Stability of Landfill Lining Systems: Report No. 1 Literature Review

R&D Technical Report P1-385/TR1

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This report has been produced to accompany R&D Technical Report P1-385/TR2 and should be used in conjunction with that report. This report provided the underpinning science to the design assessment framework provided in the 2nd report. Both these reports should be used by people who design, construct, operate and regulate landfill sites, however they should be used in conjunction with advice from a suitably experienced geotechnical engineer. Guidance will be produced as a result of this research. It is essential that any person carrying out an assessment of stability or integrity of a landfill lining system as part of a Pollution Prevention and Control Permit Application for a landfill should use the guidance as a foundation for that assessment. It is recommended that any persons assessing the ongoing stability of a landfill either as a review of a Waste Management Licence or as a result of a identified failure of the waste or liner, should consider the advice and guidance provided as a result of these reports.

Keywords
Landfill, Landfill Liner, Stability, Integrity, Geotechnical Assessment, Liner Failure, Engineering, Material Strength, Waste, Landfill Directive.

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EXECUTIVE SUMMARY

Consideration of landfill lining system stability is a fundamental part of the design of a landfill. It is a requirement of the EC Landfill Directive (1999) that is implemented through the Pollution Prevention and Control (PPC) permitting process. The stability of the waste mass, lining system and sub-grade should be ensured. Incorrect or incomplete assessment of stability has led to a number of failures both in the United Kingdom (UK) and overseas. The occurrence of failures, introduction of new materials and construction practices, developments of new design methods and ongoing changes in waste materials, together with the legislative need to remove the risk to human health and the environment have all contributed to the need for this review.

Design of landfills must include consideration of stability both within and between elements of the lining system, within the waste and also the sub-grade. This is to ensure that uncontrolled slippage of any of the elements does not occur. However, the design must also include consideration of the long-term integrity of the lining system. Stresses, and hence deformations, in both mineral and geosynthetic lining materials must be controlled to ensure preferential flow paths are not formed (e.g. shear zones in clay liners and tears in geomembranes). An assessment of integrity requires knowledge of the lining sub-grade behaviour (i.e. cut and fill slopes, cell base), consideration of interaction between elements of the lining system and an assessment of the influence of time dependent waste deformations (e.g. settlement). Use of traditional limit equilibrium stability methods cannot by themselves provide a full assessment of a lining system. Instability is taken to include failure by complete collapse and loss of integrity, therefore both are covered in this report.

This report provides information on case studies of failures and a review of international literature on landfill engineering practice, with particular reference to the stability and integrity of lining systems. It has been produced as part of the Environment Agency funded R&D Project P1-385: '*Assessment of the stability of landfill lining systems*'. From the literature review a series of limitations in current knowledge and current practice have been identified. The information gained in this literature review has been assimilated to produce guidance on the stability of landfill lining systems, and this is presented as Report No. 2 (ref: P1-385/TR2) and as a guidance note for implementation of the Landfill Directive.

CONTENTS

1. INTRODUCTION

This report provides information on case studies of failures and a review of international literature on landfill engineering practice, with particular reference to the stability and integrity of lining systems. It has been produced as part of the Environment Agency funded R&D Project P1-385: '*Assessment of the stability of landfill lining systems*'. A second report aims to provide recommendations on 'best practice' for the design and operation of stable landfills and landfill liners.

Consideration of landfill lining system stability is a fundamental part of the design and regulatory processes required for implementation of the EC Landfill Directive (1999) requirements through the PPC permitting process. Annex 1, Section 6 of the Directive states:

> *"The emplacement of waste on the site shall take place in such a way as to ensure stability of the mass of waste and associated structures, particularly in respect of slippages. Where an artificial barrier is established it must be ascertained that the geological substratum, considering the morphology of the landfill, is sufficiently stable to prevent settlement that may cause damage to the barrier."*

The stability of the waste mass, lining system and sub-grade should be ensured. Incorrect or incomplete assessment of stability has led to a number of failures both in the United Kingdom (UK) and overseas. The occurrence of failures, introduction of new materials and construction practices, developments of new design methods and ongoing changes in waste materials, together with the legislative need to remove the risk to human health and the environment have all contributed to the need for this review.

1.1 Lining Systems

All types of lining systems currently used in the UK are considered in this report. Barrier materials included are mineral (e.g. compacted clay and bentonite enriched soil), geosynthetic (e.g. geomembrane and geosynthetic clay liners) and asphaltic (e.g. dense asphaltic concrete). Materials for drainage, protection and reinforcement are also included (e.g. sand, gravel, geotextiles, geocomposites and geonets). A range of issues relevant to stability assessment of all forms of landfill is considered (e.g. shallow and steep side slope configurations).

The aim is to ensure that the design performance criteria of the barrier (i.e. in terms of leachate and gas flow) are not adversely affected during the life of the landfill by deformation of the lining system. Consideration must be given to performance during construction (i.e. pre-waste placement) and in the long-term during and following waste degradation.

