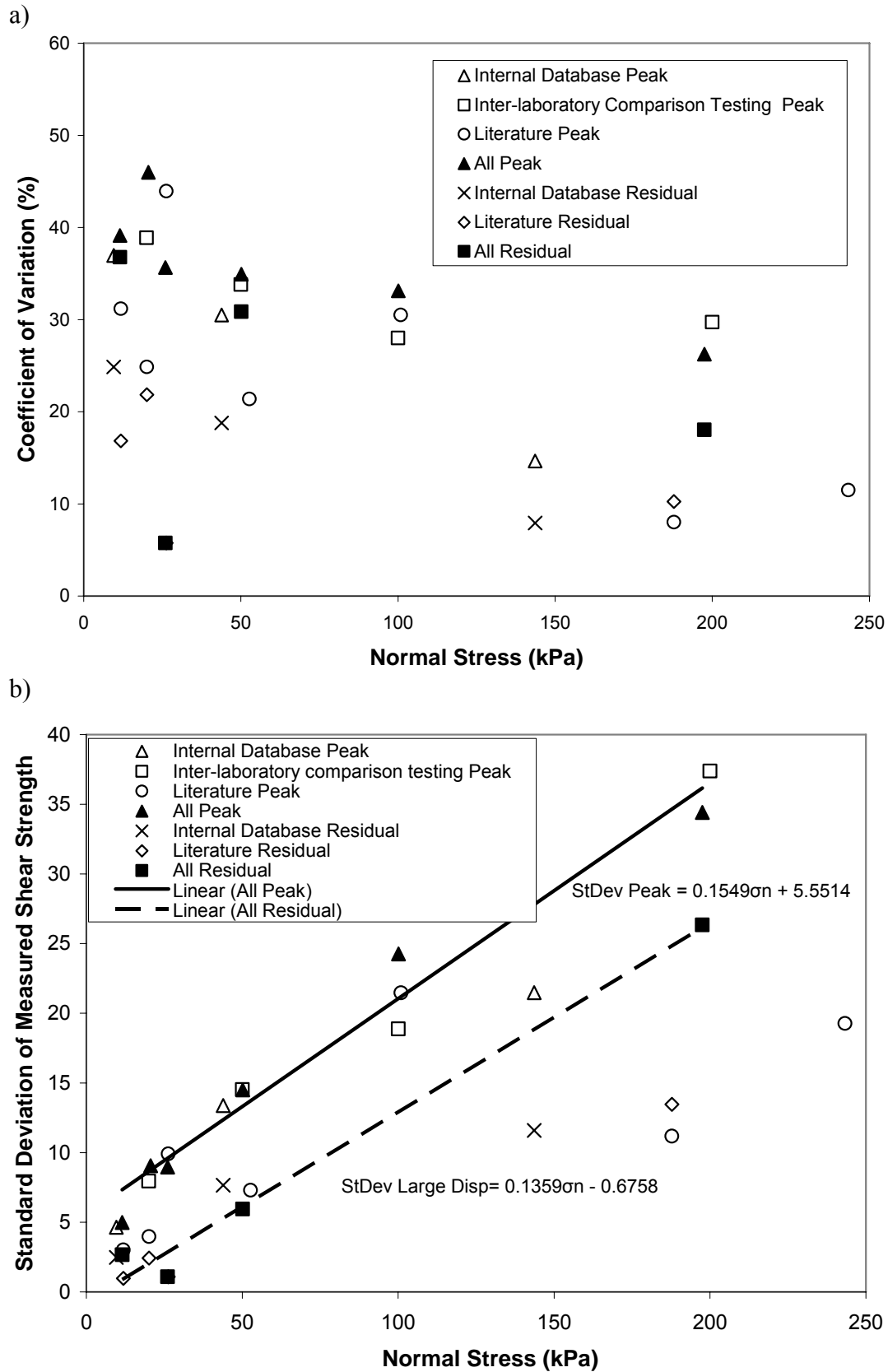


Figure 13. Non-woven geotextile vs. coarse soil from literature, internal database, inter-laboratory comparison testing and combined for peak and large displacement a) coefficient of variation vs. normal stress, and b) Standard deviation of measured shear strength vs. normal stress.

4. Probability Of Failure Stability Analysis

4.1. Analysis method for probability of failure

Risk assessment of landfill stability using probability of failure (P_f) has been discussed by Koerner & Koerner (2001), Sabatini *et al.* (2002) and McCartney *et al.* (2004). All employed the first-order, second moment reliability-based methodology (Duncan 2000). In all three cases, use of the reliability method was made possible by access to databases providing information on variability of measured interface strengths. A brief description of the methodology proposed by Duncan (2000), and used in this study, is presented in Appendix 1. As outlined in the introduction, the same landfill design cases as used by the above authors (i.e. veneer and waste slope stability) have been used in this study. This is essential if a sufficient body of experience is to be gained to guide designers on both selection of interface strength variability inputs and interpretation of probability of failure outputs from such studies.

4.2. Veneer stability

A common design case in landfill engineering is stability assessment for thin veneers of soil above one or more geosynthetic layers. These conditions are encountered during construction of side slope lining systems (i.e. stability assessment of drainage layers prior to waste placement) and capping systems. In both cases, slopes are long in relation to the soil veneer and the average normal stresses are low on the interfaces. Figure 14 shows the problem analysed, with the key variables defined. Soong & Koerner (1995) proposed a limit equilibrium assessment based on a two part wedge failure mode and including shear strength of the cover soil and seepage forces.

Effective stress analyses have been carried out for a 1.0 metre thick soil veneer with pore water pressures on the interface calculated using a parallel submergence ratio (PSR) of 0.5. Slope angles (β) between 14° (1 in 4) and 33.7° (1 in 1.5) have been analysed. Only the variability of interface shear strength has been considered in these analyses, however the method outlined by Duncan (2000) can be used to assess the influence of other parameters if required. Sliding has been analysed for three interfaces: textured HDPE geomembrane/coarse soil, textured HDPE geomembrane/ non-woven geotextile and non-woven geotextile/coarse

soil. Mean peak shear strength parameters have been obtained from the best fit lines calculated from the combined data sets and shown on Figures 5a, 3a and 6a respectively. The standard deviations of measured shear strengths have been taken from Figures 12b, 10b and 13b respectively. Analyses have been carried out using the combined data sets and also repeatability data sets. Both mean and standard deviation values have been taken over the appropriate normal stress range for the problem (i.e. 10 to 30 kPa). Shear strength parameters (α and δ) for mean, $+1\sigma_m$ and $-1\sigma_m$ measured shear strengths have been calculated for each interface. Table 2 shows the shear strength input parameters for each interface. These values differ from those shown in Table 1 because only data in the appropriate normal stress range for the problem have been used. As discussed in section 2.1, apparent adhesion values have been included as they are a function of the data sets and are used in conjunction with the slope of the failure envelope to define the measured interface shear strength over the normal stress range of interest.

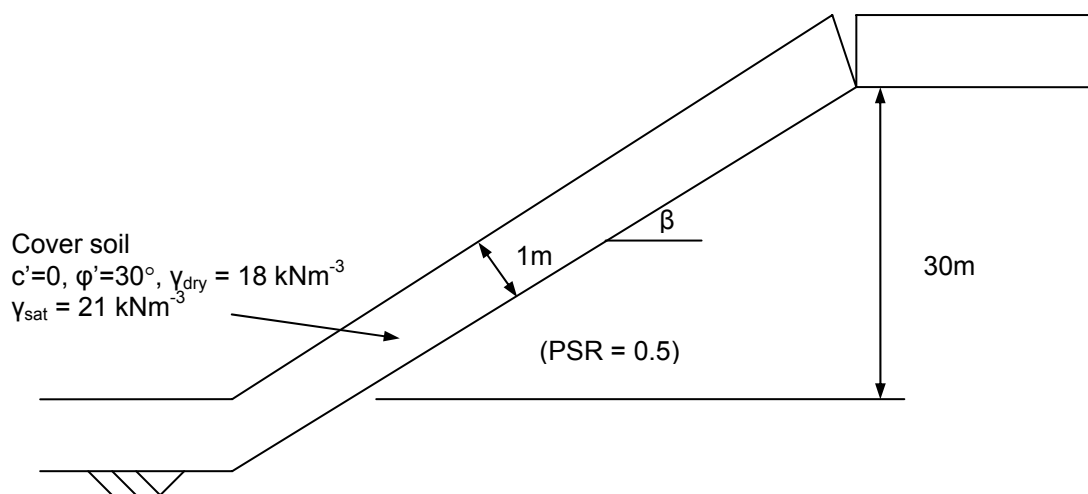


Figure 14. Diagram of the model used in the veneer stability analysis.

Table 2 Shear strength input parameters

Interface type, Data set	Shear strength parameters			
	Mean (α_m, δ_m)	Mean $+1\sigma_m$ (α_+, δ_+)	Mean $-1\sigma_m$ (α_-, δ_-)	Mean $-0.5\sigma_m$ ($\alpha_{-0.5}, \delta_{-0.5}$)
	Veneer stability (parameters obtained using data in normal stress range 10 to 30 kPa)			
Textured HDPE GM ¹ vs. Coarse Soil, Combined	3.6, 35.7	10.0, 37.0	-2.8, 34.4	0.4, 35.1
Textured HDPE GM vs. Coarse Soil, Criley & Saint John 1997	3.6, 35.7	5.6, 37.2	1.6, 34.1	2.6, 34.9
NW GT ² vs. Coarse Soil, Combined	4.3, 38.2	9.9, 43.3	-1.2, 32.3	1.5, 35.3
Textured LLDPE GM vs. NW GT, Dixon <i>et al.</i> (2000)	3.9, 32.2	5.0, 34.5	2.8, 29.8	3.3, 31.0
Textured HDPE GM vs. NW GT, Combined	-0.1, 38.9	4.2, 41.1	-4.3, 36.6	-2.2, 37.8
	Waste body stability (parameters obtained using data in normal stress range 100 to 300 kPa)			
NW GT vs. Coarse Soil	0.3, 35.4	5.9, 40.8	-5.2, 29.0	-2.5, 32.3
Textured HDPE GM vs. NW GT	3.6, 26.4	7.8, 29.3	-0.7, 23.3	1.4, 24.9

¹ GM = geomembrane; ² NW GT = non-woven geotextile

Figure 15 shows plots of P_f vs. FS_{MLV} for each interface. The interfaces with greatest variability of measured shear strengths (i.e. those involving coarse soil) show the largest P_f values for a given FS_{MLV} as expected. If a minimum $FS_{MLV} = 1.5$ is required in design, as is common practice, even the analyses based on the repeatability test data do not give a probability of failure low enough to be considered acceptable for design, as discussed below. It could be argued that it is more appropriate to compare P_f values with factors of safety calculated using characteristic shear strengths, FS_k , as these take into consideration variability, and hence uncertainty, in measured strengths. Figure 16 shows plots of P_f vs. FS_k and FS_{MLV} for the textured HDPE geomembrane/coarse soil interface based on the combined and Criley & Saint John (1997) data sets. Using characteristic shear strengths results in lower calculated factors of safety as expected, however the analyses do not indicate the full implication of the variability when compared to probability of failure values.

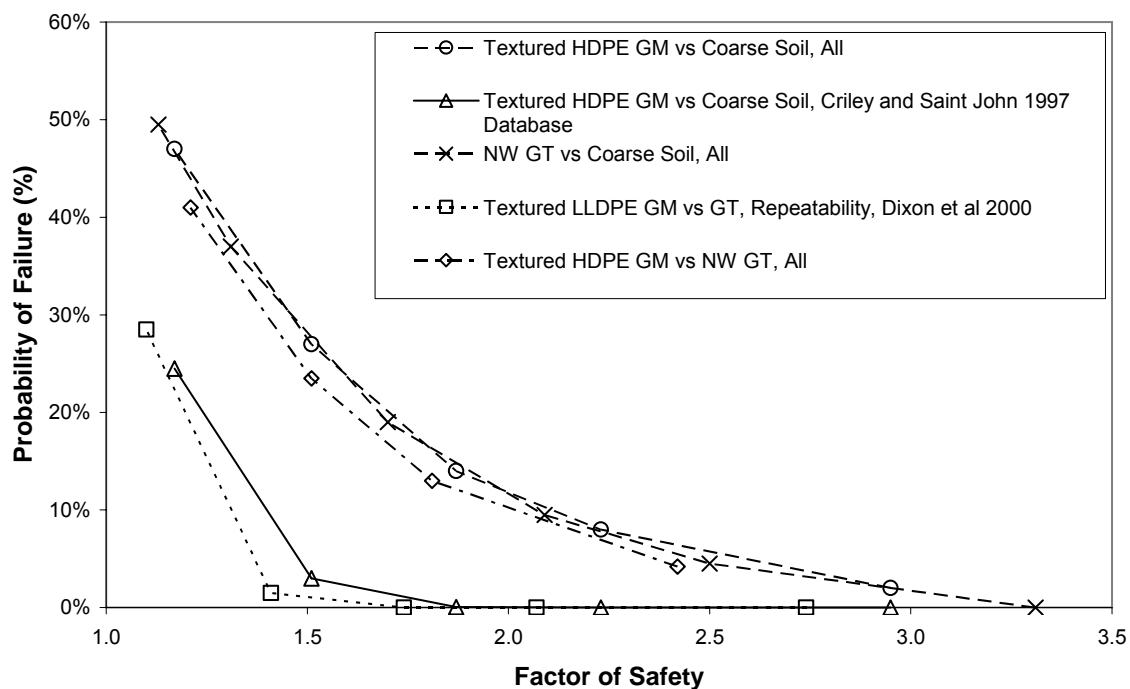


Figure 15. Probability of failure vs. factor of safety from veneer stability analysis, presenting data from combined data sets, Criley & Saint John (1997) and Dixon *et al.* (2000).

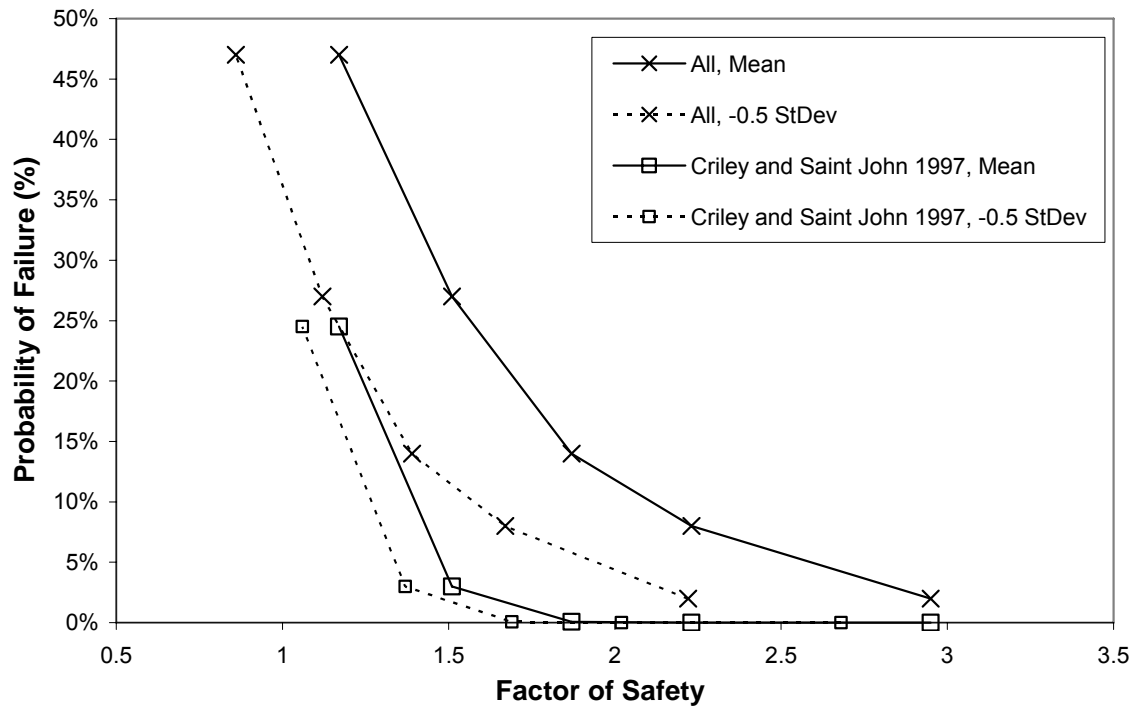


Figure 16. Probability of failure vs. factor of safety for veneer stability, showing the relationship between the mean and characteristic values for factor of safety, based on combined data and Criley & Saint John (1997) for textured HDPE geomembrane vs coarse soil.

4.3. Waste body stability

A second common design case in landfill engineering is stability assessment for a waste body placed against a side slope. This is a temporary condition in many quarry landfills and a permanent condition in valley landfills. There have been a number of failures, as discussed in the introduction, with sliding taking place along one or more interfaces within the lining system. Slope and waste geometries similar to those used by Sabatini *et al.* (2002) were selected for the reasons discussed above. Figure 17 shows the problem analysed with the key variables defined. Effective stress limit equilibrium analysis has been carried out using a standard slope stability computer package (SlopeW).

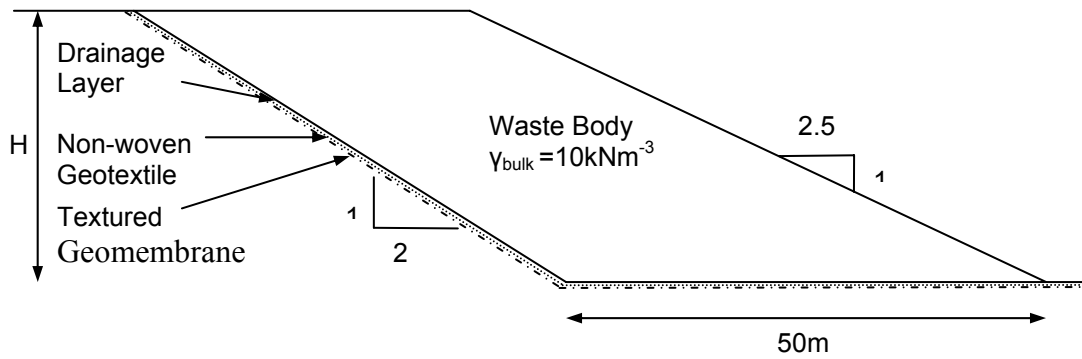


Figure 17. Diagram of the model used in the waste mass stability analysis.

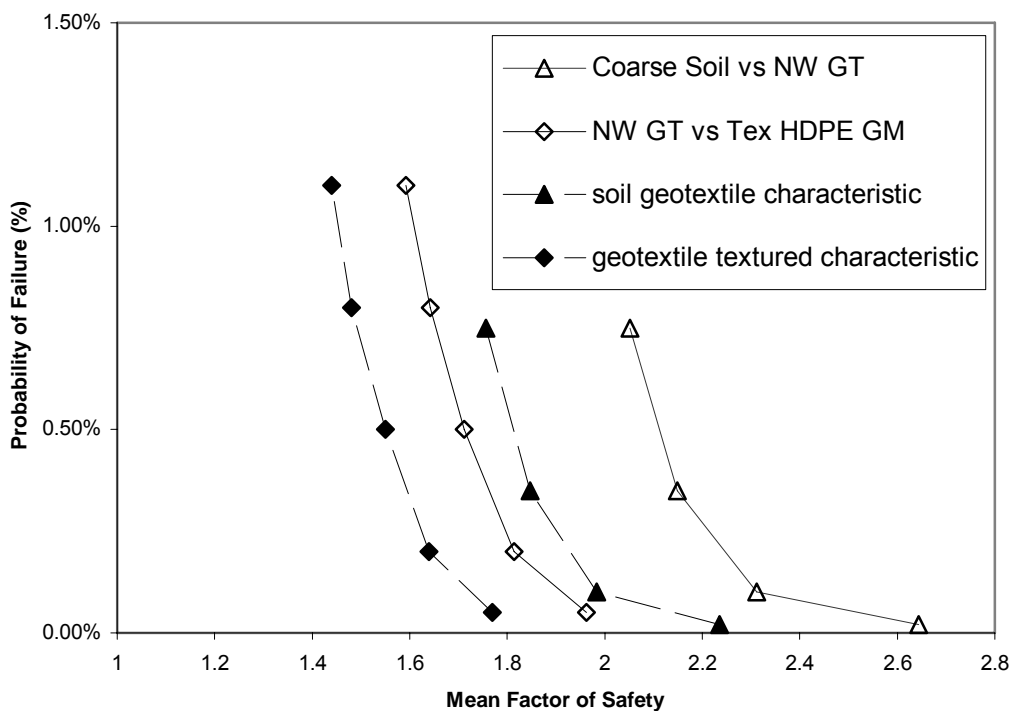


Figure 18. Probability of failure vs. factor of safety for waste body stability, showing the relationship between the mean and characteristic values for factor of safety, based on combined data.