1.2 Stability and Integrity

Design of landfills must include consideration of stability both within and between elements of the lining system, within the waste and involving the sub-grade. This is to ensure that uncontrolled slippage of any of the elements does not occur. However, the design must also consider the long-term integrity of the lining system. Stresses, and hence deformations, in both mineral and geosynthetic lining materials must be controlled to ensure preferential flow paths are not formed (e.g. shear zones in clay liners and tears in geomembranes). An assessment of integrity requires knowledge of the lining sub-grade behaviour (i.e. cut and fill slopes, cell base), consideration of interaction between elements of the lining system and an assessment of the influence of time dependent waste deformations (e.g. settlement). Use of traditional limit equilibrium stability methods cannot by themselves provide a full assessment of a lining system. Instability is taken to include failure by complete collapse and loss of integrity, therefore both are covered in this report.

1.3 Outline of Report

The information presented in this report is divided into 12 main chapters. A brief summary of each chapter is provided below.

Chapter 2 details the sources accessed as part of the review of international literature. It includes lists of academic journals and specialist conferences covering issues relevant to landfill engineering. Chapter 3 provides a brief overview of the legislative framework governing landfills and of current landfill engineering practice. A review of UK landfill failures is presented in Chapter 4. Details are provided of a questionnaire survey developed to assess the types and frequency of failures. The questionnaire was circulated to Agency staff who provided information of failures contained within the public register. The main part of the chapter is seven case histories. These have been selected to provide information on common factors contributing to failure. A summary of the site conditions and lining system has been provided for each case history. The mechanism of failure has been described, as have any remedial measures undertaken. Where possible, conclusions and recommendations have been made. The key issues raised by each failure, and lessons to be learnt, are summarised at the end of the chapter.

A brief introduction to commonly used lining systems and their components is given in Chapter 5. Chapter 6 provides information on the properties of lining system components that influence stability and integrity. Methods of measurement are discussed, with references given to relevant test standards. Typical values are provided when possible. Issues relating to the measurement of interface shear strengths between lining components are covered in Chapter 7. Methods of measurement, factors controlling measured values, likely variability and the selection of values for use in design are all discussed. Chapter 8 provides information on the engineering properties of waste that are required to assess aspects of lining stability and integrity. Challenges of assessing waste are discussed and typical ranges of values for the main parameters are given, when known. The chapter includes sections on unit weight; vertical compression (i.e. controlling settlement); shear strength; lateral stiffness; and horizontal in situ stress.

Guidance is provided on assessment of sub-grade stability in Chapter 9. Factors controlling stability of cut slopes in hard rock, cohesive soils and granular soils are discussed, as are those influencing the stability of fill slopes in both cohesive and granular soils. Stability of natural slopes is considered. Basal stability issues such as excessive settlements, base heave and filling on waste are also included. Methods for designing and assessing the performance of basal lining systems are outlined in Chapter 10 with reference to the mechanisms introduced in Chapter 9.

Design and performance of shallow slope lining systems are covered in Chapter 11. Stability and integrity issues for lining systems both unconfined (pre-waste placement) and confined (post-waste placement) are discussed in detail. Methods of analysis are introduced and specific issues related to mineral and geosynthetic liners are covered. The role of waste and sub-grade in stability are highlighted. Chapter 12 covers stability and integrity issues for steep slope lining systems. The importance of construction is highlighted and the role played by waste/lining system interaction is considered in detail. A brief discussion of failures that occur entirely within the waste body is provided in Chapter 13.

2. REVIEW PROCESS

2.1 Introduction

This report presents the findings of an extensive literature review carried out to establish the current state of knowledge with respect to the stability of landfill lining systems. This chapter briefly summarises the main sources of information that have been used in this study, and readers interested in finding further information on associated topics are encouraged to consult the journals/conference proceedings listed.

The technical information used in this report comes from a number of sources including standard textbooks, journals, conference and symposia proceedings and Golder Associates/Loughborough University internal reports.

2.2 Journals

As part of this project, the journals listed in Table 2.1 have been reviewed back to at least 1995.

Table 2.1 Summary of journal sources used

Journal	Publisher	Frequency
Geotechnique	Thomas Telford	Quarterly
Geotechnical Engineering	Thomas Telford	6 issues/year
Canadian Geotechnical Journal	National Research Council of Canada	6 issues/year
Geotechnical and Geoenvironmental Engineering	American Society of Civil Engineers	12 issues/year
Quarterly Journal of Engineering Geology and Hydrogeology	Geological Society	Quarterly
Engineering Geology	Elsevier	Quarterly
Geotextiles & Geomembranes	Elsevier	Quarterly
Geosynthetic International	International Geosynthetics Society	12 issue/year
International Journal for Numerical and Analytical Methods in Geomechanics	Wiley Interscience	15 issues/year

2.3 Proceedings: Waste Management and Landfill

Technical papers from the proceedings of recent conferences and symposia on waste management and landfill have been reviewed as detailed in Table 2.2.