Zero pore water pressures have been assumed on the interface due to the presence of the drainage layer. Slope height has been varied between 30 and 60 metres. Only the variability of interface shear strengths has been considered in this analysis. Sliding has been analysed for two interfaces: non-woven geotextile/coarse soil and textured HDPE geomembrane/non-woven geotextile. Each analysis has the same interface on the base and side slope. Mean peak shear strength parameters have been obtained from the best fit lines calculated from combined data sets and shown on Figures 6a and 3a respectively. The standard deviations of measured

shear strengths have been taken from Figures 13b and 10b respectively. Analyses have been carried out using the mean standard deviations of shear strength from combined data sets. There are currently no repeatability data sets available for these interfaces. Both mean and standard deviation values have been taken over the appropriate normal stress range for the problem (i.e. 100 to 300 kPa). Shear strength parameters (α and δ) for mean, $+1\sigma_m$ and $-1\sigma_m$ measured shear strengths have been calculated for each interface. Table 2 shows the shear strength input parameters for each interface and Figure 18 shows plots of P_f vs. FS_k and FS_{MLV} for non-woven geotextile/coarse soil and textured HDPE geomembrane/non-woven geotextile interfaces.

For limit equilibrium analyses using mean shear strengths, FS_{MLV} values greater than 2.6 and 2.0 are required for the two interfaces respectively to produce low P_f values (i.e. in the order of 0.1%). Even using characteristic shear strengths, FS_k values greater than 2.2 and 1.8 are required respectively to produce low P_f values. As for veneer stability, factors of safety typically used in design (i.e. in the order of 1.5) do not reflect the full implication of interface strength variability when compared to probability of failure values. As only combined data sets have been used in this study the results are conservative (i.e. the degree of variability is likely to be an upper bound). These analyses extend those presented by Sabatini *et al.* (2002) by demonstrating the increased probability of failure associated with using literature data sets compared to a carefully selected internal data set. Unfortunately, many designers currently only have access to the literature data sets and therefore the trends shown in this study could reflect current practice.

5. Reliability Of Landfill Stability Analysis

Consideration of shear strength variability is a critical element of stability assessment. Common practice using a global target factor of safety = 1.5 is based on the design engineer selecting 'conservative' mean shear strength values (i.e. uncertainty in shear strength is considered using engineering judgement). Use of characteristic strengths obtained via statistical analysis of measured values is an accepted approach (Eurocode 7, 1997). However, variability of input parameters is rarely obtained on a site specific basis. Probability of failure analysis does not require any input data in excess of that used to obtain characteristic strengths. However, it gives an additional benefit by providing a quantitative analysis of the reliability of the design. This has been clearly demonstrated by the increased P_f values for

analyses using literature derived interface shear strength data compared to those obtained using repeatability data.

In order to enable probability of failure analysis to be used as a decision making tool it is necessary to relate calculated values with consequences of failure, and hence to provide guidance on required values of P_f . Koerner & Koerner (2001) suggested boundary values based on the consequence of failure for a particular geosynthetic application being: low, medium or serious. For barrier applications such as landfill lining systems Koerner & Koerner (2001) proposed values of 0.3%, 0.05% and 0.01% for low medium and high consequences of failure respectively. For landfill design, low consequence could relate to instability of a soil veneer during side slope construction (e.g. a drainage layer). This type of failure typically can be repaired at relatively low cost and does not result in any uncontrolled discharge of gas or leachate into the environment. Medium consequences could relate to capping failure and slippage of a temporary waste slope. Cost of repair may be higher than side slope veneer instability but still low in relation to a serious failure. However, environmental damage could occur due to escape of landfill gas. Serious consequence of failure could relate to slippage of a waste body that has an impact outside the site. This is likely to be disruptive to site operation, costly to repair and can cause serious damage to the environment through pollution of groundwater by leachate and escape of landfill gas.

Liu *et al.* (1997) report typical lifetime probability for embankment dam failure in the order of 0.01 to 0.1%. These events result in serious consequences. Sabatini *et al.* (2002) suggest a conservative target P_f of 0.01% for waste body slippage while McCartney *et al.* (2004) do not discuss or propose target values for use in design of veneer covers incorporating GCL. As consequences of a failure can vary, the limiting values of P_f proposed by Liu *et al.* (1997) and Sabatini *et al.* (2002) are consistent with those suggested by Koerner & Koerner (2001) for serious ($P_f \leq 0.01\%$) and medium/low (P_f 0.05 to 0.3 respectively) events. For the waste slippage example shown in this paper, none of the P_f values calculated using literature data sets are less than 0.3% even though factors of safety ≥ 1.5 were obtained in some cases. This includes analyses using characteristic strengths (Figure 18).

Higher P_f values could be considered appropriate for veneer stability analyses (i.e. 0.05% for capping failure and 0.3% for slide slope veneer failure). However, all of the analyses giving FS_{MLV} or $FS_k = 1.5$ have P_f values above these suggested boundary values, including analyses

using Criley & Saint John (1997) repeatability data sets (Figures 15 and 16). This is a surprising result and either indicates poor current design practice or that the medium and low consequence acceptable values are too low. McCartney *et al.* (2004) reported factor of safety values corresponding to a P_f of 1% for GCL/textured HDPE geomembrane interfaces in an infinite slope veneer stability analysis with associated factors of safety calculated between 1.23 and 2.25 (depending upon the number of variables influencing the test results). They concluded that values of factor of safety associated with a $P_f=1\%$ can be significantly greater for slopes incorporating GCL interfaces than the typical design target value of 1.5. The findings of the current study for a range of typical interfaces is consistent with the findings of McCartney *et al.* (2004) .

As proposed by Koerner & Koerner (2001), discussion is required between regulators, owners and designers to define acceptable values in relation to the consequences of failure. Although landfill stability failures are not uncommon (Jones & Dixon 2003), and some failures are undoubtedly influenced by design, there is no evidence of systematic failure as a result of poor design. This tends to indicate that current best practice is producing designs with acceptable P_f values. Further research is required to obtain P_f values for landfill lining systems with proven good performance and known interface shear strength variability in order to aid the discussion on appropriate boundary values in relation to consequences of failure.

6. Conclusions

A large database of measured strengths, both peak and large displacement, has been presented for seven generic interfaces commonly present in landfill lining systems. The relationship between standard deviation and normal stress has been defined for combined data sets for each interface, except for interfaces involving fine soil. It is proposed that these summaries of test data can be used to supplement site specific test results in order to select appropriate mean and standard deviations for interface shear strength. These can then be used to calculate shear strength parameters for use in stability assessments.

Current practice is to carry out a limit number of site specific tests, but this provides insufficient information for the variability of interface strength to be considered in design. It is recommended that a sufficient number of site specific direct shear interface tests be carried out to provide statistical data for use in traditional limit equilibrium analyses using

characteristic values, and probability of failure analyses using the simple procedure described by Duncan (2000). In some cases, literature values are being used in lieu of site specific test results and this is considered to be unacceptable and likely to lead to unreliable designs as demonstrated by the analyses presented in this paper.

It has been shown that apart from repeatability data sets (where the same equipment, test specification and operator have been used to test samples from one source) other data sets show comparable degrees of variability. This indicates that variability caused by testing procedures, personnel, and equipment is as significant as the influence of differences in material samples forming a given generic interface.

In the combined data sets, large variability has been demonstrated, which results in unacceptable P_f values for both veneer and waste body slope stability. For veneer stability, the textured HDPE geomembrane vs. coarse soil combined dataset gives a P_f of over 25% even when the $FS_{MLV} = 1.5$. Using repeatability test data, the P_f for the same interface and slope angle (26.6°) reduces to 3% at $FS_{MLV} = 1.5$, however it is likely that this would still be considered unacceptable. These findings confirm the need for landfill designers to give greater consideration to variability of interface shear strength and to the consequences of failure when collecting information for use in design.

Designing based on combined criteria for factor of safety and probability of failure would allow uncertainty in measurement of interface shear strength to be considered fully. However, appropriate and attainable target factor of safety and probability of failure values need to be selected if this methodology is to be implemented into general practice. It is clearly unacceptable to rely on low values of FS_{MLV} using data with a large standard deviation, conversely, when repeatability tests have been carried out to derive interface shear strength, requiring a FS_{MLV} of in excess of 1.5 to achieve an acceptable P_f will in many cases be considered over conservative, and this will inhibit use of the method. Repeatability data sets have been shown to produce lower variability and hence more realistic information. It is recommended that repeatability data be used for design in place of the combined data sets. Unfortunately, to date there is only a small number of such studies reported in the literature. Additional repeatability studies on common interfaces need to be conducted.

Probability of failure analysis is an appropriate technique to apply to landfill design. The simple method used in previous studies (e.g. Koerner & Koerner 2001 and Sabatini *et al* 2002, McCartney *et al.* 2004) and in this paper requires the same input information on shear strength variability as traditional stability analyses using characteristic values. The cost of providing site specific data, which allows calculation of mean and standard deviation of measured shear strengths, is likely to be significantly less than the cost of repairing even a veneer slope failure. Regulators, operators and designers need to agree acceptable design requirements in relation to the probability of failure. This could lead to justification of the cost of obtaining the required quality of input parameters in relation to the consequences of failure.

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Paper 2, Appendix 1

First-order, second moment reliability-based analysis methodology proposed by Duncan (2000), based on the description by Koerner & Koerner (2001).

- Step 1 Assemble the mean value and standard deviations of the major variables that are to be used in the design method.
- Step 2 Calculate the most likely value of factor of safety (FS_{MLV}) using the mean values (i.e. following standard design methods).
- Step 3 Calculate the standard deviation (σ_{MLV}) and coefficient of variation (V_{MLV}) of the FS_{MLV} using the standard deviation of all the major design variables.

$$\sigma_{MLV} = \sqrt{\left(\frac{\Delta FS_1}{2}\right)^2 + \left(\frac{\Delta FS_2}{2}\right)^2 + \left(\frac{\Delta FS_3}{2}\right)^2 + \dots} \quad (3)$$

$$V_{MLV} = \sigma_{MLV} / F_{MLV} \quad (4)$$

When calculating each FS_i^+ and FS_i^- value, all other ΔFS_i variables are kept at their most likely values.

- Step 4 Using the values of F_{MLV} and V_{MLV} , determine the probability of failure (P_f) using Koerner & Koerner (2001) table 1, which shows the probabilities that the factor of safety (FS_{MLV}) is smaller than 1.0 based on a lognormal distribution for the factor of safety. Alternatively, the analytical approach given by Duncan (2000) could be used.
- Step 5 Assess the calculated factor of safety in respect of the P_f value. A P_f of 0% means there is no likelihood of failure.

Paper 2, Appendix 2 Full reference list for interface shear strength testing data base

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Paper 3: Analysis of a landfill directive compliant steepwall lining system

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Full Ref:

Fowmes, G.J., Jones, D.R.V. and Dixon, N. (2005). Analysis of a landfill directive compliant steepwall lining system. *Proceedings 10th International Waste Management and Landfill Symposium*, Sardinia. pp369.

Abstract

The design of landfill barriers requires the assessment of stability and integrity of the lining system. The Landfill Directive requirement for a geological barrier makes the design and assessment of steepwall lining systems extremely complex. A numerical method using the finite difference modelling code, FLAC, is proposed for assessing the stability, in terms of deformations, and integrity of both artificial sealing liners and geological barriers. Staged construction and non linear interface logic is applied to the model. Output data includes axial strain in geosynthetics, relative shear displacements and deformations. Limitations of the model are the constitutive model applied to the waste and the availability of accurate input data.

1. Introduction

The increased demand for waste disposal facilities in the UK requires the use of steep sided voids such as disused quarries. These voids require lining systems, which must remain stable on the steep perimeter side slopes. Following the introduction of the EC Landfill Directive (1999), enforced in the UK through the Landfill (England and Wales) Regulations 2002, a geological barrier, with properties equivalent to a thickness of 1m and hydraulic conductivity of $1.0 \times 10^{-9} \text{ ms}^{-1}$, is required along the base and up the sides of landfill sites. In addition, geosynthetic artificial sealing liners are often required to limit the lateral migration of landfill gas. Guidance on landfill stability assessment published by the UK Environment Agency (Dixon & Jones, 2003) requires an assessment of both landfill stability and lining system integrity.

Jones and Dixon (2005) consider stability failure as the ultimate limit state, where large scale movements occur with complete loss of function. Integrity, the serviceability limit state, may involve small scale movements, resulting in overstressing, and hence loss of function, in geosynthetic and geological barriers. Maintaining stability of a lining system, especially on a steep slope, becomes increasingly difficult with the inclusion of a geological barrier. In addition to stability, the integrity of the geological barrier must be assessed to ensure that excessive shear strains and hence increased hydraulic conductivity do not occur.

Stability assessment of lining systems containing geosynthetic elements are traditionally carried out by limit equilibrium techniques. It is believed that this method is over simplistic and does not accurately demonstrate the application of forces to the geosynthetic components and the resultant strains. The forces applied to the geosynthetic components, due to waste settlements, are often overlooked in the design process. A model needs to accurately represent the horizontal forces applied by the waste body and the down drag on the lining system due to waste settlement. The down drag component of the waste body is expected to primarily affect the tensile forces acting on the geosynthetics, whilst the horizontal support component is expected to primarily affect the stability of the lining system and the deformations that occur (Fowmes *et al* 2005).

A benched quarry side slope geometry is analysed in this paper comprising of a reinforced sand structure constructed in order to provide a smooth and stable subgrade for the geosynthetic elements. A geological barrier and artificial geosynthetic sealing liner are included in the design. A numerical analysis has been carried out to assess the stability and integrity. The model has been run without waste present and with waste present. Waste properties have been selected to allow for short term compression under overburden pressure resulting in settlements of approximately 15%. Long term time dependant settlements due to degradation have not been considered in the present study.

2. Numerical Modelling

The finite difference numerical code FLAC (version 4.00) has been chosen to analyse steep wall lining systems primarily due to its ability to model large strains. In “large strain” mode, as displacement occurs within the defined problem, the code updates the positions of the gridpoints within the mesh to account for the displacement. In many numerical codes, and in “small strain” mode in FLAC, the grid points are not updated as deformation occurs and displacements are calculated from the initial grid positions. It is of particular importance that large strains are modelled accurately due to the extremely large settlements that can occur in the waste mass.

2.1 Lining system geometry

The lining system analysed in this investigation was a reinforced sand body on a benched hard rock quarry slope. The 0.5m thick geological barrier was placed in direct contact with an incompressible stable rock subgrade, and then an engineered reinforced sand body was placed on the clay in an attempt to ensure the stability of the clay. An expanded polystyrene facing sheet was modelled above the reinforced sand used to provide a flat surface for placement of a geomembrane. A non woven needle punched geotextile is modelled between the geomembrane and the waste. Figure 1 shows the system geometry.

2.2 Multilayer geosynthetic liners

Previous analyses carried out by Byrne (1994), Jones (1999), and Jones & Dixon (2005), have used FLAC to analyse waste barrier interaction along a single interface. In these investigations two mesh segments, one representing the lining subgrade and the other the waste, were separated by a single interface. The properties of this interface were given values to represent the critical slip plane.

In the current study a two geosynthetic layer lining system is analysed. Beam elements are used to represent the geosynthetic elements (Itasca 2002) and interfaces defined between them. As beam elements can only interact with the model via interfaces, multiple beam layers can be placed between two meshes. Correct definition of the interfaces is imperative to the correct behaviour of the model. As individual geosynthetic elements are defined by individual beam elements, each can be given individual tensile strength properties. In this investigation two geosynthetics and hence three interfaces are defined. A geomembrane overlies the expanded polystyrene on the side slope, and forms a composite lining system with the clay along the base. Overlying the geomembrane is a geotextile protection layer. The waste is then placed directly in contact with the geotextile protection layer. It will be possible in future to add additional complexity to the system with additional slip surface geotextiles and a drainage layer.

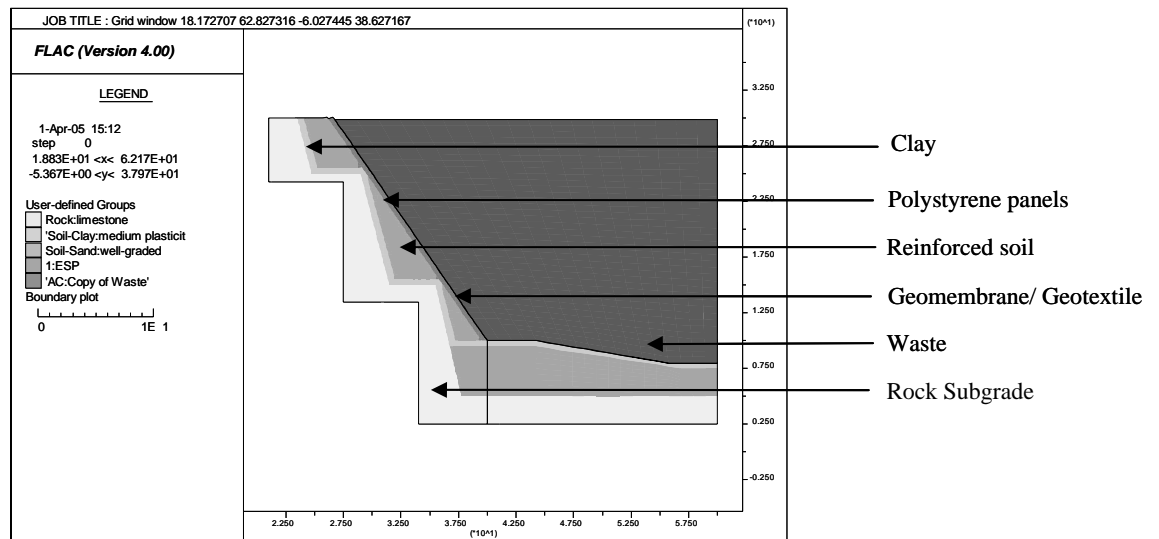


Figure 1. Lining system materials.

2.3 Construction sequence

Test models have shown that accurate representation of the construction sequence is necessary as this controls the sequence and magnitudes of the applied forces. For the analysis reported here the model was built in lifts of approximately 2.5m in height. Where waste has been modelled, the waste was also placed in 2.5m lifts following the placement of the lining system.

2.4 Non-Liner stress strain behaviour for interfaces

The use of strain softening interfaces was discussed by Jones & Dixon (2005). When using limit equilibrium techniques, the designer must chose a single set of shear strength parameters that can be peak, residual or factored values. Use of peak values can over estimate the strength of an interface, especially if large strains occur; however, use of residual parameters may give an unrealistically conservative design. The basic FLAC code allows linear elastic interface properties to be entered and a subroutine provided by Itasca (2002) allows strain softening interfaces, but only between grid elements. A routine has been developed by the authors within the FLAC sub-code, FISH, which allows a non linear stress strain response to be entered for the interfaces between geosynthetic (beam) elements. As such, peak strength can be

mobilised at a given displacement, and this value is subsequently reduced towards residual if further slip occurs along the interface.

2.5 Engineering detail

In order for the model to represent the realistic situation, engineering detail such as anchorage and attachments of reinforcements must be considered. However, simplifications of such details are necessary, but realistic behaviour must still be maintained. Modelling of the geomembrane was initially carried out without anchoring throughout the construction sequence. This may have resulted in slippage of the geomembrane rather than development of tension. As the geomembrane would be temporarily anchored and hence restrained from movement during placement of the waste, this model is over simplistic. The same model was run again with the top of the geomembrane anchored for each lift. This resulted in a tensile stress in the upper section of the geomembrane. The tensile forces transferred to the geomembrane were a function of the interface shear strength between the geosynthetic layers, as discussed below. Following this investigation temporary anchoring of the geomembrane has been used in subsequent analyses.

Another area of concern is the attachment of the geosynthetic reinforcing layers, within the sand structure, to the expanded polystyrene facing layer. Alterations to the model were carried out in order to investigate the deformations with and without attachment to the face. It was found that fixing the reinforcements reduced the deformations in the outer wall by on average 5%, and up to 17% in places. In the following investigation this has been modelled as not fixed, but further research into this area of the model is required.

3. Self Supporting Lining Systems

Dixon *et al* (2004) identified that current municipal solid waste placed using common practice provides insufficient lateral support to ensure the stability and integrity of a mineral only steep walled lining system. Waste adjacent to the wall provides low and variable lateral support, and due to the unwillingness to compact waste near to the geosynthetic lining system, waste material has low stiffness adjacent to the lining system.

As there is uncertainty about the amount of horizontal support derived from the waste body the model was run with no waste present. This area of the analysis primarily looks at the overall stability and local deformations that occur within the lining system. Integrity is considered in terms of shear strains within the compacted clay geological barrier. As there is no waste mass present and hence no down drag, the geosynthetic lining component is omitted for this area of the analysis as it has very limited effect on the behaviour of the underlying lining system. Additional calculations would be required to assess the integrity of the geosynthetics under self weight.

3.1 Influence of soil reinforcements

Models were run with different tensile yield properties of the geosynthetic reinforcement. Table 1 shows the input parameters used in the analyses. Figure 2 shows the deformations at the outer wall of the expanded polystyrene layer; this would represent the surface on which the membrane is laid.

Table 1 – Tensile yield of geotextile reinforcement

Run code	Yield Strength (kN)
NoWaste3	20
NoWaste4	10
NoWaste8	0.5

With tensile yield strength of 20kN, the maximum predicted horizontal displacements were less than 100mm, however, when the tensile was reduced to 10kN, stability failure occurred following the 6th lift, at a height of 15m. When looking at this failure in terms of horizontal deformations, the maximum deformations coincided with the base of each bench, with movements up to 600mm occurring before the model was stopped. When the tensile yield properties were reduced to a nominally low value of 500N, the model failed after 2 lifts, at a height of 5m. Figure 2 shows large horizontal deformations at the base of the lining structure as the reinforcement is unable to prevent a close to circular shear failure forming. In addition to the deformations in the outer surface of the expanded polystyrene, strains in the geological barrier are also

considered. Figure 3 shows the shear strain occurring within the reinforced soil geological barrier.

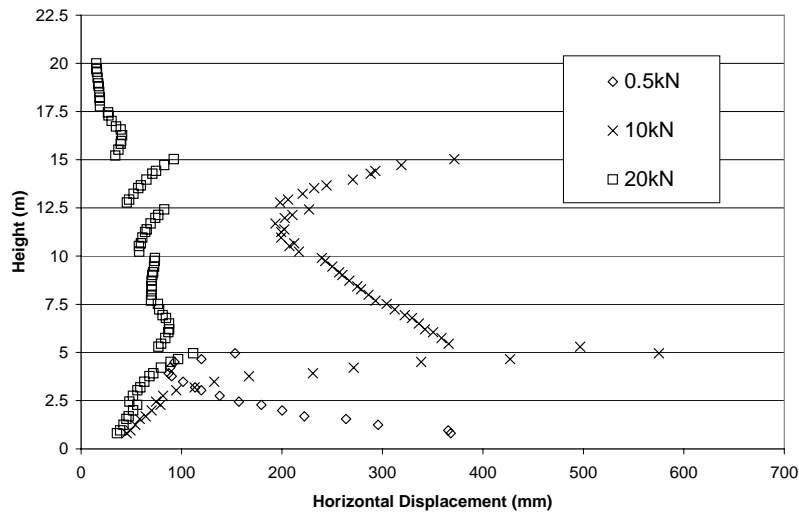


Figure 2. Horizontal Deformations at outer face of expanded polystyrene for different reinforcement yield strengths

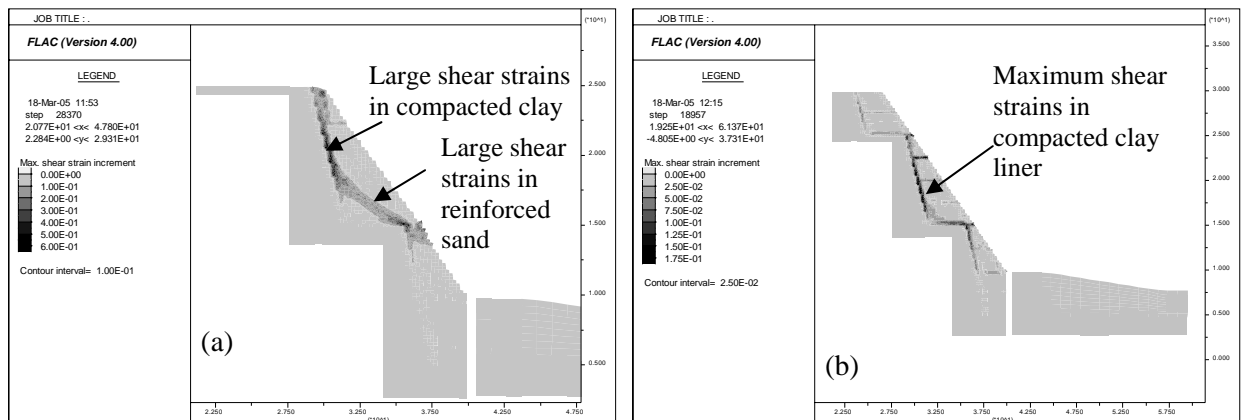


Figure 3. Shear strains in lining system constructed in the absence of waste a) Tensile yield of reinforcements 10kN Shear Strain >50% b) Tensile yield of reinforcements 20kN Shear Strain >10%

Figure 3a shows the lining system constructed using reinforcement with tensile strength of 10kN. The maximum strains are in excess of 50% and the failure plane can be seen to propagate down from the geological barrier, through the reinforced sand and through the toe at the first bench. Figure 3b show the shear strains with reinforcement strengths of 20kN. The reinforced sand mass shows much less

deformation, however, the geological barrier still displays shear strains in excess of 10%, which would still be considered unacceptable in design. A linear elastic perfectly plastic Mohr Coulomb model is applied to the clay and this is an area that requires further consideration in the modelling process, both in terms of constitutive model and input parameters.

3.2 Arguments against “stability only” analyses

It is unreasonable to only assess a self supporting lining system in the absence of waste. The effect of down drag on the lining system needs to be assessed in terms of axial strains in the geosynthetics and the forces transferred into the remainder of the lining system due to waste settlements. In addition to the effect of down drag, the weight of the waste acting on a lining system will cause compression of the reinforced soil body. In the lining system analysed in this paper, compression of the reinforced sand and compacted clay layers is induced by the self weight of the waste.

4. Waste Supported Lining Systems

The waste was modelled as a linear elastic perfectly plastic material with a Mohr Coulomb failure envelope. It is acknowledged that this model is over simplistic for representation of a complex heterogeneous material. However, even with a simplistic model finding meaningful input parameters is problematic. More advanced models are under development, Machado *et al* 2002, Dixon & Zhang (2005), and it is envisaged that these will ultimately be included in the analysis. The input parameters used in this investigation are displayed in Table 2. These parameters allow for settlements of approximately 15% representing compaction under overburden pressure. Long term time dependant settlement processes, caused by waste degradation, have not been considered in this study.

Table 2 - Parameters of the waste body (after Jones & Dixon, 2005)

Property	Value
Young's Modulus	500kPa
Poisson's ratio	0.3
Unit Weight	12 kN/m ³
Friction Angle	25°
Cohesion	5kPa

4.1 Influence of horizontal support

With a self supporting lining system stability is ensured without the need for horizontal support from the waste body. If however the design relies on a degree of support from the waste body for stability, the initial horizontal stresses in the waste control behaviour. If movement of the lining system into the waste occurs, stiffness of the waste controls the magnitude of lining deformation. Even in self supporting designs, horizontal forces exerted on the lining system by the waste body must be considered as they control the interface confining pressures and hence mobilised shear strengths, and they may result in compression of the sub-grade support.

4.2 Influence of down drag

As waste settles, stresses are transferred into the geosynthetic liners. An investigation was carried out into the effect of waste down drag on the geosynthetic tensile strain. Waste stiffness was altered in order to generate different down drag magnitude of the waste body. Table 3 shows the modulus properties used in each of the model runs.

To compare the effect of down drag, Figure 4 show the axial strains developed in the geotextile. Only the protection geotextile is considered here as the transfer of force into the geomembrane is discussed in the next section. The results show a large reduction in the axial strains transferred into the geosynthetic lining system as a result of stiffer waste. This is primarily attributed to the reduction in waste settlements. An

assumption is made in this analysis that the geotextile remains continuous from the top to the bottom of the slope.

Table 3 - Waste modulus characteristics.

Run Code	Waste Properties	
	Young's modulus	Poisson's Ratio
AltWaste11	0.5MPa	0.3
AltWaste9	1MPa	0.3
AltWaste10	2MPa	0.3

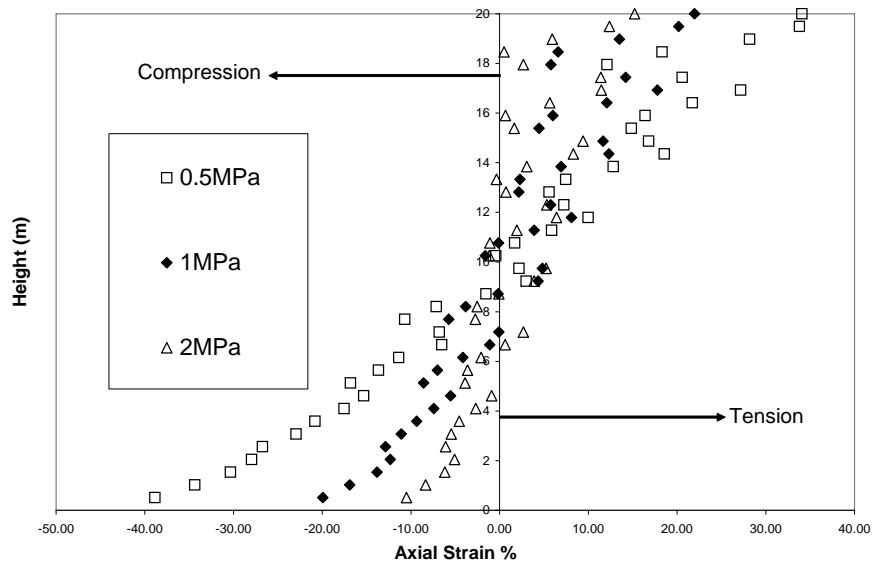


Figure 4. Influence of waste stiffness properties on geotextile axial strain. (N.B. Tensile strain is positive)

4.3 Influence of interface behaviour

The behaviour of the interfaces is fundamental to both the stability and integrity of the lining system. In this analysis, the influence of the interface between the geosynthetics on the transfer of stress, and hence strain from the geotextile to the geomembrane, is assessed.

The first part of the analysis was simplified to give at a linear stress strain relationship for the interface allowing easier comparison. The input parameters and resulting

strains in the geomembrane are given in Table 4. Run “IntProp5b” used a linear friction angle of 9° , except for lift 5, where a friction angle of 45° was applied. This high friction region could represent a geometric or constructional irregularity, for example a fold in the geosynthetics, restricting the slippage of one geosynthetic layer past the other.

The results show that an increase in the interface friction between the geosynthetic layers results in an increased axial strain within the geomembrane. The affect of the high friction section in run IntProp5b caused significant increase in the axial strain experienced in the geomembrane layer. This shows the consideration that must be given in addressing the construction detail and the likely heterogeneity in the surface between the geosynthetics. Three dimensional effects such as slope angle change will also have an impact on the slip surface and should also be considered. It is recommended that sensitivity analysis also be carried out at the other interfaces. The interface between the geomembrane and the expanded polystyrene will also have an influence on the tensile strength developed in the geomembrane.

Table 4 - Parameters of the geomembrane to geotextile Interface

Run Code	Interface Friction Angle (Degrees)	Maximum axial tensile strain in geomembrane
IntProp3b	9	1.6%
IntProp6b	12	2.6%
IntProp7b	15	4.4%
IntProp1b	18	5.1%
IntProp2b	27	11.4%
IntProp4b	36	12.7%
IntProp5b	9 (45 on lift 5)	9.9%

4.3.1 Strain softening interfaces

The effect of strain softening behaviour was addressed along the interface between the geosynthetic elements. The interface friction angle was altered but instead of a single

value being varied as in the analysis above, various parts of the interface friction/displacement curve were altered. Figure 5 shows the friction angle/displacement relationships for the interface between the geomembrane and geotextile used in the three of the analyses reported here. In the first instance, run PP_ssint5, shear strength is mobilised, then minimal post peak shear strength reduction is observed. The effect of increasing the peak shear strength and the residual shear strength were then analysed in runs PP_ssint6 and 7 respectively. Table 5 shows the axial strain and displacement data derived from these analyses.

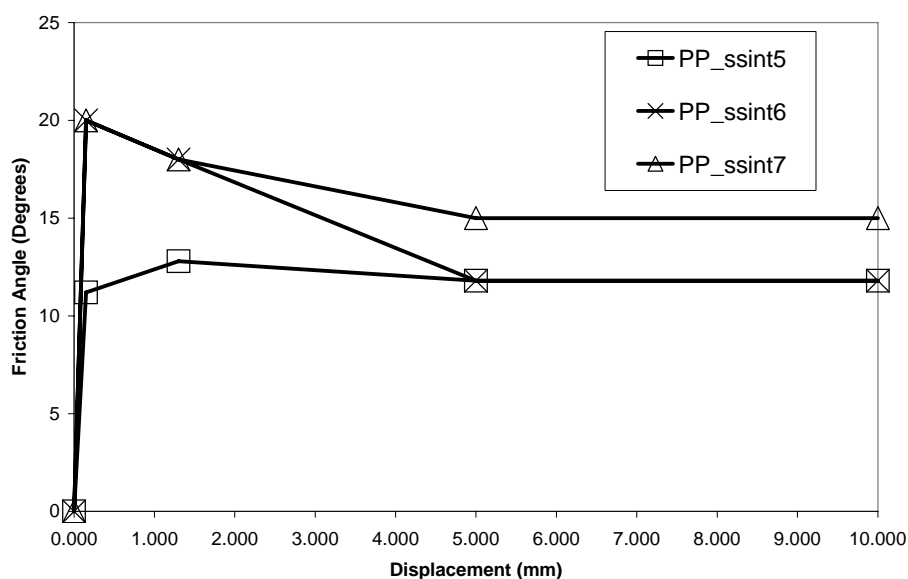


Figure 5. Curves of friction angle against displacement used in the analysis.

Due to the large displacements, an increase in peak shear strength along the interface has a much smaller effect than a residual shear strength increase. The increase in residual strength in run PP_ssint7 resulted in an increase in transferred stress from the geomembrane to the geotextile and a maximum axial strain increase of 80% in the geomembrane. Where the displacements at the interface are small, more shear stress is transferred to the geomembrane and hence more axial strain is developed. When evaluating an actual design it is suggested that the complete strain curve is analysed. Figure 6 shows the axial strain in the geomembrane from analysis PP_ssint7 above. If significant strains are predicted in a geomembrane, inclusion of a secondary slip surface geotextile between the existing geotextile and the waste can provide an additional slip plane.

Table 5 - Strain softening interfaces

Run Code	Maximum Axial Strain in geomembrane	Minimum Displacement along interface	Maximum Displacement along interface
PP_ssint5	2.47%	39mm	214mm
PP_ssin6	2.47%	39mm	228mm
PP_ssint7	4.47%	32mm	200mm

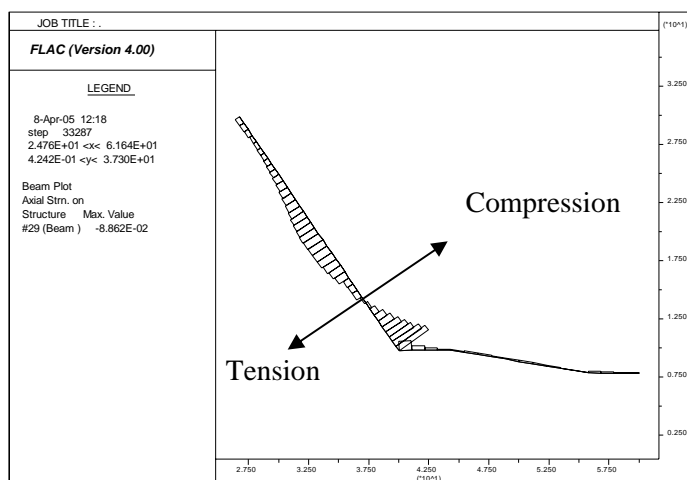


Figure 6. Axial strains in the geomembrane (maximum tensile value 4.47%)

5. Implications For Further Designs

The technique proposed in this paper can be used to assess the behaviour of a proposed lining system, in terms of strains, relative displacements between components and stresses. It is not suggested at this stage that this tool be used to give exact values for use in design, primarily due to limitations of the input data. However, the model can be used to investigate possible modes of failure and identify areas within a design that may require improving.

5.1 Further investigation

In order to represent the effects of down drag caused by waste settlement and horizontal support from the waste body, an appropriate waste constitutive model is

required. It is of great importance to the waste barrier interactions that the forces and displacements acting on the lining system due to the waste body are known. Waste models such as those under development (Dixon & Zhang, 2005) may ultimately be integrated in to this design methodology. The effects of time dependant settlement caused by degradation of the waste will need to be addressed by the model in order to assess long term stability and integrity of the lining system.

Modelling of this lining system, although subject to the limitations described above, has identified that the geological barrier could experience areas of high shear strain. Further work is required here in order to both accurately model the clay liner and to implement and develop materials for the geological barrier which will reduce this deformation. Validation of this model is required to ensure that response of the model is correct. Three methods of validation are proposed; back analysis of published data, centrifuge testing and full scale site instrumentation.

6. Conclusions

A method is proposed for assessing the integrity and stability of a Landfill Directive compliant lining system. The FLAC modelling code allows simulation of large strain problems and can be used to represent settlement in a waste body. Modelling of multilayered lining systems has been carried out using individual components to model the artificial sealing liner and a protection geotextile. This approach allows the transfer of stress, and hence strain, in the geosynthetic liners to be assessed. Stains in the geological barrier can also be predicted.

Limitations in the constitutive models and input parameters, in particular for the highly heterogeneous waste body, mean that at the current stage of development this technique should be used as an indication of processes occurring and likely areas of instability and integrity failure. Further work will investigate issues of degradation controlled waste settlements, material and geometry variability along with construction processes and engineering detail.