Table 2.2 Summary of waste management and landfill conferences and symposia sources used

Conference	Year	Location
Eighth International Waste Management and Landfill Symposium	2001	Cagliari
Seventh International Waste Management and Landfill Symposium	1999	Cagliari
Sixth International Landfill Symposium	1997	Cagliari
Fifth International Landfill Symposium	1995	Cagliari
Fourth International Landfill Symposium	1993	Cagliari
Third International Landfill Symposium	1991	Cagliari
Second International Landfill Symposium	1989	Cagliari
First International Landfill Symposium	1987	Cagliari
6th Annual Solid Waste Association of North America (SWANA) Landfill Symposium	2001	San Diego
5th Annual SWANA Landfill Symposium	2000	Austin
4th Annual SWANA Landfill Symposium	1999	Denver
3rd Annual SWANA Landfill Symposium	1998	Palm Beach Gardens
2nd Annual SWANA Landfill Symposium	1997	Sacramento
1st Annual SWANA Landfill Symposium	1996	Delaware
Wastecon – SWANA International Conference	2001	Baltimore
Wastecon – SWANA International Conference	2000	Capetown
Engineering Geology of Waste Disposal, Engineering Group of the Geological Society	1993	Cardiff

2.4 Proceedings: Environmental Geotechnics

Technical papers from the proceedings of recent conferences and symposia on environmental geotechnics have been reviewed as detailed in Table 2.3.

Table 2.3 Summary of environmental geotechnics conferences and symposia sources used

Conference	Year	Location
GeoEng 2000	2000	Melbourne
3rd International Congress on Environmental Geotechnics	1998	Lisbon
2nd International Congress on Environmental Geotechnics	1996	Osaka
1st International Congress on Environmental Geotechnics	1994	Edmonton
3rd British Geotechnical Society Geoenvironmental Engineering Conference	2001	Edinburgh
2nd British Geotechnical Society Geoenvironmental Engineering Conference	1999	London
1st British Geotechnical Society Geoenvironmental Engineering Conference	1997	Cardiff
14th International Conference on Soil Mechanics and Foundation Engineering (ICSMFE)	1997	Hamburg
13th International Conference on Soil Mechanics and Foundation Engineering (ICSMFE)	1994	New Delhi
Green 3: 3rd International Symposium on Geotechnics related to the European Environment	2000	Berlin
Green 2: 2nd International Symposium on Geotechnics related to the European Environment	1997	Krakow, Poland
Green '93 1st International Symposium on Geotechnics related to the European Environment	1993	Bolton, UK

2.5 Proceedings: Geosynthetics

Technical papers from the proceedings of recent conferences and symposia on geosynthetics have been reviewed as detailed in Table 2.4.

Table 2.4 Summary of geosynthetics conferences and symposia sources used

Conference	Year	Location
2nd European Geosynthetics Conference	2000	Bologna
1st European Geosynthetics Conference	1996	Maastricht
6th International Conference on Geosynthetics	1998	Atlanta
5th International Conference on Geotextiles, Geomembranes and Related Products	1994	Singapore
4th International Conference on Geotextiles, Geomembranes and Related Products	1990	The Hague
3rd International Conference on Geotextiles, Geomembranes and Related Product	1986	Vienna
2nd International Conference on Geotextiles.	1982	Las Vegas
1st International Conference on Geotextiles	1977	Paris
8th North American Geosynthetics Society Conference	2001	Portland
7th North America Geosynthetics Society Conference	1999	Boston
6th North America Geosynthetics Society Conference	1997	Long Beach
5th North America Geosynthetics Society Conference	1995	Nashville
4th North America Geosynthetics Society Conference	1993	Vancouver
3rd North America Geosynthetics Society Conference	1991	Atlanta
2nd North America Geosynthetics Society Conference	1989	San Diego
1st North American Geosynthetics Society Conference	1987	New Orleans

2.6 References in Text

Specific academic papers, specialist reports and textbooks are referenced in the text. They are used to highlight ideas and results that come from the research work of others, provide the reader with information on where more detailed descriptions of work can be found and to provide guidance on where the reader can find texts covering background information and theory. All references cited in the text are listed in full in the Reference section at the back of the report.

3. LEGISLATIVE FRAMEWORK AND CURRENT PRACTICE IN LANDFILL ENGINEERING

This chapter introduces the legislative framework and current best practice in landfill engineering. It is not an exhaustive account but gives an overview of relevant areas.

3.1 Legislation Summary and Background

3.1.1 Historical development

Historically, the selection and development of landfill sites in England and Wales has been principally subject to control under two types of statutory legislation. Firstly, the various Town & Country Planning Acts which chiefly seek to control the land use and/or the restoration process and secondly, various acts that have introduced a licensing or pollution control aspect.

The application of planning controls have been centred on assessing whether the proposed development is a suitable use for the land in question. This originally considered the impact on the development would have on the surrounding. However with the introduction of the Environmental Impact Assessment Regulations (SI 1999 No 293), planning authorities were required to consider the impact of certain activities (including landfill sites) on the surrounding environment. This was achieved through the close consultation of the planning authority with the waste regulation authorities and other statutory consultees (e.g. the Water Authorities which lead to the formation of the National Rivers Authority).

The Control of Pollution Act 1974 (COPA) was the first piece of legislation that introduced the requirement for landfill sites to be licensed. The local authorities were the regulatory authorities for this regime. Licenses required operators of landfills to control the escape of landfill gas and discharges of water from the site, and a licence could be refused in the event that such pollution was likely.

Prior to this Act and indeed for many years after, landfill engineering consisted solely of access and infrastructure, with all initial sites being licensed as dilute and disperse with no containment engineering for pollution control.