Acknowledgements

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Paper 4: Use of randomly reinforced soils in barrier systems

G.J. Fowmes, N. Dixon and D.R.V. Jones

Full Ref:

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Abstract

This paper investigates the use of fibre reinforcement as a means of increasing the shear strength in a low permeability soil barrier system, whilst maintaining the required hydraulic properties. Bentonite enhanced soil (BES) has been used as a host material since fibres can be added during the existing mixing process, thus requires minimal additional mixing plant. Results have shown that fibre reinforcement can provide both peak and post peak shear strength improvements. Behaviour of the soil fibre composite is improved by increasing fibre content to at least 0.5% (by weight) and fibres of 20mm in length are shown to give greater strength increases than fibres of 10mm. Preliminary permeability tests have shown that the fibre reinforcement has little effect on the hydraulic performance and a typical target permeability for use in landfill applications of $1 \times 10^{-10} \text{ms}^{-1}$ can still be achieved.

1. Introduction

Low permeability liners are required in many containment applications. Geosynthetic liners can be used in a range of situations, including on steep side slopes, where the use of mineral liners can be problematic. However, in landfill applications, following the introduction of the EC Landfill Directive (1999), enforced in the UK through the Landfill (England and Wales) Regulations 2002, a geological barrier must be included in the lining system, even on steep slopes. The finite difference numerical modelling code, FLAC, has been used to assess the stability and integrity of landfill lining systems (Fowmes *et al.*, 2005) and it has been shown that low permeability geological materials such as compacted clay and bentonite enhanced soils (BES) can experience large strains, particularly on steep side slopes. Figure 1 shows predicted high shear strains in a low permeability mineral liner on a benched quarry side slope. The mineral liner is located behind a reinforced soil support system against the stable quarry wall. This demonstrates that a method of controlling strains in such layers is required.

Continuous reinforcements may not be a viable option in low permeability barrier layers as they could provide a preferential flow path for the migration of fluids and gasses. It is therefore proposed that discrete fibre reinforcements can be implemented to improve stress strain behaviour whilst not compromising the permeability. Fibre reinforcement gives improved strength and stiffness characteristics by the mobilisation of tensile forces within the fibres. The tensile forces help to bind the sample together and resist deformation. Two failure modes are identified resulting in a bilinear failure envelope (Al-Refeai, 1991). Below a critical confining pressure the fibres will fail by pullout, above the critical confining pressure the dominant failure mechanism is by tensile yield. Dhillon (1999) identified two types of fibre reinforcement, extensible and inextensible. Inextensible fibres can rupture before the maximum tensile strain of the host material where extensible fibres allow much higher strains before failure. It is not appropriate to just present single values for the shear strength of discrete fibre reinforced soils as the effect on the complete stress strain behaviour of the soil needs to be assessed.

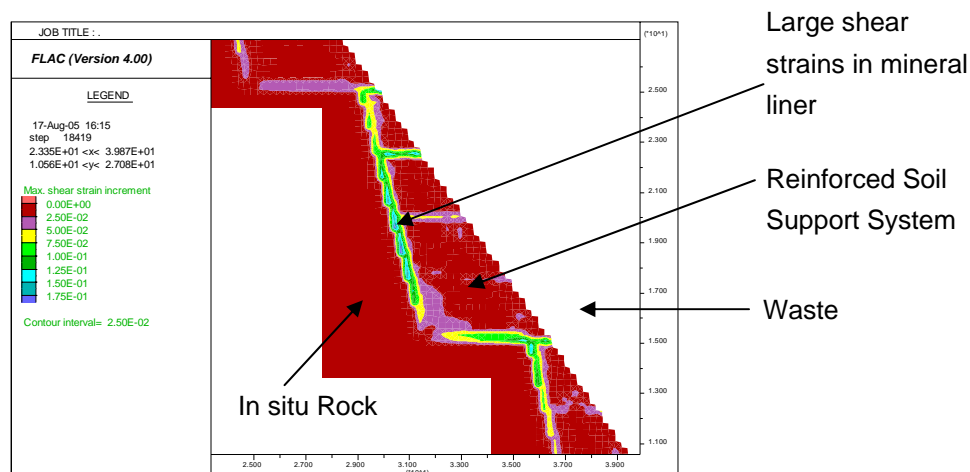


Figure 1. Predicted shear strains in a low permeability landfill lining system.

The effect of fibres on soil mass behaviour is material specific, for example Gray and Ohashi (1983) identify that low modulus fibres increased both peak and post peak strength, whereas Maher and Gray (1990) found only improvements in post peak behaviour. Hence it is believed that fibre and soil specific testing is required to determine the loading response and hydraulic behaviour of a soil fibre composite.

Fine grained plastic soils are usually used as mineral barriers, however, introducing fibres to form a well mixed relatively uniform material is problematic. Although the use of fibre reinforced clay has been reported by other authors (Miller and Rifai, 2004) the experience in this investigation was that mixing of fibres into stiff clays was very difficult, especially when trying to represent a mixing process that could be reproduced on site at a commercial scale. The only feasible way that was found to mix the fibres was to increase the moisture content of the clays, and this would be expensive and counterproductive, due to reduced shear strength, on site.

In regions where there is not a supply of an appropriate natural fine grained material, bentonite enhanced soil (BES) is often used. BES is a low permeability material which utilises the swelling properties of bentonite to fill the voids between sand particles (Jefferis, 1998). Mixing of fibres into the composite is easier than in fine grained plastic soil due to the dominant granular sand component and relatively dry mix. Mixing plant is already required on site hence the fibres could be added without the need for additional plant mobilisation.

2. Laboratory Testing

2.1 Material selection and mixing

Samples of bentonite enhanced soils were produced in the laboratory using a z-blade mixer. Mixing was not found to be problematic, except when fibre content exceeded 0.5% by weight, although this finding may be specific to the particular mixing equipment used in this study. Hand operations were required to achieve an even distribution of fibres in the BES prior to the mixing. However, commercial scale belt fed mixers with fibres added in the appropriate proportions to the other materials prior to entering the mixer should produce appropriate initial fibre distribution.

Tests were conducted using a range of fibre lengths. 10mm and 20mm long uncrimped polypropylene monofilament fibre, with 0.1mm diameter, and a commercially available 35mm length polypropylene crimped fibre were used in this investigation. Polypropylene fibres are suitable in waste containment applications as they have a proven track record in resistance to common leachates. The behaviour of fibre reinforced bentonite enhanced soil is affected by a wide range of variables, some of which are as follows:

- Fibre dosage
- Fibre length
- Fibre diameter
- Fibre crimping
- Bentonite content
- Fibre modulus
- Fibre yield point
- Fibre-soil surface friction angle
- Stress State
- Moisture content at mixing

A laboratory investigation has been conducted to assess the viability of fibre reinforcements for use in low permeability barriers. Compaction tests, 38mm and 100mm diameter unconsolidated undrained triaxial tests and falling head permeability tests have been carried out. The aim of the investigation was to assess if a strength increase could be achieved while retaining the required low permeability with fibre inclusions. Fibre length, fibre content (defined as fibre weight/dry soil weight), fibre type and moisture content were all studied in this investigation. For all of the testing described in this paper a nominal bentonite content of 10% (by dry weight) was used with Leighton Buzzard sand as the host soil. Although the

affect of fibre reinforcement is material dependent on the host material, this sand and bentonite content represent those which are typically used on site.

2.2 Compaction tests

Compaction tests were carried out in accordance with BS 1377, Part 4: 1990 using a 4.5kg rammer. The compaction tests gave the same optimum moisture content for unreinforced and fibre reinforced BES, however, the compacted dry density was lower with the fibres present. Although there is a difference in the mass of the fibres compared to the soil component, this does not account for the total reduction in dry density. Based on the observed behaviour all subsequent investigations use an overall moisture content of 12.5% unless otherwise stated.

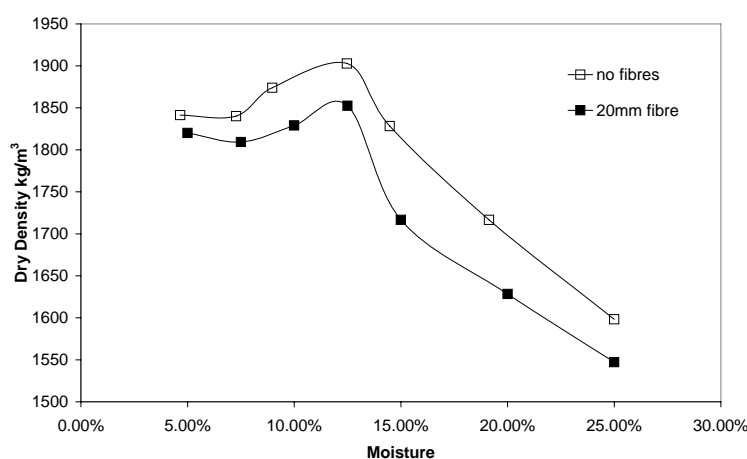


Figure 2. Dry density against moisture content for both unreinforced and fibre reinforced BES.

Prabakar and Sridhar (2002) reported reduction in dry density and optimum moisture content with increasing fibre content. However, Fletcher and Humphries (1991) reported increases in maximum dry density with fibre addition. These conflicting findings are probably a result of material variations and show the importance of material specific testing.

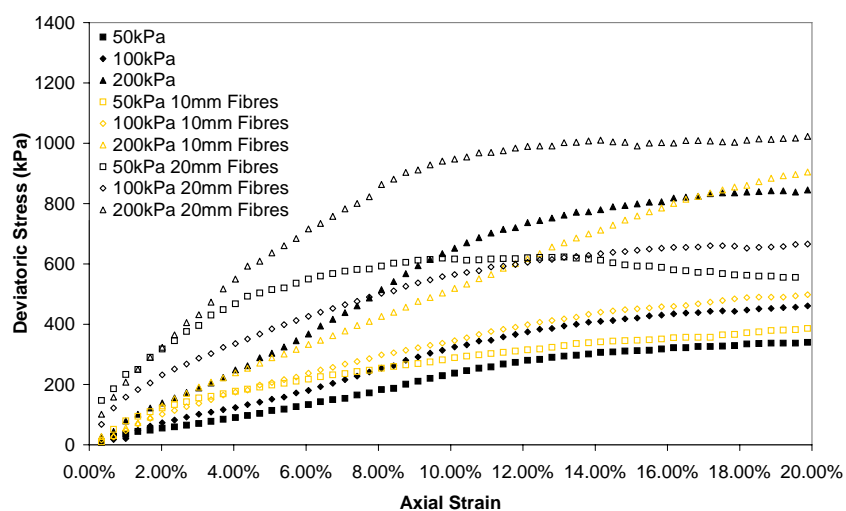
2.3 Strength tests

In order to compare the strength behaviour of the fibre BES composites, unconsolidated undrained triaxial tests were carried out in accordance with BS 1377, Part 7:1990. It was decided to use 32mm diameter samples to allow a greater number of tests to be carried out.

As the fibres are long relative to the sample diameter results may be affected by trapping of fibres between the sides of the sample and the membrane and also the samples may not undergo sufficient deformation to mobilise fibre reinforcement. It should also be noted that samples may not have been fully saturated. Despite these limitations, tests provided a comparative tool to assess the strength performance of unreinforced and fibre reinforced BES.

2.4 Fibre reinforcement and fibre length

A comparison was carried out between unreinforced BES and BES reinforced with 10mm and 20mm uncrimped fibres. Fibre reinforced BES is shown in Figure 3 to give improved stress strain behaviour over unreinforced samples. 20mm fibres give the greatest performance improvement over unreinforced BES. Although an improvement in the stress strain characteristics is observed with 10mm fibres, it is much less pronounced than with longer fibres. Shear strength parameters for linear best fit lines through peak strengths are also given. Both reinforced and unreinforced samples deformed by barrelling.



	c (kPa)	ϕ (°)
No Fibre	26.7	32.4
10mm	32.3	32.7
20mm	88.4	30.6

Figure 3. Stress strain curves and derived peak shear strength parameters showing the influence of fibre inclusion and length.

Santoni *et al.* (2001) found that monofilament fibres 51mm in length significantly outperformed those of 25mm in length. These longer fibres were not considered here as they may induce preferential fluid migration, however, they will be considered in future studies. Prabakar and Sridhar (2002) found in tests on silty soils that the shear strength of the soil sample increased up to a maximum fibre length of 20mm, above this length the fewer number

of fibres resulted in the fibres failing to bind the soil in a single interlocking matrix. Fewer numbers of fibres are present at longer fibre lengths as the total fibre mass remains the same and the mass of each individual fibre increases.

2.5 Fibre content

Fibre content comparisons were carried out for two fibre types; 20mm uncrimped fibres and 35mm crimped fibres. Fibre content of 0, 0.25% and 0.5% are compared and the results shown in Figure 4. A clear increase in shear stress at a given strain can be seen with fibre content of 0.5% at both fibre lengths and at all normal stresses. For 0.25% content of 20mm fibres shear stress increases are observed at 200kPa, little stress gain is achieved at 100kPa and a decrease in stress at a given strain is seen at 50kPa. It is suggested that this could be due to the failure mode of the fibres with pullout and sliding along the fibres at low confining stresses, and mobilisation of tensile stress at higher confining stresses. The peak strength parameters show the strength increase due to the fibre inclusion. The low apparent cohesion values at 0.25% with uncrimped fibres and 0.5% with crimped fibres are a function of a relatively large strength increase at 200kPa rather than a poor performance at lower normal stress, and therefore a high friction angle is derived.

With 0.25% addition of 35mm crimped fibre a modest strength increase is seen at 50, 100 and 200kPa confining stresses, with the strength increase much smaller than with 0.5% fibre content. It is believed that due to the crimped nature of the fibres, pullout is restricted. It must be noted that the crimp dimensions of the fibre should be proportional to the grain size; thus if the distance between the crimp is too small, the fibre may have to “uncoil” before any tension can be mobilised.

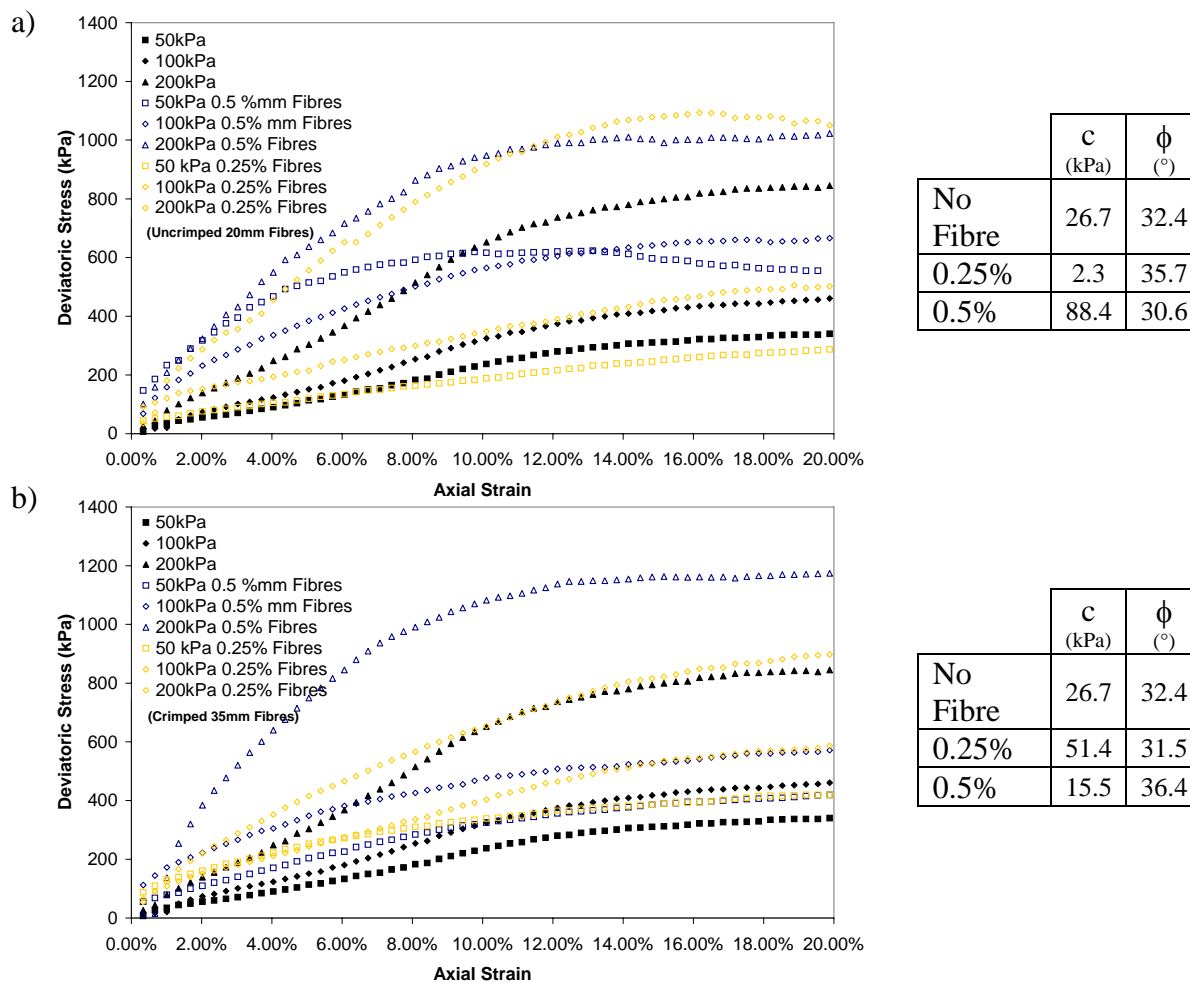


Figure 4. Stress strain curves and derived peak shear strength parameters showing the influence of fibre content (weight %) using a) 20mm uncrimped and b) 35mm crimped fibres.

2.6 Moisture content

Moisture content was varied to investigate the effects of hydration of the BES. The effects on the fibres of doubling the overall soil moisture content from 12.5% to 25% were investigated and the results are shown in Figure 5. A large drop in shear strength of the unreinforced BES occurs at the higher moisture content. The shear strength was improved by the presence of both 35mm crimped and 20mm uncrimped fibres, however, was still significantly lower than unreinforced BES at 12.5% moisture content. The shorter 10mm fibres were found to have a detrimental effect of the shear strength of the BES as they easily pulled out and provided a preferential slip surfaces in the shear zone during the observed barrelling of the sample. Deconstructed samples showed no failure or elongation of fibres indicating slip had occurred. It must be noted that the additional moisture content was added to the mixture prior to

compaction, so in addition to strength drop due to the presence of moisture, strength will may also occur due to the lower dry density achieved during compaction (see Figure 2).

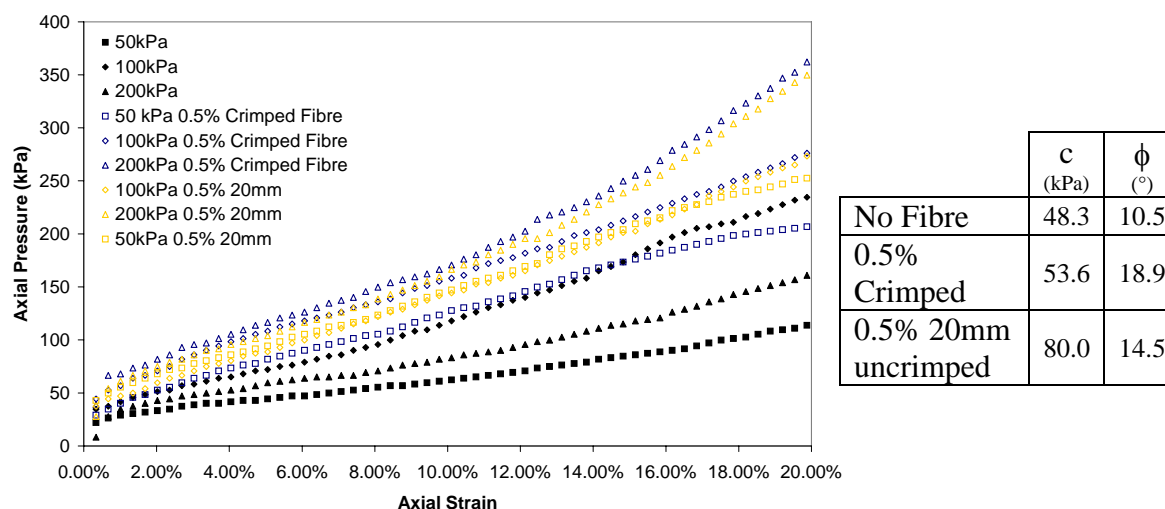


Figure 5. Stress strain curves and derived peak shear strength parameters (based on 20% axial strain) showing the influence of fibre reinforcement in BES with 25% overall moisture content.

2.7 100mm diameter triaxial tests

Due to the fibre lengths being large relative to the 32mm diameter triaxial sample, the sample size may have influenced the results. Therefore, 100mm diameter triaxial tests were conducted to investigate this aspect. The samples were all compacted and tested with a moisture content of 12.5% and bentonite content 10%. The three samples tested were; unreinforced BES, BES with 0.5% 20mm uncrimped fibres and BES with 0.5% 35mm crimped fibres. The results are shown in Figure 6. The unreinforced BES gave a peak strength value at 9.1% axial strain. The 35mm crimped fibres gave a peak value at 8.1% axial strain with shear stress increases, compared to the unreinforced BES, at all comparable axial strains. These samples failed with a distinct shear plane, and deconstructing the samples showed pullout failure had occurred. The sample reinforced with the 20mm uncrimped fibres did not give any shear stress increase over the unreinforced sample until 3% strain, but gave a strain hardening effect up to 10%. After 10% strain the material behaved in a perfectly plastic manner, with no post peak strength loss, barrelling of this sample occurred but no shear plane formed.

When comparing the results from the 32mm and the 100mm triaxial samples the results appear quite different when measured in terms of axial strain, however, the stress vs displacement curves are a much closer match, with similar peak strength values mobilised at comparable displacements. It is believed that the displacements occurring along the shear zone, which mobilise tensile reinforcements, are critical and not the overall strains. Displacements in the order of 15mm to 20mm were typically required to mobilise peak strengths in the 32 and 100mm diameter samples.

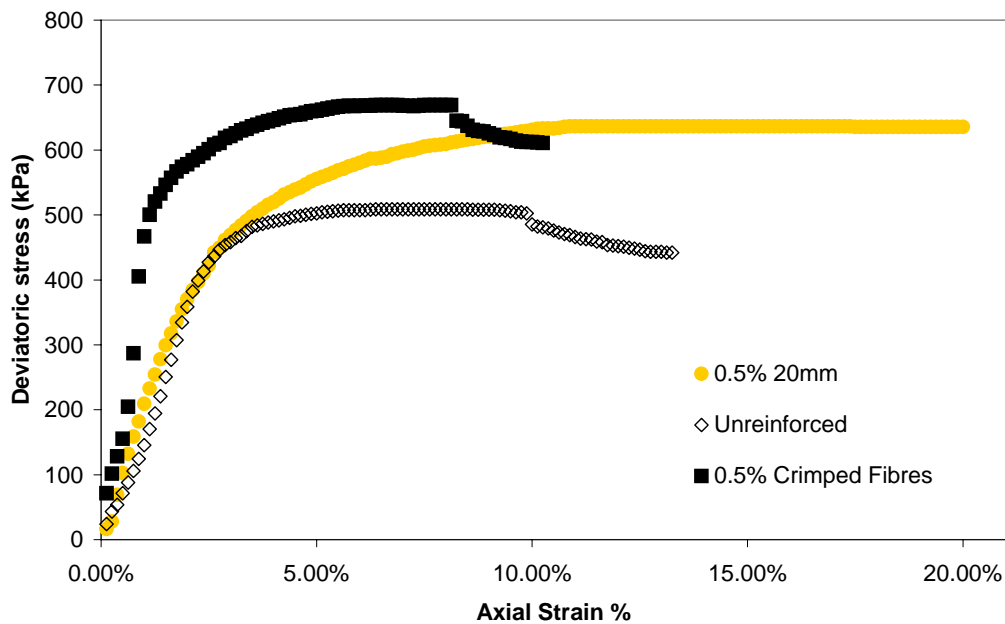


Figure 6. Influence of fibre reinforcement in 100mm triaxial samples.

3. Permeability tests

Preliminary falling head permeability tests have been carried out on samples of fibre unreinforced and reinforced BES containing 0.5% of 35mm crimped fibres and compacted with a moisture content of 12.5%. The material was left for 48 hours prior to testing in order to allow the bentonite to hydrate. Results showed little variation between the fibre reinforced and the unreinforced BES samples. Permeability values of less than $1 \times 10^{-10} \text{ms}^{-1}$ were measured. It is known that in needle punched Geosynthetic Clay Liners (GCL) the swelling nature of the bentonite means that despite continuous inclusions through the bentonite core, low permeability is still achieved. The principle is the same in fibre reinforcements; as the bentonite swells it closes any paths along the sides of the fibres thus retarding fluid migration.

4. Discussion and Further Research

On examining deconstructed samples following compaction most of the fibres were oriented between approximately 0 and 65 degrees of horizontal. McGown *et al.* (1978) identified that as the reinforcement orientation moves away from the plane of maximum tensile stress, their influence diminishes. Compaction and deformation will align the fibres from random towards the maximum tensile stress plane. The aligned fibres induce a greater reinforcing effect, when subjected to loads in the same axis as the compaction, thus giving the fibre reinforced soil strain hardening behaviour. Santoni *et al.* (2001) describe that strain hardening characteristics were seen up to strains of 25%. A possible negative effect of this process to be considered with the orientation of fibres is that this will increase the likelihood of preferential flow paths forming. This is of particular concern on steep side slopes as the aligned fibres could form preferential paths allowing lateral flow through the liner. Further investigations should assess anisotropic hydraulic conductivity. Additional permeability tests are planned using flexible wall equipment in order to reduce potential fluid transport at the sample sides.

In all deconstructed samples pullout was the predominate mode of failure. It may be necessary to select fibres with higher fibre soil interface friction angle to allow greater mobilisation of tensile stress. If fibres are under tension extensibility can be controlled in BES-fibre composites by the selection of polymer varieties. The nature of the fibres should be selected to provide the desired composite properties.

Further work is required on large diameter drained triaxial tests, in order to derive parameters for use in numerical modelling of mineral lining systems. Additional combinations of fibre type and length should be considered, in addition to different host sands. One area of concern is the lower shear strength when the bentonite becomes hydrated, although fibres have been shown to improve shear behaviour. Further work is required to assess whether this is sufficient to enable the use of these products on steep side slopes.

5. Conclusions

Strength increases can be achieved in low permeability BES liners using fibre reinforcement. The magnitude of shear strength increase and strains at which the improvements occur is dependent on the fibre type, length and dosage. Fibre pullout is increased by the lubricating

effect of hydrated bentonite. This must be considered, and fibre types and concentrations selected, such that they do not have an adverse effect when samples contain hydrated bentonite. Initial permeability results have shown no adverse effect from fibre inclusions, however, more detailed analysis of permeability and compaction induced anisotropy is required. This study indicates that reinforced BES has improved strength and deformation behaviour and warrants further investigation for use as a geological barrier on steep slopes.

Acknowledgements

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Paper 5: Validation of a numerical modelling technique for multilayered geosynthetic landfill lining systems

G.J. Fowmes, N. Dixon, and D.R.V. Jones

Full Ref:

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Abstract

It has become common practice to conduct numerical analyses to assess the stability and integrity of side slope landfill lining systems, however, information that can be used to validate such models is extremely limited. This paper contains data from a series of large scale laboratory tests containing geosynthetic elements of a multilayered lining system exposed to downdrag forces from a compressible synthetic waste material (rubber crumb). These data are compared to the results from numerical analysis of the same problem. The numerical results are from initial best estimate analyses, with interface and synthetic waste properties derived from a laboratory testing programme and geosynthetic material properties from manufacturers. The observed trends of tensile stresses in the geosynthetics and relative displacements at interfaces in the laboratory testing 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.

Keywords

Landfill design, numerical analysis, geomembrane tension, multiple layer lining systems, interface displacements.

1.0 Introduction

Guidance on landfill stability assessment published by the UK Environment Agency (Dixon & Jones, 2003) requires an assessment of both landfill stability and lining system integrity. Stability assessments of landfill lining systems are traditionally carried out by limit equilibrium techniques. However, such techniques cannot assess integrity (Long *et al.* 1994). The forces applied to the geosynthetic components, due to waste compression and degradation, are often overlooked in the design process. In order to represent compression and degradation controlled settlement induced downdrag, a model needs to represent the horizontal forces applied by the waste body and down drag on the lining system due to waste settlement. Previous studies by Fowmes *et al.* (2005 and 2006) have used the FLAC modelling code to assess the integrity of geosynthetics in multilayered lining systems. Whilst attention has been paid to accuracy of input parameters, it has been assumed that the model, including the implementation of multiple layered geosynthetic interface interaction, is correct.

Limited data is available for comparing measured behaviour of multiple layered geosynthetic lining systems to that predicted by numerical modelling techniques. Villard *et al.* (1999) present a model using Finite element modelling techniques, of a landfill lining system during construction and compare predicted behaviour to field measurements. This study showed a good correlation, however, the post waste placement data is not presented for either the field or numerical models due to a poor match between the data sets. The study reported in this paper aims to validate a numerical modelling technique by comparing interaction of geosynthetic landfill lining system components subjected to downdrag forces in a large scale laboratory model with the results of a numerical analysis of the same system.

2.0 Laboratory Testing

Large scale laboratory testing was carried out so that measured behaviour could be compared to a numerical model of the same system. It must be emphasised that the physical model is not intended to directly represent behaviour of a landfill lining system; it was designed to represent the interaction of lining system components when exposed to downdrag forces and hence generate post-peak interface displacements

experienced in side slope landfill lining systems. Thusyanthan *et al.*, (2004) investigated tension in a scaled geomembrane on a shallow side slope in a centrifuge model, including dynamic loading, however, only a single geosynthetic was included in the lining system. A test without increased gravitational acceleration was adopted in this investigation as it allowed full scale geosynthetics to be used, and thus interface behaviour could be characterised using standard direct shear apparatus. A compacted clay barrier layer was not included in the laboratory model. Formation of a planar vertical face would have been problematic, as it would have introduced a number of additional variables (i.e. related to moisture content and density) and it is not routinely present directly beneath the geomembrane in steep slope lining systems in UK practice. A wood subgrade was employed, which could be readily characterised and used to represent geomembrane support systems on steep slopes.

2.1 Test chamber design

The test chamber consisted of a 1 m x 1 m x 1 m void that was filled with compressible rubber crumb. One side of the test chamber was formed of a vertical wooden wall supported by a frame on which samples of geomembrane covered with a geotextile were placed (Figure 1). The front of the test chamber consisted of a 25 mm thick glass panel to allow movements in the system to be observed. The remaining two walls were sheet steel. The wooden subgrade was selected as it would provide a low friction interface beneath the geomembrane thus allowing measurable tensile strains in the geomembrane.

The sheet steel and glass walls of the test chamber were lined with a 0.1 mm thick sacrificial plastic sheet (shown in Figure 1) which reduced the friction between the box side and the compressible synthetic waste (rubber crumb) in order to lessen the edge effects imposed by the test chamber dimensions. The plastic sheet moved with the synthetic waste, allowing slip of the interface between the box side and sacrificial plastic layer. Direct shear testing showed that the interface friction angle between the rubber crumb and test chamber was 7° when a sacrificial sheet was included. The low friction on the side wall, resulted in an observed compression in the lower 200 mm of the synthetic waste being equal to 91% of the settlement in the top 200 mm of the

synthetic waste. Based on these measurements it was considered valid to model the experiment as a plane strain problem subjected to one dimensional compression.

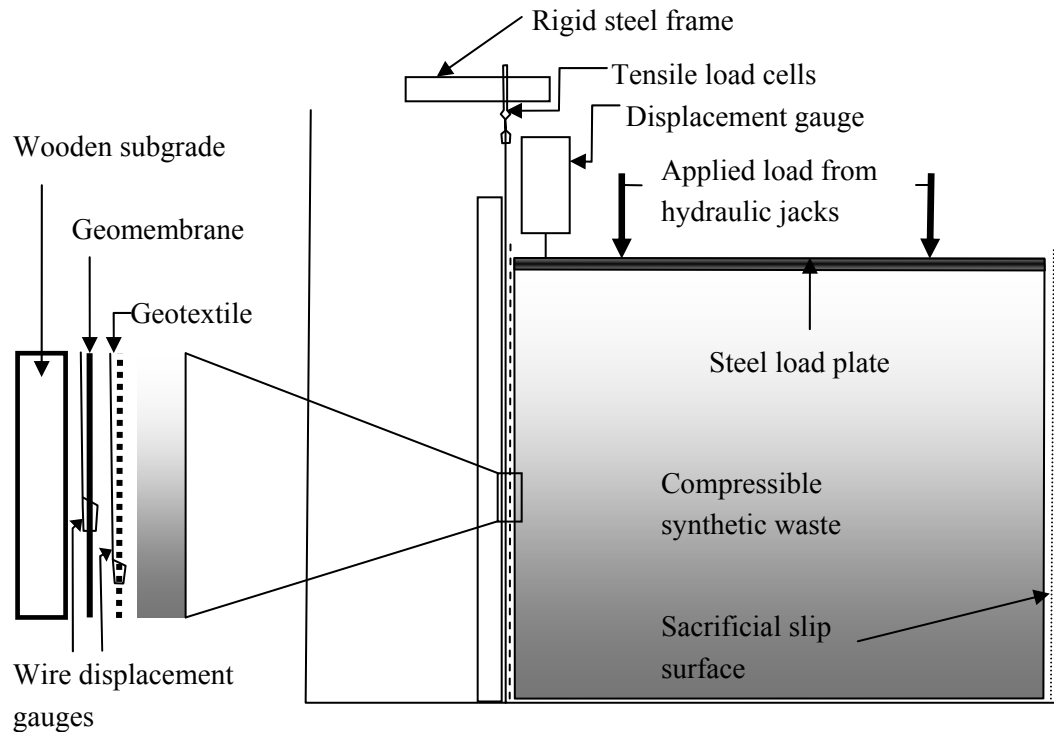


Figure 1 Schematic of measuring box

A vertical slope was adopted to simplify load application, as a rigid plate of fixed dimensions was used for load application. This meant that the load application area could be kept constant, which reduced uncertainty in calculating the horizontal stress applied to the lining system.

2.2 Synthetic waste

Rubber crumb, with a grain size ranging from 2 to 8 mm, was selected as a synthetic waste material as it has similar compression behaviour, shear strength and mobilised horizontal stresses to municipal solid waste. However, unlike waste it does not exhibit large heterogeneity which would be problematic in tests of this scale. As the rubber particles do not yield during compression (i.e. elastic particle compression and particle reorganisation occurs representing recoverable and non recoverable settlement respectively), the rubber can be reused.

The compressive behaviour of the rubber crumb was tested at different scales using a CBR mould, a 0.125m³ test chamber and in the 1m³ test chamber. The shear strength of the rubber crumb was measured using a 100 x 100 mm shear box. The material was sheared at 1 mm/min in the first series of tests and 0.1 mm/min in the second series of tests, with no perceptible difference in the behaviour under the more rapid shearing. The shear strength of the rubber crumb under direct shear can be defined by a friction angle of 29.3° and an apparent cohesion of 3 kPa.

2.3 Geosynthetic materials

A non-woven geotextile was used with tensile strength (in the machine direction) of 20 kN/m, a thickness (at 2 kPa) of 3.9 mm, and tangential tensile modulus (at 50% strain) of 1.2x10⁴ kPa. The geotextile was not anchored at the top and therefore did not develop tension at the top.

Three geomembranes were used in the investigation; two textured linear low density polyethylene (LLDPE) (one blown film, one impinged) geomembranes and one mono textured (impinged texturing) high density polyethylene (HDPE) geomembrane (tested both textured and smooth side up). The geomembrane properties are summarised in Table 1. The geomembrane was anchored at the top and therefore tension was able to develop.

Table 1 Geomembrane properties

	Type G LLDPE	Type S LLDPE	Type G HDPE
Polymer Type	LLDPE	LLDPE	HDPE
Manufacturer	Manufacturer G	Manufacturer S	Manufacturer G
Texturing	Double	Double	Mono
Texturing Type	Impinged	Blown film	Impinged
2% modulus	4.2x10 ⁵ kPa	4x10 ⁵ kPa	7x10 ⁵ kPa
Compressive modulus (assumed)	4.2x10 ⁴ kPa	4x10 ⁴ kPa	7x10 ⁴ kPa
Thickness	1 mm	1 mm	1 mm
Yield strength			16 kN/m
Yield Elongation			9 %
Break Strength	12 kN/m	17.5 kN/m	10 kN/m
Break Elongation	250 %	400 %	100 %

2.4 Load application

A total of 75 kN (7 x 10kN hydraulic loading increments, and 5 kN kentledge) of load was applied to the upper surface of the compressible synthetic waste. A hydraulic four point loading system was used to apply up to 70 kN to a rigid steel load plate (Figure 2). Due to the presence of the glass front to the test chamber the pressure application was limited to the 80 kPa exerted at the base of the chamber (75 kN applied load and 5 kN from the self weight of the material). Vertical strains of 28% were generated in the synthetic waste using this loading arrangement, which compare with total waste settlements in the range 20% to 30% (Jones and Dixon, 2003)

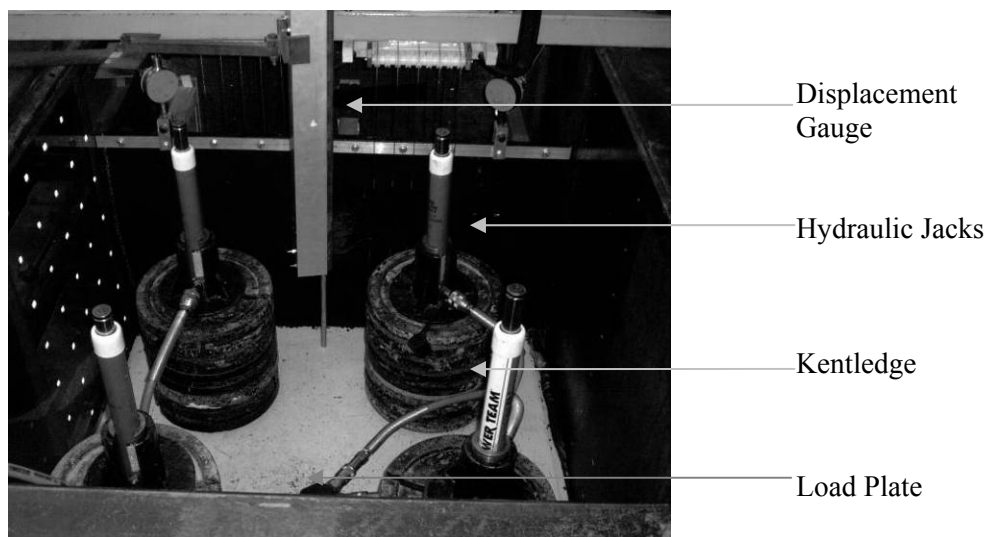


Figure 2 Photograph showing loading equipment

2.5 Instrumentation

2.5.1 Geosynthetic displacement

The relative displacement of five points on each of the geomembrane and geotextile were measured by attaching 1 mm diameter wires to the geosynthetics. The extensometer wires were attached at distances of 200 mm, 400 mm, 600 mm, 800 mm and 1000 mm from the base of each geosynthetic and were attached by passing them through a preformed hole in the geosynthetic and securing them with a brass swage to prevent deformation of the attachment and the geosynthetic. Although it is acknowledged that creating a hole in the geomembrane would not be acceptable on

site, this technique was used in the laboratory experiment as it created a smaller inclusion on the interface than the alternative if using a welded lug attachment. To prevent additional interaction with the geosynthetics, the wires were contained within brass tubing, 3 mm internal and 4 mm external diameter. This isolated the wires from the interface (i.e. so they could move freely) while providing adequate crushing resistance to avoid “pinching” of the wires. The wires were run, via pulley wheels, over displacement measuring boards (Figure 3) with each tensioned using a 200g static weight.