However the legislation did introduce and promote a holistic, risk assessment approach and was instrumental in raising awareness of the pollution potential of landfill sites. By the late 1970’s and early 1980’s the water authorities, and the National Rivers Authority (NRA) from 1989 (a statutory consultee under both planning and licensing), was beginning to say that in certain locations, where risks of groundwater pollution were present, then containment was required. This was initially achieved through the introduction of a (sometimes engineered) clay liner.

The publication of Waste Management Paper 26 in 1986 by the then Department of the Environment, although not statutory guidance, was instrumental in setting initial standards for landfill engineering. It describes all the main elements of engineering that we recognise today, including:

- bulk earthworks;
- underdrainage;
- liners - mineral and synthetic;
- leachate management; and
- monitoring.

However, it also discussed the use of permeable liners to provide a zone of attenuation beneath a landfill. It is also worth noting that at that time some licences were still being issued for dilute and disperse facilities.

Firstly, the introduction of the waste management licensing regulations (WML) 1994 and, secondly, enacting the 1995 Environment Act, brought about the creation in 1996 of the Environment Agency. This brought together both the NRA consultation function and the local authority licensing function. In particular, Regulation 15 sought to enact the EEC directive of 1979 on the Protection of Groundwater Against Pollution Caused by Certain Dangerous Substances (80/68/EEC) prohibiting discharge of certain (List 1) substances and limit discharges of other (List 2) substances.

Under Regulation 15, prior investigation and requisite surveillance which includes a hydrogeological risk assessment for the facility were requirements of a landfill licence. This site specific risk assessment approach has characterised the UK's approach to environmental protection at landfills initiated in the COPA. The likelihood of finding a landfill site with no engineered lining that demonstrates no acceptable discharge from the site is slim. Thus the de facto requirement to line landfills was formalised through a hydrogeological risk assessment introduced to meet an EC Directive on groundwater.

In 1995 the Department of the Environment updated its guidance on Landfill Design and Construction in Waste Management Paper 26B. Again, the risk assessment process was used to determine the need for landfill containment and to assist in the derivation of the performance requirements for the landfill lining system.

Another fundamental change that occurred through implementation of the Waste Management Licensing Regulations (1994), was the introduction of the surrender provisions (including financial provisions). This meant that the waste management licence holder could only be relieved of the legal requirements of that licence once it had been demonstrated that the site was unlikely to cause pollution if left without active controls. This meant that the licence holder could not walk away from the potentially polluting landfill site. This also introduced an expectation that lining systems would have to remain in place for substantial periods of time.

3.1.2 Current legislation

The introduction of the Pollution Prevention and Control (PPC) regime through the 1999 Act has changed the environmental permitting of industrial activities including landfill, that have a pollution potential. The principle of PPC is through the application of Best Available Techniques (BAT), which sets out standards of operation that is acceptable on an industrial sector basis. Implementation of the Landfill Directive via the Landfill Regulations (2002) is considered to constitute BAT for landfilling. All new PPC permits for landfills have to comply with the technical measures outlined within Annex I of the Directive. Each operational landfill will have to apply for a Landfill Directive PPC Permit on a phased basis between 17 July 2002 through to 31 March 2007. Site conditioning plans will inform the phasing of the permitting of these sites.

Schedule 2 of the Landfill (England and Wales) Regulations 2002 sets out the containment measures required at landfill sites. These set out some minimum requirements, including the following:

- siting of landfills where a geological barrier is provided;
- provision of a two component lining system;
- no unacceptable discharge from the waste throughout is lifetime; and
- construction of a barrier in a manner to ensure that it remains stable.

In setting out minimum requirement within the Regulations, it is expected that the actual construction of the barrier will be designed on the basis of risk to the surrounding environment both from release of leachate to groundwater and surface waters and the release of landfill gas.

At the time of writing this report the Regulations have only just come into force. As a result the full detailed guidance supporting this regime has not been completed. To keep abreast of the guidance as it is published reference should be made to the Environment Agency web site at www.environment-agency.gov.uk.

The groundwater regulations for the England and Wales came into force in April 1990 to implement the Groundwater Directive (EEC 80/68) for activities other than landfills permitted under the Waste Management Licensing regime. A permit under the PPC regulations will be subject to a requirement for "prior investigation", the EA require that a Hydrogeological Risk Assessment be undertaken as part of this 'prior investigation'.

However the above draft guidance states that if the applicant can demonstrate, via a risk assessment, that an alternative design will not lead to an unacceptable discharge of leachate or gas, then this is justification for not employing the prescribed measures detailed in Annex I of the Landfill Directive.

3.2 Current Practice in Landfill Engineering

Current practice is widely accepted in its diversity and application. It covers application from simple engineered clay liners to complex double composite liners that incorporate leak detection and collection.

Technology has progressed in recent years and new aspects include:

- greater understanding of the issues involved in designing steep slope lining systems;
- greater understanding of geomembranes and bentonite enriched soils (BES);
- acceptance that the "Christmas Tree" clay liner may be unstable; and
- introduction of new lining materials such as dense asphaltic concrete (DAC).