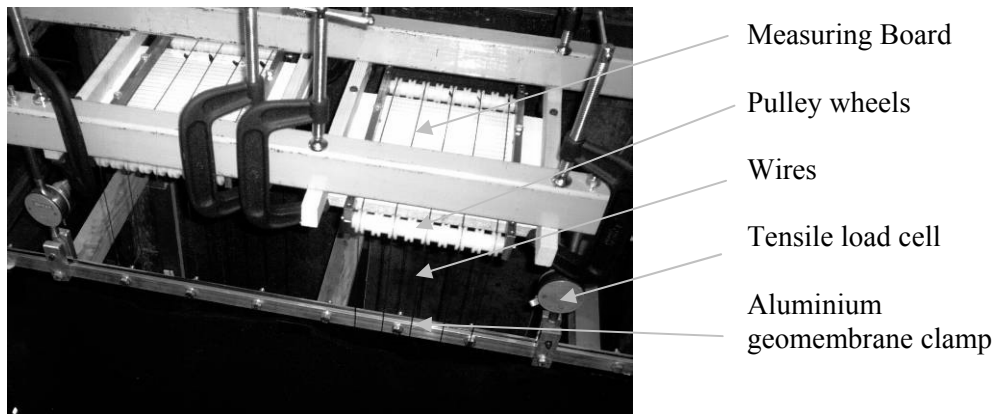


Figure 3 Geosynthetic instrumentation

2.5.2 Geomembrane tensile stress

The geomembrane samples were restrained in the vertical direction using a clamp attached to a fixed steel frame using two, 12 kN limit and 1 N resolution, tensile load cells (Figure 3). The cells were attached to the aluminium flat bar geomembrane clamp to through a two plane articulated joint to allow the cells to remain parallel to the load even if small movements in the clamp alignment occurred. No movement of the clamp arrangement was observed during the tests. A rigid frame, constructed from steel box section with welded joints, was assembled from which the tensile load cells were hung. Preliminary load tests were carried out with linear variable displacement transducer (LVDT) attached to ensure that deformations in the load cell support structure would not influence the readings.

2.5.3 Load plate displacement

An MTS Temposonics position sensor was used to measure the vertical displacement of the rigid loading plate. Vertical displacement of the load plate was measured a distance of 50 mm from the lining system, and located centrally (Figure 2). A four point loading system was used to retain the horizontal orientation of the loading plate.

2.5.4 Horizontal stress

In a separate series of tests, the horizontal stresses at the synthetic waste - lining system interface were measured using vibrating wire pressure cells. Two 400 mm x 400 mm cells were placed in the test chamber, one placed horizontally at a depth of 650 mm from the rigid and uncompressed lining support surface (350 mm from the base) and the other placed vertically, mounted in the lining support system, located with its centre coinciding with the plane of the horizontal cell. The loading sequence, of 5kPa kentledge and 7 x 10kPa hydraulic load increments, was applied, and readings taken from both pressure cells. To verify the readings taken from the cells, the test was then repeated with the two cells interchanged, and horizontal : vertical stress ratio calculated for each test and each cell. The results showed that the horizontal pressure at the interface increased linearly with increasing vertical pressure and the ratio of the horizontal to vertical pressure (K_0) was 0.55 (with a standard deviation of 0.013 for the 4 readings).

3.0 Interface shear strength

Interface shear strength involving geosynthetics can show considerable variability (Koerner & Koerner 2001, Stoewahse *et al.* 2002), therefore, material specific interface shear tests were carried out on the materials used in the laboratory investigation. Three types of interface were tested in a direct shear apparatus with a shear area of 300 x 300 mm designed specifically for measuring geosynthetic interface behaviour:

- Wood subgrade – Geomembrane: The geomembrane was clamped to the lower (moving) box, whilst the load was applied to the wood placed in the (stationary) upper box.

- Geomembrane – Geotextile: The geomembrane was clamped to the lower box and the geotextile was attached to the upper box. Load was applied to the geotextile through a 50 mm layer of synthetic waste.
- Geotextile – Synthetic waste: The geotextile was clamped to the lower box. The upper box contained a 50 mm thick layer of synthetic waste through which the load was applied.

Normal stresses of 10, 30 and 50 kPa were used to be representative of expected stresses acting normal to the lining system. Tests were carried out at a rate of 1 mm/minute to a displacement of 80 mm. The shear stress displacement curves were then used to assess the interface stiffness and strain dependant interface shear strength properties for each combination of multilayered lining elements. It should be acknowledged that in some cases a true residual value of shear strength was not reached at the 80 mm displacement achieved in the shear box.

Figures 4 and 5 show the direct shear results for lining systems involving LLDPE textured geomembranes Types G and S respectively. The synthetic waste – geotextile interface shear strength is the same in both cases, however, the interface shear strength of the Type G LLDPE geomembrane – geotextile interface is significantly higher than the Type S LLDPE geomembrane – geotextile interface. For lining systems involving mono textured HDPE geomembrane with the textured side up (Figure 6), the shear stress displacement curves are very similar to the Type G LLDPE geomembrane, as they both involve the same texturing type and are produced by the same manufacturer. For lining systems involving mono textured HDPE geomembrane with the smooth side up (Figure 7), the geotextile – geomembrane interface strength is clearly much lower than in the other cases. The presence of texturing has little effect on the smooth wood subgrade geomembrane interface as there are no appreciable asperities on the wood with which the texturing can interact. The peak and large displacement shear strengths are summarised in Table 2.

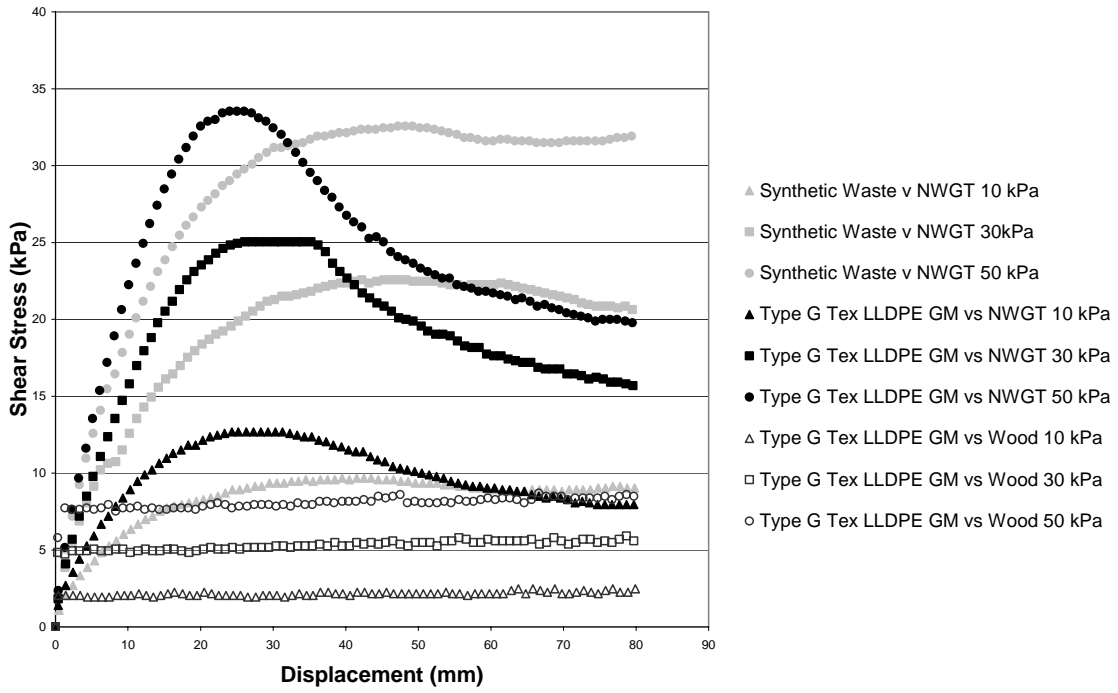


Figure 4 Shear stress displacement curves measured from direct shear tests (Type G textured LLDPE Geomembrane).

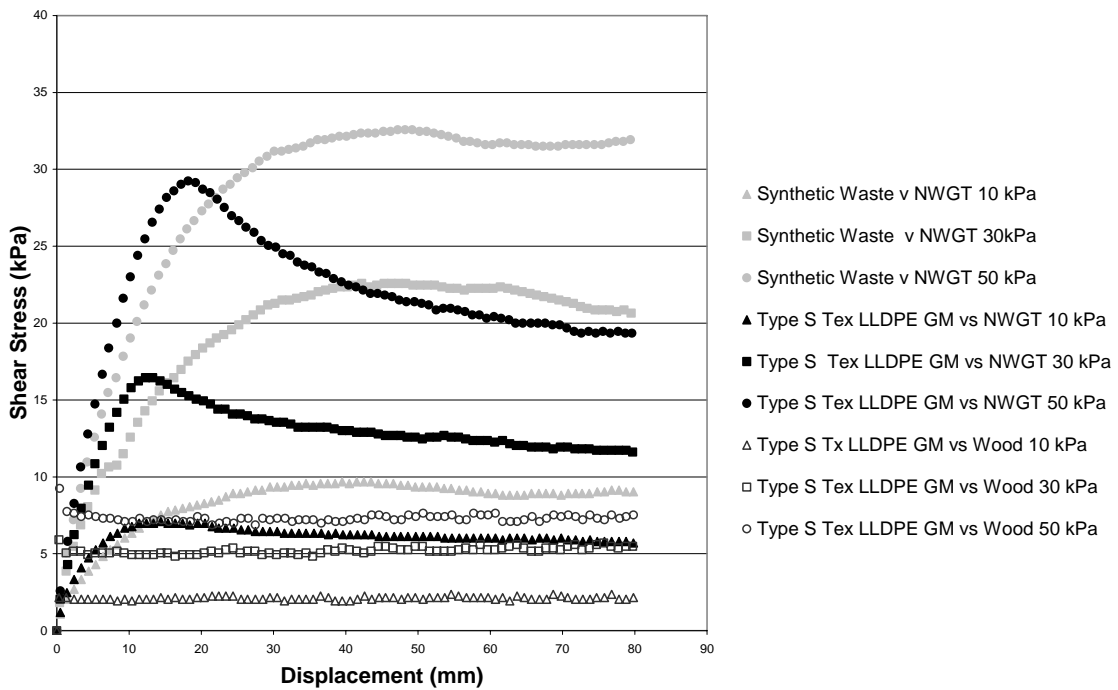


Figure 5 Shear stress displacement curves measured from direct shear tests (Type S textured LLDPE geomembrane).

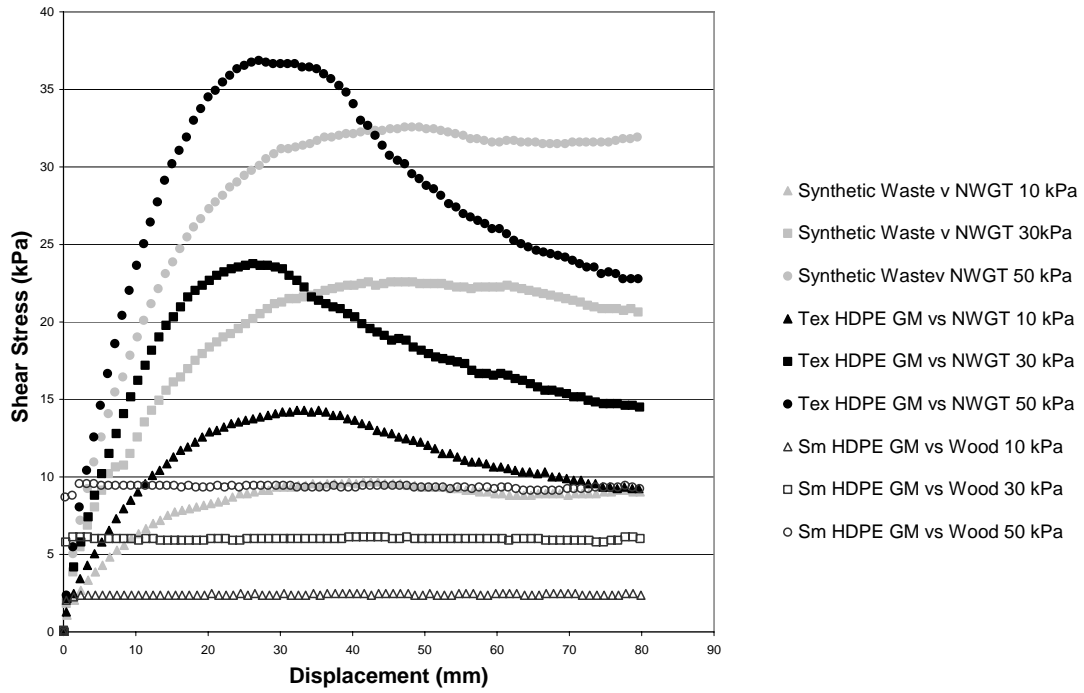


Figure 6 Shear stress displacement curves measured from direct shear tests (Mono textured HDPE geomembrane, textured side up).

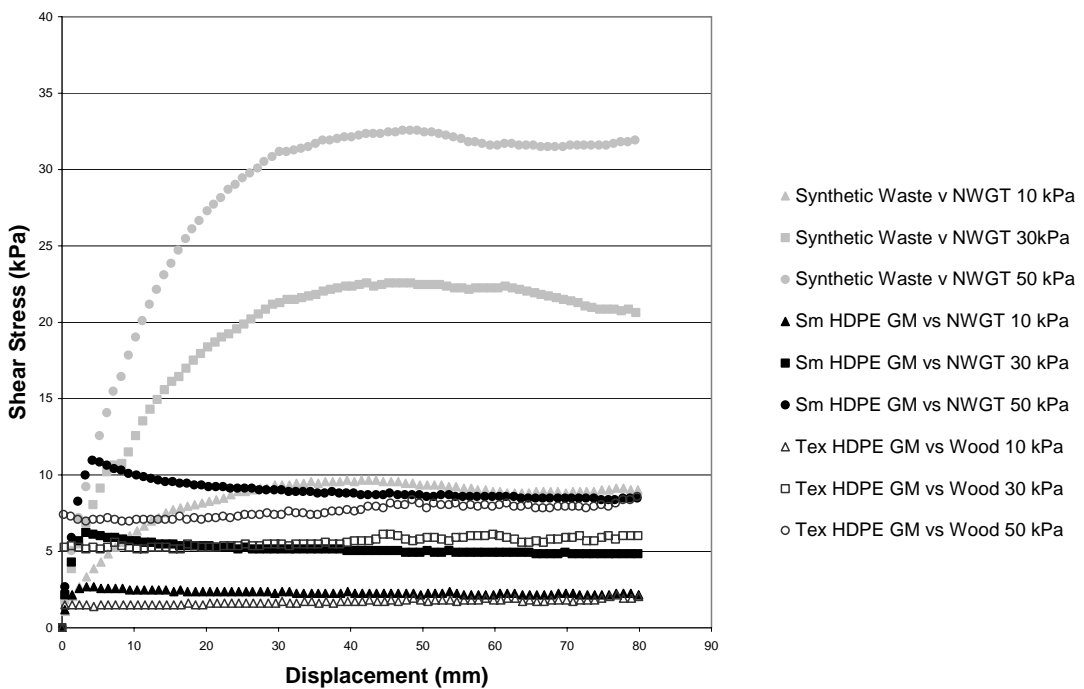


Figure 7 Shear stress displacement curves measured from direct shear tests (Mono textured HDPE geomembrane, smooth side up).

Table 2 Summary of peak interface shear strengths

Interface	α_{peak} (kPa)	δ_{peak} (°)	α_{LD} (kPa)	δ_{LD} (°)	Test
Synthetic Waste vs. Geotextile	4.4	29.9	3.3	29.8	All
Type G LLDPE GM vs. Geotextile	8.2	27.5	5.6	16.5	T2 & T5
Type G LLDPE GM vs. Wood	1.0	8.9	0.5	8.1	
Type S LLDPE GM vs. Geotextile	1.0	29.0	2.0	18.8	T6 & T9
Type S LLDPE GM vs. Wood	0.7	9.8	0.8	7.7	
Type G HDPE (tex) GM vs Geotextile	8.0	29.4	5.4	18.7	T3
Type G HDPE (smooth) GM vs Wood	0.8	10.1	0.5	10.2	
Type G HDPE (smooth) GM vs Geotextile	0.4	11.7	0.4	9.0	T4
Type G HDPE (tex) GM vs Wood	0.8	9.2	0.4	8.0	

LD = Large displacement

4.0 Numerical modelling

The finite difference numerical explicit modelling code FLAC (version 4.00) has been selected to analyse side slope lining systems primarily due to its ability to model large strains and previous experience using it to assess multilayered geosynthetic interfaces (Fowmes *et al.* 2005, 2006).

4.1 Modelling grid

The finite difference modelling grid used in the analysis consists of 3 zones of elements, representing the wood subgrade, the compressible synthetic waste and the relatively incompressible steel test chamber side. In the modelling grid, before deformation, each zone represents a 20 mm x 20 mm cube of material under plane strain conditions (Figure 8).

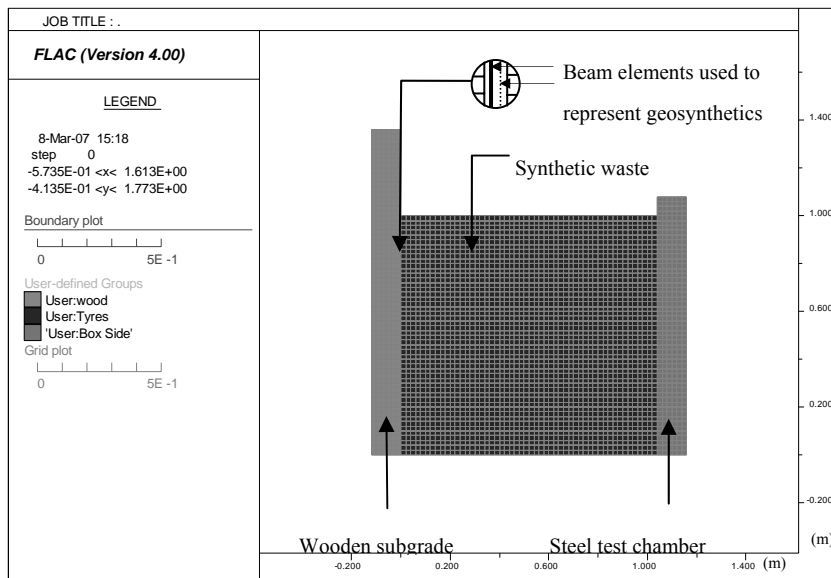


Figure 8 FLAC modelling grid used to assess model with 20 mm grid zones (prior to deformation).