The Environment Agency has produced a number of internal guidance documents relevant to lining performance that are used extensively within the UK waste management industry. To date, the following have been issued:

- guidance on the use of geomembranes in landfill engineering;
- guidance on non-woven protector geotextiles for landfill engineering;
- guidance on the use of geosynthetic clay liners in landfill engineering;
- a methodology for cylinder testing of protectors for geomembranes;
- guidance on bentonite enriched soils for landfill engineering; and
- earthworks on landfill sites.

Other guidance documents are currently being drafted. Although these documents are not specifications produced by the Agency, they are regarded as forming the basis for current best practice for the development of site specific Construction Quality Assurance (CQA) Plans.

Under the current system, CQA Plans are submitted to the Agency for acceptance prior to construction of the cell on site. The CQA Plan will indicate the quality assurance measures to be put in place to ensure the compliance of the works to the standards and specifications of the PPC permit. Independent third party CQA is then required on site to ensure that the measures in the CQA Plan are implemented. The results of the testing undertaken together with observations made by the CQA engineer during the construction programme are then submitted to the Agency in a CQA Report. Validation of the information submitted to the Agency is required before any waste may be placed within the cell to which it relates.

4. REVIEW OF UK FAILURE CASE STUDIES

4.1 Approach

The study of past failures is an important tool for gaining knowledge of the key factors that control the performance of geotechnical structures. Lessons can be learned regarding the relative importance of assumptions, parameters and methods used in design, and on the controlling influence of site conditions and construction processes. There is significant anecdotal evidence regarding failure of UK landfill lining systems. However, there is little if any information on UK landfill failures published in the literature. To date there has been no systematic monitoring of liner structural performance. The emphasis of previous guidance has been placed on monitoring leachate, landfill gas and groundwater quality.

The aim of this chapter is to provide information on failure modes and the main factors that control failure, and to identify common trends and lessons to be learned. Case studies have been selected to represent the range of observed problems. The information contained in this section has come predominantly from public records held by the Environment Agency, in relation to the waste management sites it has licensed. In many cases the primary sources are records made during site inspections by EA officials, correspondence between the EA and the operator/designer and from consultant reports on failures.

This chapter is divided into three main sections. The first (Section 4.2) details the results of a survey of EA staff carried out to try and assess the frequency of lining system failures, obtain information on typical modes and causes, and to obtain any national and regional trends. The second (Section 4.3) provides an in-depth assessment of the causes of seven failures. The case histories have been selected to cover the range of typical failure modes experienced in UK landfills. Details include the types of lining systems, description of the failure mode, discussion of the factors causing the failure, any remedial measures, conclusions and any recommendations. The final part of the chapter (Section 4.4) provides a summary of the key factors controlling each of the failures and the lessons to be learned.

4.2 Questionnaire Survey

4.2.1 Survey details

A survey form '*Record of Stability Incident*' was developed to obtain information on the frequency and mode of failures in UK landfill lining systems. EA representatives on the Project Steering Group were involved in developing the survey form. The survey was designed for use by individual Environment Agency staff. It has five main sections. Part 1 asks for general information relating to experience, area of work (i.e. EA region), experience of failures (i.e. number) and general views on the importance of stability issues. Part 2 requests information on the type of lining system and mode of specific failures. Part 3 covers the causes of instability and Part 4 asks for information on any remedial works carried out. Part 5 requests information on any stability issues that are not currently well understood or are inadequate. Copies of the survey form were distributed to the regions via representatives on the Project Steering Group. The aim was for the survey to be distributed nationally to scientific officers and inspectors. Specific information included on the survey form is confidential. The questionnaire is reproduced below.

Record of Stability Incident

Background & Aims

An essential part of any assessment of a landfill lining system is the need to consider its stability.

The need for such a stability assessment is an implicit requirement of the Landfill Directive, which is due to be instrumented into UK Law later this year. To aid in the practical assessment of stability issues, the Environment Agency has let a contract for the production of a guidance document on stability design. Golder Associates, in conjunction with Loughborough University, has been appointed to produce this document.

To ensure that the guidance document concentrates on the problems encountered in industry we need to provide a realistic database of stability incidents throughout the UK. This database will lead to a best practice guidance document and may provide a basis for the design and management of landfill sites in the future.

It would aid the project greatly if you could complete this confidential form. Part 1 should only be answered once for each area and Parts 2, 3 and 4 should be answered for each stability incident.

Thank you

Part 1 – General Questions

Section 1.1 - General

1) Which Environment Agency region/area do you cover?	
2) How many stability incidents have you experienced in the last 10 years?	
3) How important do you feel landfill stability is to the protection of the environment?	(Not Important) (Very Important) (1) (2) (3) (4) (5)
4) Do you think the Landfill Directive will improve the quality of landfill stability assessments?	(Not Much) (Great Deal) (1) (2) (3) (4) (5)
5) How is the EA currently positioned to be able to review operator's stability assessment?	(Badly Positioned) (Well Positioned) (1) (2) (3) (4) (5)

Part 2 – Instability Questions

Section 2.1 – General

1) What is the site name and phase/cell ?		
2) What is the geology directly beneath the site?	a) Soil – Granular b) Soil – Cohesive c) Rock – High permeability/aquifer d) Rock – Low permeability/ aquiclude	( ) ( ) ( ) ( )
3) What type of landfill site?	a) Shallow sided void (<30^0) b) Steep sided void (>30^0) c) Above ground/land raise	( ) ( ) ( )

Section 2.2 – Method of Containment

1. What type of lining system was employed?	a) In-situ clay b) Compacted Clay Liner (CCL) c) Geosynthetic Clay Liner (GCL) d) Bentonite Enhanced Soils e) Single Geomembrane f) Geomembrane CCL Composite g) Geomembrane GCL Composite h) Multi Layer Composite i) Other (……………..…..………)	( ) ( ) ( ) ( ) ( ) ( ) ( ) ( ) ( )
2) What is the main type of waste?	a) Domestic b) Commercial c) Industrial d) Inert e) Other (……………..…..………)	( ) ( ) ( ) ( ) ( )

Section 2.3 – Instability Questions

Did the instability involve the slope (including capping)or the base?	a) Slope b) Base	( ) ( )
1) Where did the instability occur?	a) In the subgrade b) In the lining system c) In the waste d) In the capping e) Interface of subgrade/liner f) Interface of liner/waste	( ) ( ) ( ) ( ) ( ) ( )
2) What type of instability occurred?	a) Geomembrane failure b) Basal heave c) Movement of the subgrade d) Failure in the waste slope e) Failure in the side slope liner	( ) ( ) ( ) ( ) ( )
3) When did the instability occur?	a) During construction b) Prior to waste placement c) During waste placement d) After waste placement	( ) ( ) ( ) ( )

Part 3 – Cause of Instability (Only answer what is applicable)

Section 3.1 – Geomembrane Failure

1) If the geomembrane failed, what was the cause of failure?	a) Tear of sheet b) Weld failure c) Sliding along interface d) Back pressure from water	( ) ( ) ( ) ( )

2) If water was present, where did it come from?	a) Groundwater flow b) Condensation c) Poor/broken drainage d) Other seepage (……...…………...)	( ) ( ) ( ) ( )

Section 3.2 – Basal Heave

1) What type of basal heave occurred?	a) Low volume water break through b) Significant volume of seepage c) Other seepage (…...…..………...)	( ) ( ) ( )

Section 3.3 – Movement of the Subgrade? (e.g. excessive settlement or collapse)

1) In your opinion what caused the subgrade to move?	a) Soft formation b) Natural cavity c) Mining d) Other (…………….………………….)	( ) ( ) ( ) ( )

Section 3.4 – Failure in the Waste Slope?

1) Was the Failure Rotational or Translational?	a) Rotational b) Translational	( ) ( )
2) What caused the waste to move?		

Section 3.5 – Failure in the Side Slope Liner

1) What was the width of the tension cracks?	a) <5mm b) 5mm-1m c) >1m d) Not recorded	( ) ( ) ( ) ( )
2) What was the lateral extent of the tension cracks?	a) <1m b) 1m – 10m c) >10m d) Not recorded	( ) ( ) ( ) ( )
3) What was the failure slope height?	a) <5m b) 5-15m c) >15m d) Not recorded	( ) ( ) ( ) ( )

Part 4 – Remedial Works

Section 4.1 - General

1) Were there any remedial works carried out?	a) Yes b) No	( ) ( )
2) If **Yes –** what where they?		
3) If **No** - please comment?		

Final Question

Do you think that there are any issues relating to the stability of landfills and landfill lining systems that are not adequately considered at present?

4.2.2 Results of survey

A total of 26 questionnaires were returned from Thames, Northeast, Northwest and Southern regions and each described a single landfill failure. The results of the survey are given in Table 4.1 and summarised in Table 4.2 below.

4.2.3 Discussion of survey results

General concerns expressed by those surveyed were:

- The independence, impartiality and competence of third party CQA Engineers;
- Absence of Agency guidance on design and construction of steep slope lining systems;
- the use of processed shale in the construction of steep slope lining systems; and
- knowledge of the interaction between different materials used in a lining system.

It is worth noting that around 60% of the failures reported in the questionnaires involved groundwater and or surface water. Although 26 questionnaires were eventually returned, none were received from Anglian, Midland, Southwest regions nor the Environment Agency Wales. It is known that many landfill failures have occurred in these other areas. The number of failures reported in the questionnaires do not reflect the actual number that have occurred at UK landfills. It likely, however, that the questionnaire results do reflect the mechanisms and causes of failures and therefore it is worth investigating some of the failures further in the case histories in Section 4.3 below.