4.2 Constitutive model for synthetic waste

The synthetic waste has been modelled using a linear elastic material model with a Mohr-Coulomb failure criterion, with shear strength parameters of 29.3° friction angle and 3kPa apparent cohesion. By necessity the numerical modeller must simplify the real world problem and in commercial modelling applications, particularly for municipal solid waste, it is common practice to use a linear-elastic constitutive model with a Mohr-Coulomb failure criterion. Hence it is considered appropriate to assess the validity of the modelling process using this commonly applied constitutive material model. A limitation of using a linear elastic Mohr-Coulomb constitutive model is that the volumetric strain hardening of the synthetic waste is simplified to a linear modulus. A model with coupled volumetric and shear behaviour which are interdependent may be more appropriate in the case of synthetic waste. Although with two modes of internal deformation, particle deformation and particle rearrangement occurring, the synthetic waste has complex behaviour that cannot be currently modelled using commercially available material constitutive models. Using data from confined compression testing of the synthetic waste, as described in section 2.2, a secant constrained modulus (at 75 kPa applied stress), was obtained and a Young's

modulus of 189 kPa was calculated. A Poisson's ratio of 0.25 was assumed to generate appropriate settlement and horizontal stress behaviour.

4.3 Modelling of geosynthetics

The geosynthetic elements have been represented in the model using structural beam elements (following Itasca, 2002). Multiple beam elements can be placed in the nulled region between two grid elements, in this case one grid representing the synthetic waste and the other grid representing the wooden subgrade. The beams only interact, with each other and with the grid, through interfaces, which control the interface shear and normal displacement characteristics.

The beams were modelled using a linear elastic law, however, code was written to allow for the material to have a lower modulus in compression than in tension. An arbitrary value of $E_{\text{Tension}} = 10 (E_{\text{Compression}})$ was adopted from experience to ensure that in compression the interface properties control the compression behaviour rather than a rigid beam resisting compression. This is similar to the approach adopted by Villard *et al.* (1999), who used a compressive stiffness of 20 and 10 times lower than the tensile stiffness for the geotextile and geomembrane respectively. Each beam element, prior to deformation, measured 20 mm in length.

Tensile strength data for the geomembranes and geotextiles was acquired from the manufacturers. The data for the LLDPE geomembranes included a 2% strain secant modulus which was considered appropriate for use in the investigations as preliminary calculations predicted strains of this magnitude. Data supplied for the HDPE geomembrane gave yield stress and strain values allowing a secant modulus at yield, however, this greatly underestimates the small strain (<2%) modulus. Giroud (1994) shows the 2% secant modulus of a HDPE to be over 3.5 times greater than a secant modulus at yield. It was thus decided to adopt a small strain tensile modulus (based on a 2% secant modulus), as this represented the appropriate magnitude of strain expected in the analysis. It is not recommended that 2% modulus values be adopted in design as this may result in overestimation of the material stiffness at strains in excess of 2%. In such cases, a conservative secant modulus at yield, or, if sufficient data is available, a strain dependant modulus may be adopted.

4.4 Interface modelling

Each interface represents, prior to deformation, a contact length of 20 mm. Interfaces are given normal and shear stiffness values, and interface shear strength. The normal stiffness is taken as an arbitrarily high value to maintain numerical stability and solution speed, whilst avoiding any appreciable interpenetration. Interpenetration of interfaces, where one side of the interface passes into the other and an overlap of grid zones occurs, can be a particular problem when modelling large strain problems across multilayer lining systems as the interpenetration is cumulative across all of the interfaces in the lining system.

The interface model uses initial stiffness values, measured from the direct interface shear tests, and displacement dependent limiting shear strength values (failure envelope). A user coded piecewise function is used to define the relationship between the failure envelope and the relative interface shear displacements. Input data for this function is derived from direct interface shear tests. Verification of this method is provided by comparing the relative shear displacements calculated by this code and the relative shear displacements (RSD) calculated manually from node and grid position data. The strength parameters were defined as a strain dependant law through piecewise friction angle against displacement and adhesion against displacement functions. The 80 mm shear strength values in the measured direct shear tests were taken as large displacement values which remained constant with further displacement. A simple Mohr-Coulomb linear elastic interface, with a friction angle of 7° , is included between the synthetic waste and test chamber side.

4.5 Modelling of load application

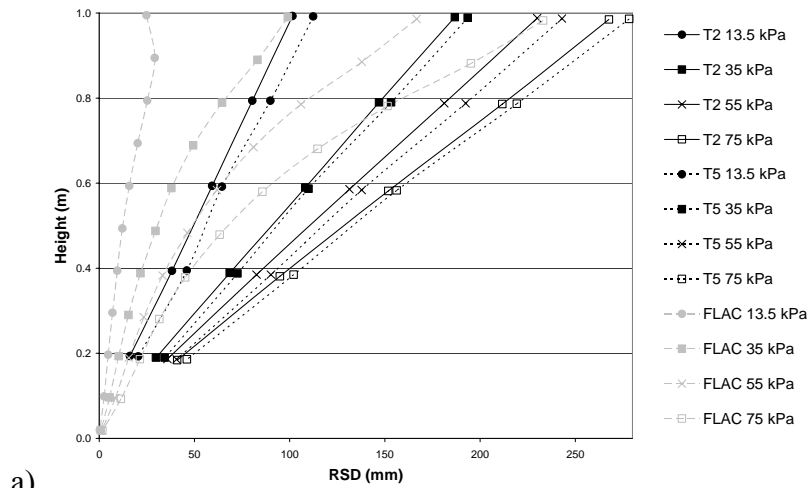
The load was applied using a vertical stress to the upper surface of the grid. It is acknowledged that this allows for some slight deformation in the upper surface of the synthetic waste, whereas in the laboratory experiment, the upper surface of the synthetic waste remains horizontal due to the rigid load plate. Attempts were made to model the rigid load plate; however, this resulted in numerical instability (calculation difficulties) induced by the large stiffness gradient between the load plate and the

synthetic waste. It was thus decided to apply the load directly to the upper surface of the synthetic waste.

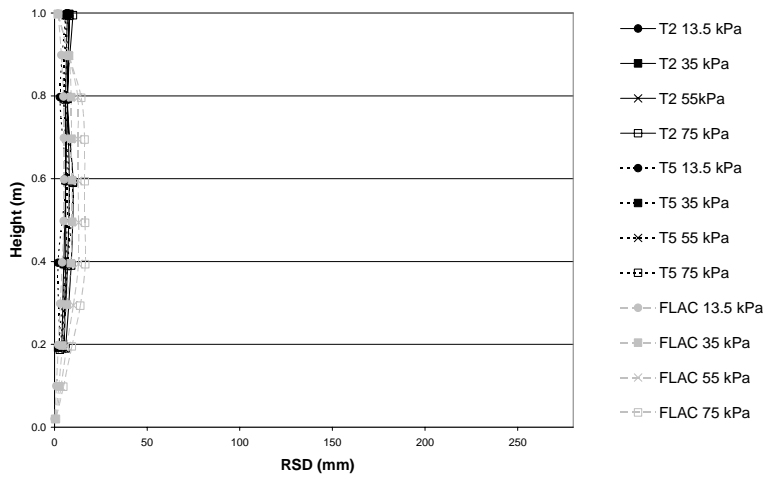
5.0 Results

5.1 Textured LLDPE geomembrane (Type G)

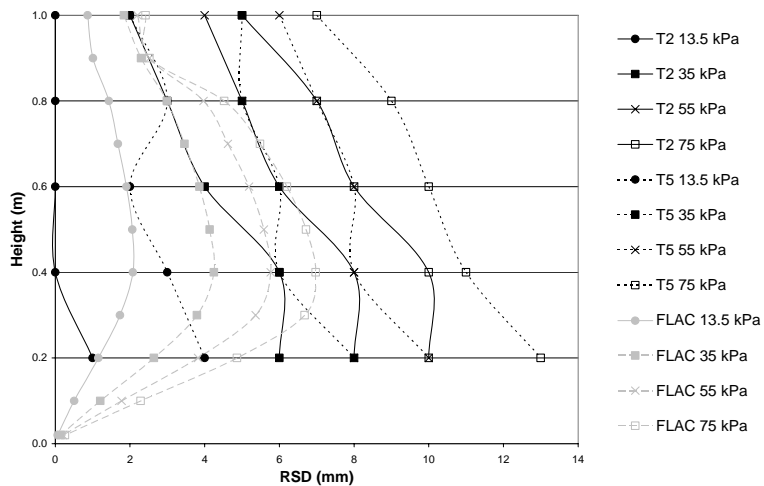
Two tests were carried out, T2 and T5, using Type G textured LLDPE geomembrane. The interface shear stress displacement curves for these tests are shown in Figure 4, whilst the results from the numerical and laboratory modelling are shown in Figure 9 as relative shear displacements at the interfaces. The two laboratory tests show similar results, indicating that behaviour of the test is repeatable. As the peak strength is higher for the geomembrane - geotextile interface than the geotextile – synthetic waste interface (Table 2), the largest relative shear displacements occur between the synthetic waste and geotextile and the displacements measured between the geotextile and geomembrane do not reach post peak values. The FLAC model showed a similar trend, although the constitutive model used did underestimate the displacement, between the synthetic waste and geotextile at depth in the synthetic waste profile.



a)



b)



c)

Figure 9 Relative interface shear displacements between a) synthetic waste vs. geotextile b) Type G textured LLDPE geomembrane vs. geotextile and c) Type G textured LLDPE geomembrane vs. wooden subgrade.

The FLAC model shows a similar trend to the laboratory model for the displacements between the geomembrane and the wood subgrade, however, the model underestimated the displacement (and hence extension of the geomembrane). Most of the discrepancy occurs at the upper sampling point, in the unconfined section of geomembrane above the level of the (undeformed) synthetic waste. It may be the case that the secant modulus used underestimates the tensile strength at the strains developed in this investigation. A comparison between the tensions developed at the geomembrane anchorage in all of the laboratory tests and in the numerical analysis is shown in Figure 10. The maximum values recorded in the laboratory tests are 3.49 and 3.47 kN/m, whilst the FLAC analysis gives a maximum value of 3.93 kN/m. This is considered to be an acceptable correlation.

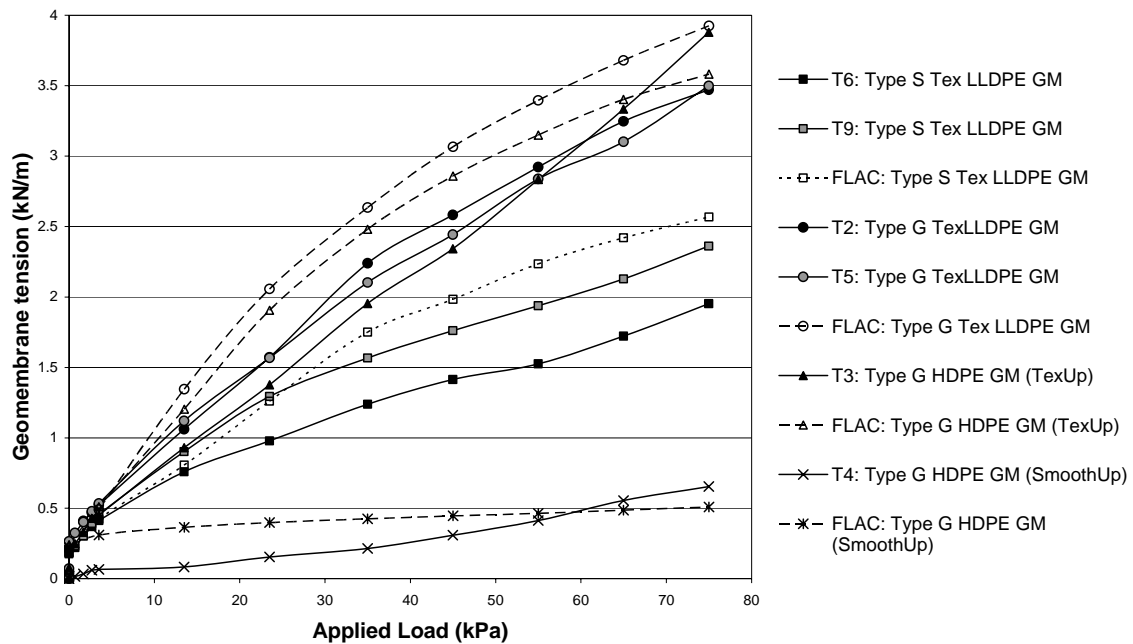


Figure 10 Tension in the geomembrane, at anchorage, from laboratory tests and FLAC models

5.2 Textured LLDPE geomembrane (Type S)

Unlike the Type G LLDPE geomembrane, which was delivered from the factory, the Type S LLDPE geomembrane was obtained from site and had some clay deposits on the surface. In the first test, T6, the clay was removed using a damp cloth, however, it

was subsequently thought that this may have damaged the texturing on the surface of the material, and as such a second test, T9, was carried out where the geomembrane was cleaned using a water jet.

The interface displacements show significantly greater displacements between the geomembrane and geotextile in T6 than in T9 (Figure 11) indicating that the cleaning process in T6 had damaged the texturing and lowering the interface shear strength. As a result the displacement between the geotextile and synthetic waste were lower in T6. The shear box tests used to derive the numerical input parameters were carried out on water cleaned geomembrane samples as in T9 and the results of the FLAC modelling shows good correlation to T9. The interim displacements between the geotextile and the synthetic waste are underestimated by the FLAC model and this is likely to be because the secant stiffness was chosen for the waste material rather than a volumetric strain hardening model.

The tension developed in the geomembrane during the test is shown in Figure 10. The tensile force at the geomembrane anchorage in T9 (2.36 kN/m) shows a good correlation with the FLAC model (2.57 kN/m), while a lower value is recorded for T6 (1.95 kN/m) due to the lower transferred stress across the geotextile – geomembrane interface. Greater tension was developed when testing the Type G geomembrane compared to Type S LLDPE due to the post peak strength reduction that occurred between the Type S LLDPE geomembrane and the geotextile which allowed increased displacement and lower shear stress transfer to the geomembrane.

5.3 Mono-textured HDPE geomembrane (textured side up) (Type G)

The displacements between the geomembrane – geotextile and the geotextile - synthetic waste interfaces (shown in Figure 12) are very similar to those shown in Figure 9 for Type G textured LLDPE geomembrane as they have the same texturing type and are from the same manufacturer, with the only difference being the polymer composition. The change in polymer type does reduce the displacement on the geomembrane – wood subgrade interface, due to the reduction in tensile strain in the geomembrane with the higher modulus HDPE geomembrane.

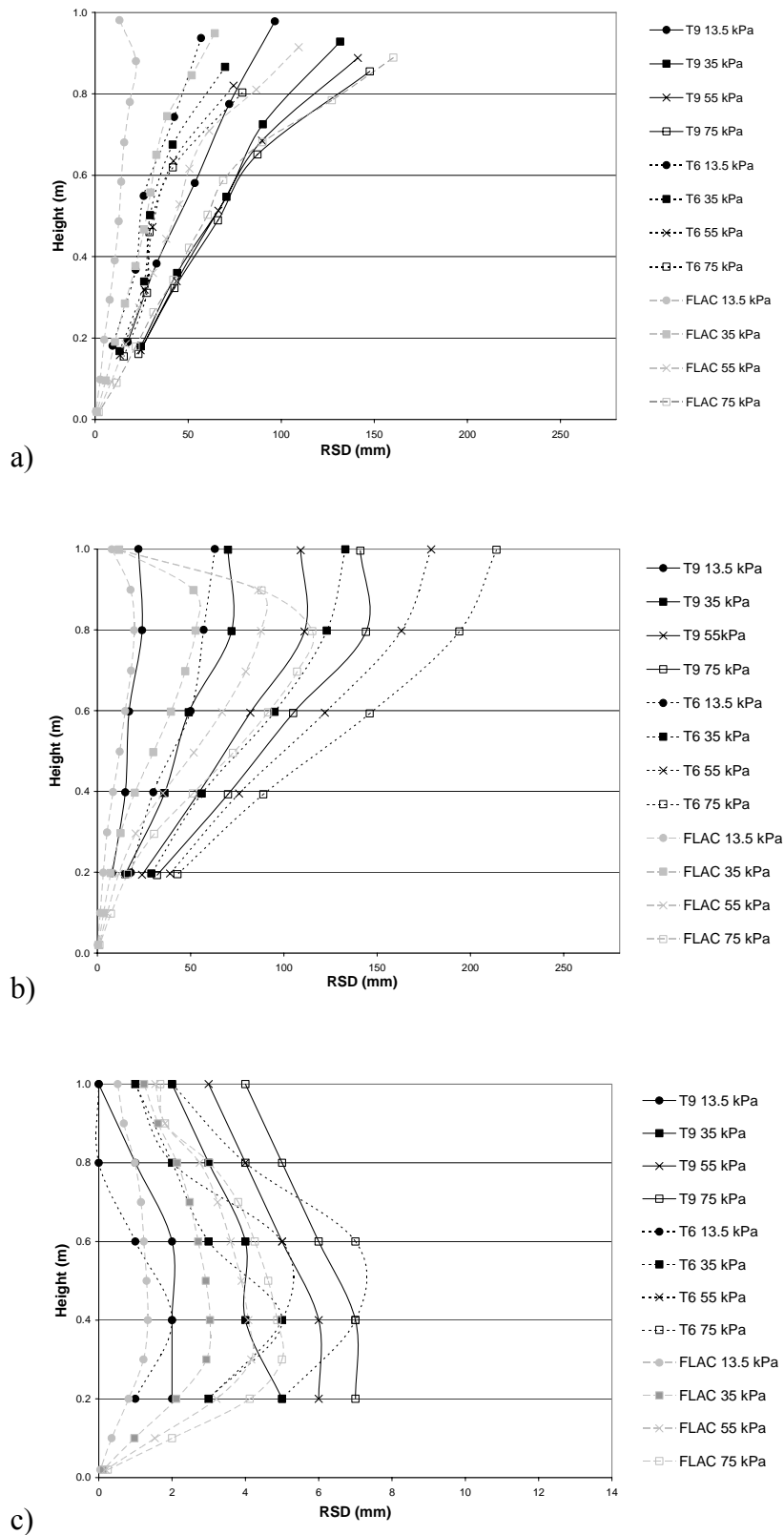


Figure 11 Relative interface shear displacements between a) synthetic waste vs. geotextile b) Type S textured LLDPE geomembrane vs. geotextile and c) Type S textured LLDPE geomembrane vs. wooden subgrade.