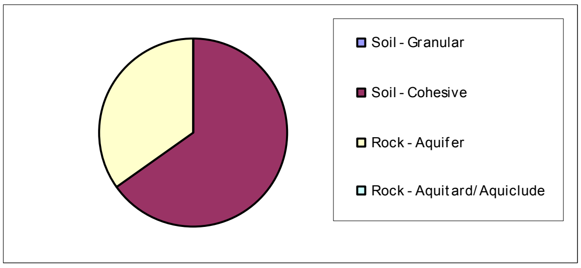
Table 4.1 Questionnaire survey results

	Sec. 1.1					Sec. 2.1			Sec. 2.2		Sec. 2.3				Sec. 3.1		S3.2	S3.3	Sec. 3.4		Sec. 3.5			Sec. 4.1	
Questionnaire No.	1	2	3	4	5	1	2	3	1	2	1	2	3	4	1	2	1	1	1	2	1	2	3	1	2
1a	T	-	5	3	4	-	b	b	b	mix	a	a	c	b	-	d	-	d	-	-	c	c	b	a	-
1b	T	-	5	3	4	-	b	a	b	mix	a	b	e	b	-	d	-	-	-	-	-	-	b	a	-
2	T	-	5	3	3	-	b	b	a	mix	a	e	c	b	-	d	-	d	-	-	c	b	c	a	-
3	NE	-	4	3	1	-	b	b	b	a	a	e	e	b	-	a	-	-	-	-	b	c	b	a	-
4	NW	-	5	4	3	-	c	b	f	a	a	g	f	b	-	-	-	-	-	-	-	-	-	a	-
5	NW	-	5	4	3	-	c	b	I	a	a	b	e	b	-	-	-	-	-	-	b	b	-	a	-
6	NW	-	5	4	3	-	c	b	f	a	a	b	e	a	-	-	a	-	-	-	-	-	-	a	-
7	NW	-	5	4	3	-	c	a	f	a	a	a	c	a	-	-	-	a	-	-	b	c	a	a	-
8	S	-	-	-	-	-	b	a	f	a	a	a	c	a	-	-	-	-	-	-	-	-	-	a	-
9	S	-	-	-	-	-	b	b	f	a	a	b	a	b	d	d	-	-	-	-	-	-	-	a	-
10	S	-	-	-	-	-	b	b	f	a	a	b	a	b	d	a	-	-	-	-	-	-	-	a	-
11	S	-	-	-	-	-	b	b	f	a	a	b	a	c	a	a/c	-	-	-	-	-	-	-	a	-
12	S	-	-	-	-	-	b	c	a	a	a	c	d	c	d	a	-	a	a	-	-	-	-	a	-
13	S	-	-	-	-	-	b	a	f	a	b	b	e	c	c	-	-	-	b	-	-	-	-	a	-
14	S	-	-	-	-	-	b	b	f	a	a	b	e	a	d	a	-	a	a	-	b	c	c	a	-
15	S	-	-	-	-	-	b	a	f/g	a	a	b	a	c	d	a	-	-	-	-	-	-	-	a	-
16	S	-	-	-	-	-	b	a	f/g	a	a	a	c	a	-	a	-	a	a	-	d	d	d	a	-
17	NE	-	-	-	-	-	c	b	b	-	a	a	c	c	-	-	-	c	-	-	-	-	-	a	-
18	NW	-	-	-	-	-	a/b	a	f	-	b	b	-	c	c	-	-	-	-	-	-	-	-	a	-
19	NW	-	-	-	-	-	b	a/c	f	mix	b	a/b	b	a	-	a	b	-	-	-	-	-	-	a	-
20	NW	-	-	-	-	-	c	b	i	mix	a	-	-	a	-	-	-	-	-	-	-	-	-	a	-
21	NW	-	-	-	-	-	c	b	i	mix	a	b/c	-	a/c/d	-	-	-	-	-	-	-	-	-	b	-
22	NW	-	-	-	-	-	c	a	f	mix	b	a	b	a	-	a	b	-	-	-	-	-	-	a	-
23	NW	-	-	-	-	-	b	a/c	b	mix	a	a/b	e	b	-	a	-	-	-	-	b	c	b	a	-
24	NW	-	-	-	-	-	b	a	f	mix	b	a/b	b	a/b	-	a	b	-	-	-	-	-	-	a	-
25	NW	-	-	-	-	-	c	b	i	mix	a	c	d	d	-	-	-	-	-	-	-	-	b	a	-

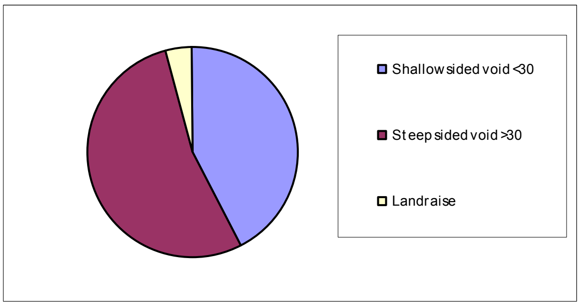
Note: Details of the actual questions asked and the available answers are given in the example questionnaire in Section 4.2.1.

Table 4.2 Summary of questionnaire survey results

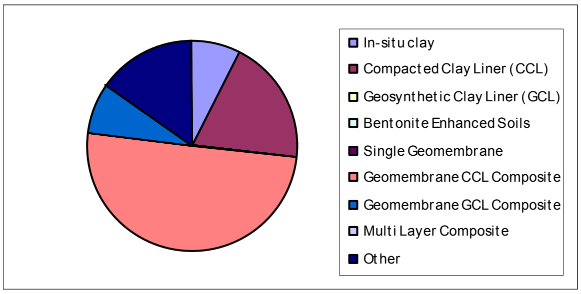
Question: What is the geology beneath the site?
Comments: The majority of sites that suffered a failure are underlain by cohesive soil. No failures were recorded on granular soils or low permeability rock.