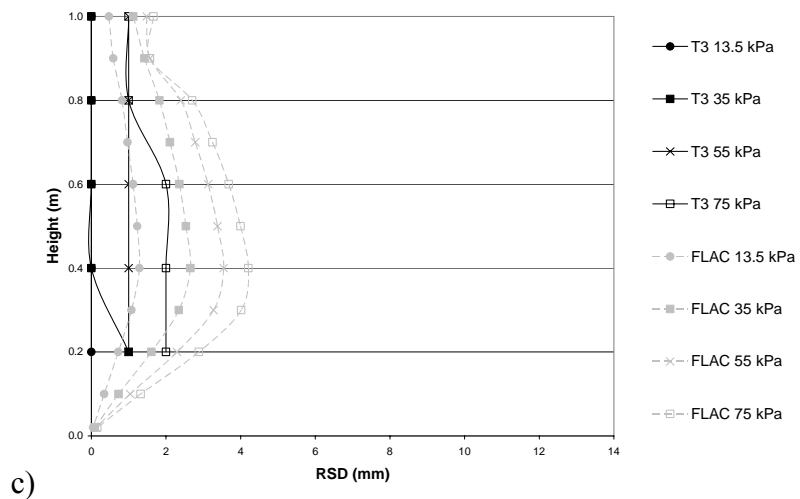
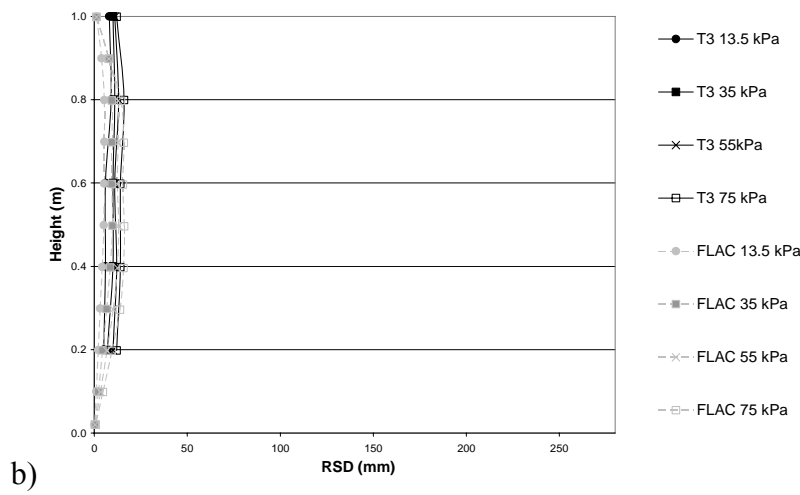
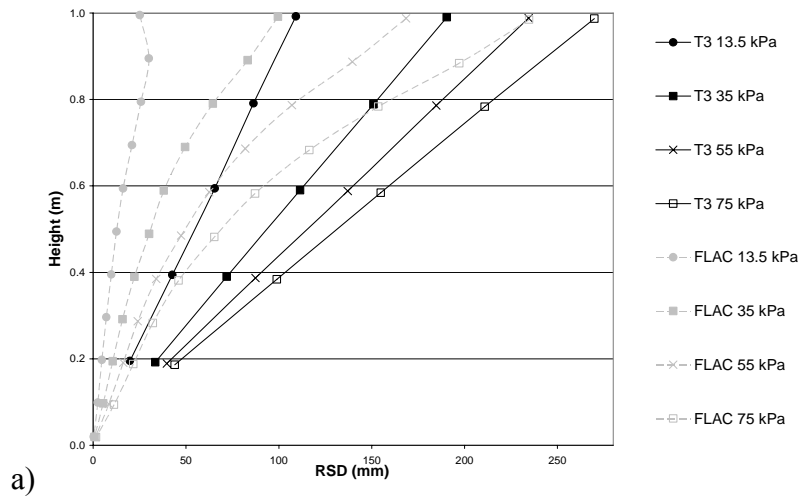


Figure 12 Relative interface shear displacements between a) synthetic waste vs. geotextile b) Type G textured HDPE geomembrane vs. geotextile and c) Type G smooth HDPE geomembrane vs. wooden subgrade.

A comparison between the measured and modelled tension at the geomembrane anchorage is shown in Figure 10. A good correlation is observed between the measured and modelled value. The results show a small increase in measured tension compared to the Type G LLDPE geomembrane, possibly due to greater stress relaxation with increased strain in the LLDPE geomembrane. The behaviour of the FLAC model is very similar in both cases as the interface properties for the two analyses have very similar input parameters (see Table 2)

5.4 Mono-textured HDPE geomembrane (smooth side up) (Type G)

A second test was carried out on mono-textured HDPE geomembrane, in this case with the geomembrane placed smooth side up (i.e. against the geotextile). The effect on interface friction between the geomembrane and the wood was small ($<1^\circ$ change in friction angle, see Table 2) However, there is a large reduction in interface shear strength between the geomembrane and the geotextile compared to the textured geomembrane – geotextile interfaces and this resulted in large (>200 mm) displacements along this interface (Figure 13). As a result, the displacements on the geotextile - synthetic waste interface were reduced, and as less stress was transferred to the geomembrane, the geomembrane strains and geomembrane – wood subgrade displacements, were small. The FLAC analysis shows the same trends that were observed in the laboratory experiment. Although the FLAC analysis predicts more displacement between the geotextile and the synthetic waste and slightly less between the geomembrane and geotextile, the trend with depth of the displacements are well matched, as are the geomembrane – wood subgrade interface displacements.

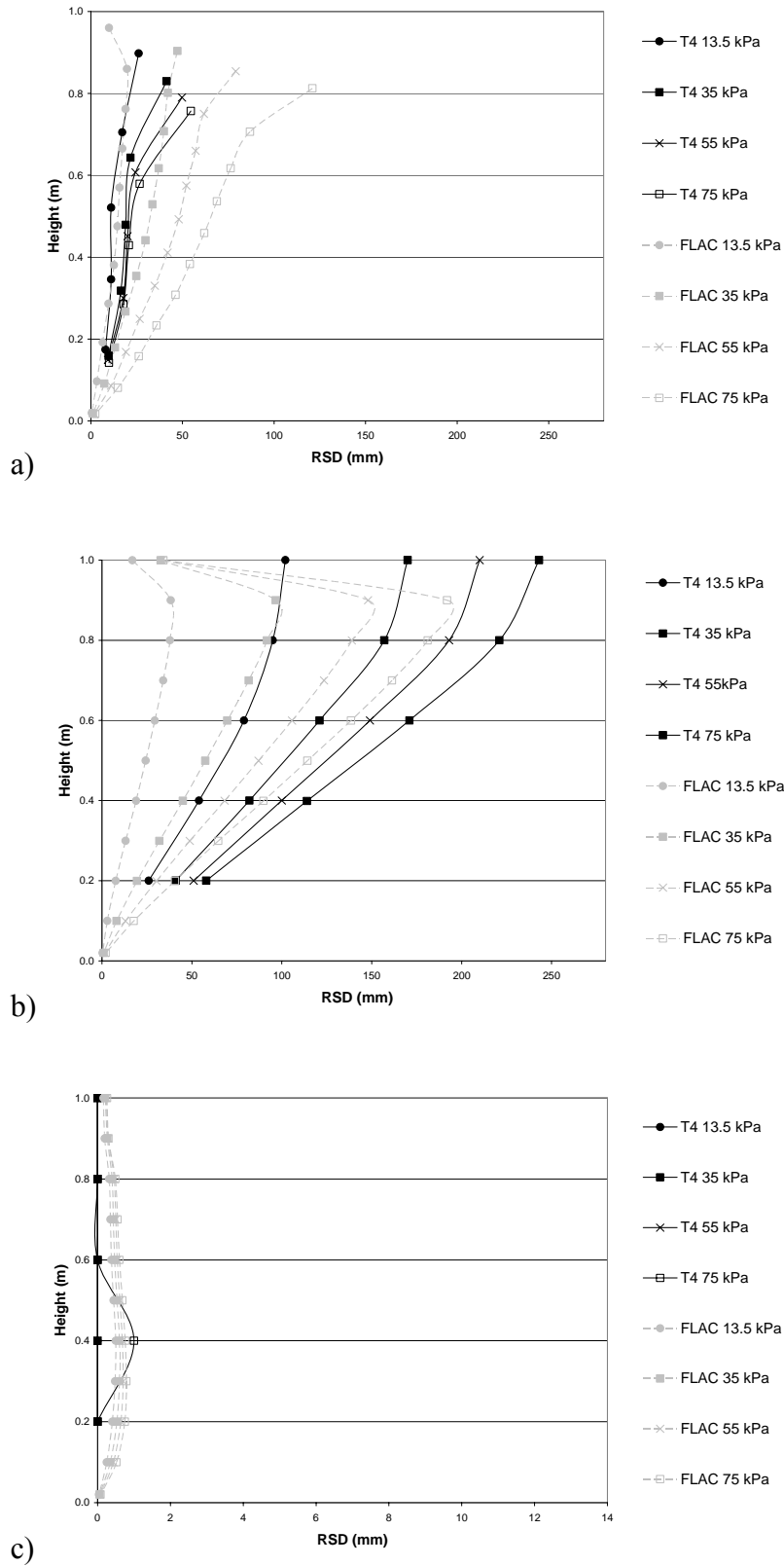


Figure 13 Relative interface shear displacements for a) synthetic waste vs. geotextile b) Type G smooth HDPE geomembrane vs. geotextile and c) Type G textured HDPE geomembrane vs. wooden subgrade.

The recorded maximum tensile force for T3 was 0.65 kN/m (Figure 10). This was much less than where a textured geomembrane – geotextile interface was present as the smooth geomembrane – geotextile interface results in lower stress transfer to the geomembrane. The FLAC model also shows the reduction in shear stress transferred into the geomembrane compared to textured membranes, although the shape of the curves with depth do not fit as well as for previous experiments. The constitutive model used for the synthetic waste may be responsible for this as a Mohr-Coulomb model generates sufficient horizontal stresses on the interfaces to transfer load into the lining system under self weight. However, at small loads the synthetic waste, which volumetrically hardens, does not generate sufficient shear stress in the lining system to cause tension in the geomembrane. Conversely, as the material stiffness increases with compression, there is increased stress transfer as shown at higher applied loads.

6.0 Discussion

The FLAC numerical modelling using multiple strain softening interfaces has been shown to reproduce the behaviour of a two layered geosynthetic lining system subjected to downdrag forces from a compressible material (i.e. synthetic waste body). Some discrepancies between the measured and the modelled results have been observed, however the general trends of displacement and strain magnitudes are represented by the numerical modelling.

The constitutive model used for the synthetic waste in this investigation is not able to reproduce the full observed behaviour of the synthetic waste in compression as it does not account for the volumetric strain hardening, and this may account for some of the discrepancies between the FLAC and laboratory models. However, it was considered appropriate to use the linear elastic Mohr-Coulomb model as it is the most commonly applied in commercial design and reliable input parameters could be readily obtained.

For the Type G textured LLDPE geomembrane and the Type G HDPE geomembrane, textured side up, the FLAC model shows less displacement than the physical model at the synthetic waste geotextile interface. The laboratory results assume displacement of the rubber crumb is a simple function of compression at the upper surface, and measured values within the geosynthetics on the other side, however, the FLAC

results show the relative positions of the grid and the beams (geosynthetics). The difference in these calculations is that the recorded values in the laboratory are omitting shearing of the synthetic waste, which accounts for the difference in the relative displacements compared to the FLAC predictions.

The representation of the geomembrane tensile load response is usually reported by manufacturers as stress and strain values at yield, and this may not be representative of stiffness behaviour at small strains, and can result in overestimation of predicted strain values. In this investigation 2% secant elastic moduli have been selected for the geomembranes and geotextile. This is still a simplification as stress strain response for geosynthetics is typically non linear and strain-rate dependent (Wesseloo *et al.*, 2004) Ideally, a representative geomembrane model would include the full stress strain behaviour, obtained from wide width tensile tests, however, this data was not available for use in this study and is not commonly available for design.

In all of the analyses, but particularly when analysing the mono-textured HDPE geomembrane with the smooth side up, the model must account for compression of the geotextile, and compression of the lower portions of the geomembrane. Geotextile behaviour under compression is very difficult to model, firstly because the geomembrane compressive modulus under confined conditions is difficult to measure and secondly due to buckling failure modes occurring that result in formation of folds under high compressive strains (Villard *et al.*, 1999). Geotextile folding is an extremely complex phenomenon to model numerically and although the modulus was reduced to account for reduced compressive stiffness when folds occur, modelling of actual folds is beyond the scope of continuum modelling techniques, particularly under commercially viable time scales. In the numerical modelling in this study, arbitrarily low compression stiffness was chosen so that compression behaviour was dependant on interface properties. However, at large compressions, where folding occurs this is still likely to underestimate the displacement of the geotextile. It is also observed in displaced samples that once folds form they tend to propagate at a point of focussed stress transfer into the geotextile. Figure 14 shows folding observed during the post test exhumation of Test T4, this resulted in greater geotextile compression in the lower 200 mm than the FLAC model predicted. The authors would suggest that designers consider the likely compressive strains in the system they are

analysing, and if large scale compression of a geosynthetics is predicted they allow for inaccuracies of the modelling procedure when assessing numerical model outputs and deriving design factors of safety.

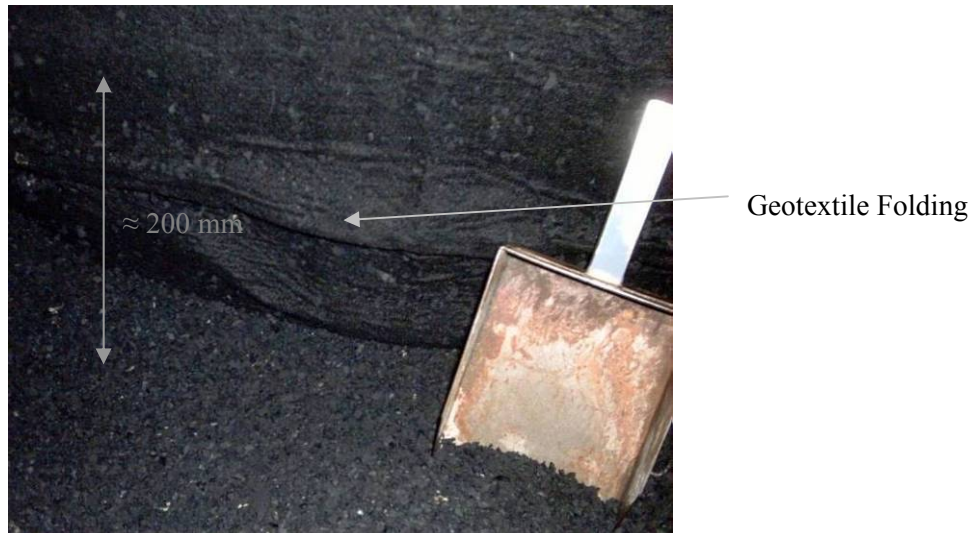


Figure 14 Geotextile folding in lower 200 mm (Test 4: HDPE Geomembrane: Smooth up)

The FLAC model takes into account the complexities of synthetic waste behaviour, geosynthetic stiffness and interface shear strength mobilisation and post peak shear strength reduction, which is not possible in limit equilibrium analysis. The tension in the geomembrane was predicted by the FLAC model with a good degree of accuracy especially where a textured geomembrane – geotextile interface was present (12% difference at 75 kN applied load, for the Type G LLDPE geomembrane, and 8% difference for the Type G HDPE geomembrane textured side up and for the Type S LLDPE geomembrane). For the mono-texture HDPE geomembrane (smooth side up), the model prediction was less accurate (27% difference at 75 kN applied load) which is believed to be due in part to the simplified modelling of the compressive behaviour of the geomembrane.

The measured shear stress vs. shear displacement behaviour of geosynthetic interfaces is known to exhibit natural variability (Dixon. *et al.* 2006, Criley & Saint John 1997). Between three and five direct shear tests were carried out in this study on each interface to determine the interface shear strength and stiffness characteristics. The interface shear strength values for the geomembrane – geotextile interface from the

testing have been compared to those values published by Dixon *et al.* (2006) and to an internal database that includes tests at low normal stresses. The values show good correlation, although it is interesting to note the difference in interface shear strength between the Type S LLDPE geomembrane and the Type G LLDPE geomembrane, (Table 2) despite the fact that both are 1 mm textured LLDPE geomembranes.

The peak interface shear strength for the Type G geomembrane – geotextile interface is greater than that for the geotextile – synthetic waste, hence interface post peak strength reduction, and associated large displacements do not occur on this interface. Despite a lower post peak strength at the Type G LLDPE geomembrane – geotextile interface than at the geotextile - synthetic waste interface, as post peak shear strength reduction does not occur, slip occurs at the interface with the weaker peak strength; the geotextile – synthetic waste interface. This agrees with Gilbert (2001) who states that the peak strengths are required to identify the location of slippage whilst the residual strengths are then needed to establish the residual strength of the system. The Type S geomembrane - geotextile peak interface shear strength is lower than the peak strength for the geotextile – synthetic waste interface, hence, post peak shear strength reduction occurs between the geotextile and geomembrane, with associated larger displacements. This highlights the importance of site specific interface shear strength testing. A designer selecting literature data for this scenario may greatly underestimate interface displacements, or geomembrane tension. It is acknowledged that even when the same tests are carried out by the same operator that interface shear strength variability occurs (Sia and Dixon, 2007) hence the measured strengths carried out in this investigation may have some variability from the actual interface strengths of the materials used in the model tests and this may account for some of the discrepancies between the measured and modelled test results.

The resolution of the displacement measuring equipment was limited to ± 0.5 mm, allowing a strain resolution of 0.5 % over a 200 mm gauge length. It is suggested that for further laboratory investigations that higher resolution displacement gauges would allow greater strain resolution and/or smaller gauge lengths. For field scale instrumentation it is suggested that the resolution used would be appropriate. The casing required to protect the wires would need to be revised for field scale experiments, as although the brass tubing was effective at laboratory scale, cost would

likely inhibit its use at field scale. Under sloping lining systems, where a component of the self weight of the waste mass is also on the lining system, crushing resistance of the wire casing would become more important.

7.0 Future work

The authors acknowledge that this investigation was to model the behaviour of a multilayered geosynthetic system subjected to downdrag forces and may not be representative of an actual landfill lining system. In particular, inclusion of a compacted clay liner underlying the lining system will make the model conditions representative of commonly used composite lining systems on shallow slopes but at the cost of significantly increased complexity. In order to further validate numerical models being used by designers and assess lining system behaviour, in addition to laboratory investigations on other material combinations, full scale field instrumentation of a landfill site should be carried out to assess model accuracy under real world conditions. The models reported here are focussed on waste like compression behaviour under self weight, and loading that would be applied by subsequent lifts of waste (i.e. only short-term construction, filling, behaviour is considered). It would be beneficial to measure lining system behaviour in response to degradation induced settlement, and the stress and stiffness changes induced by this.

Stress transfer through overlying drainage (i.e. gravel) and protection layers plays an important role in waste barrier interaction. It is suggested that future work be carried out to assess the stability and integrity of drainage layers, to determine the stress transfer through such layers and the effect of draining layer instability on behaviour of underlying geosynthetic lining systems.

Development of new geomembrane instrumentation techniques such as fibre optics and thin film pressure gauges would allow less intrusive measurements and a potential higher degree of precision at both laboratory and field scale. Development of instrumentation under controlled laboratory conditions as described herein would allow calibration and assessment of instrument performance and durability under loading conditions prior to being used in field applications. Further work to estimate

the effects of geosynthetic compression on numerical results should also be considered.

8.0 Conclusions

The paper has shown that laboratory scale behaviour of multilayered geosynthetic lining system subject to downdrag forces can be represented, to a reasonable degree of accuracy using the large strain FLAC finite difference modelling techniques incorporating strain softening interfaces. There are some discrepancies between the modelled values and the measured behaviour, which the authors believe are due to simplifications in modelling geosynthetic axial stress response (both tension and compression) and in the constitutive model used to represent the synthetic waste (rubber crumb). However, it is concluded that the modelling code and application methodology are appropriate.

The numerical model represents geosynthetic materials in tension with far greater precision than in compression, as compressive moduli of geosynthetics in confined conditions are not available and are very difficult to measure. The complexity, from a numerical analysis perspective, is greatly increased by the presence of folding in the geotextile, which occurs at large compressive strains. It is beyond the scope of the FLAC numerical modelling code to analyse this process or to predict where it may occur.

The use of numerical modelling techniques allows prediction of displacements, stresses and strains in multilayer geosynthetic lining systems with non linear interface behaviour. However, 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. The scope of this analysis was to assess the comparison between the laboratory model and the FLAC predictions and not to assess or predict the performance of a landfill lining system. Whilst it is believed that this project represents a significant step in the validation of the numerical model behaviour, full scale field instrumentation of a landfill site would allow for assessment of model accuracy under in service conditions.

9.0 Acknowledgements

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