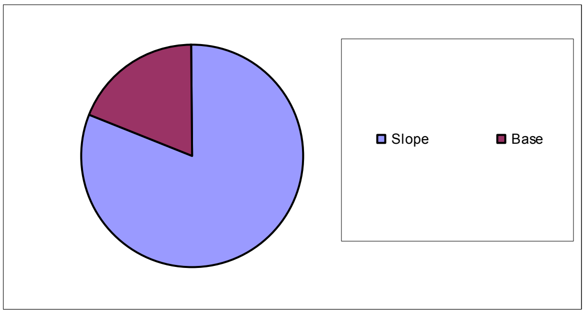
Question: What type of landfill site?
Comments: Although the majority of failures occurred in steep sided landfills, 42.3% occurred in shallow (less than 30° side slope gradient) landfills



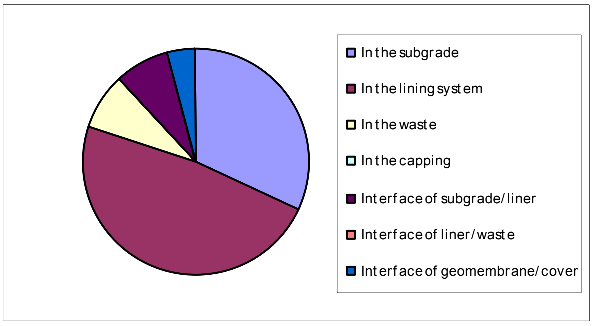
Question: What type of lining system was employed?
Comments: The landfills had a range of lining systems however 50% were composite clay and geomembrane lining systems.

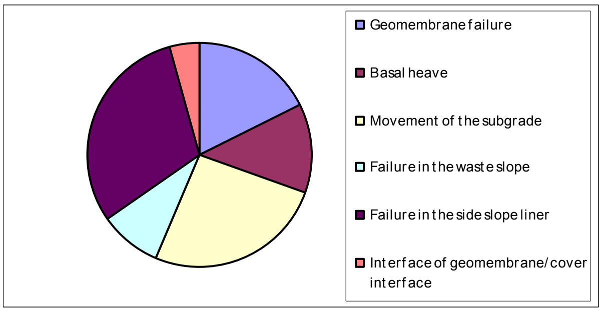
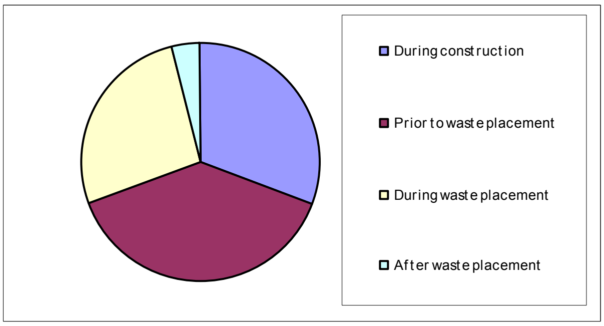


Question: Did the instability involve the slope (including capping) or base?
Comments: As expected, the majority of the failures involved the landfill side slope.



Question: Where did the instability occur?
Comments: Nearly half (48.0%) of the failures occurred in the lining system, whilst a large minority of failures (32%) were in the subgrade.



Question: What type of instability occurred? *Comments:* Most failures comprised movement in the side slope liner (30.4%), however 26.1% of the failures were movements in the subgrade. There were a significant amounts of geomembrane failures (17.4%) and 13% of failures were basal heave.	
Question: When did the instability occur? *Comments:* The majority of failures reported were before or during waste placement. 30.8% occurred during construction, 38.5% prior to waste placement and 26.9% during waste placement. The number of post waste placement failures (3.8%) may not give a true reflection as structural monitoring is seldom carried out.	

4.3 Case Histories

4.3.1 Rationale

This section provides a brief summary of the reasoning behind the selection of the case histories included in the review. Each case history has been chosen to highlight one or more common failure mechanisms that relate to either aspects of the design or construction process. They highlight key issues and lessons learned that relate to design and construction of landfill liners. A brief outline of each case history is provided in Table 4.3 below.

Table 4.3 Outline of case histories

Case history	Outline
No. 1	• Compacted clay liner/geomembrane composite liner; • very shallow slope; • failed during waste placement; • failure occurred in compacted clay liner.
No. 2	• Compacted clay liner underlain by drainage geocomposite; • failed during construction (placement of clay liner on 1 in 2 slope); • failure occurred at interface within geocomposite drain.
No. 3	• Steep slope lining system; • "Christmas tree" compacted clay liner; • liner constructed in lifts; • toppling type failure during construction.
No. 4	• Composite liner consisting of compacted clay liner/geosynthetic clay liner and geomembrane; • failed prior to waste placement; • uncontrolled groundwater; • localised softening, slumping and basal heave of compacted clay liner.
No. 5	• Compacted clay liner; • failure of over steep temporary waste slope; • boundary between phases of filling controlled shear surface; • shearing also in compacted clay layer.
No. 6	• Geomembrane liner; • failed prior to waste placement; • uncontrolled groundwater; • localised uplift of geomembrane and softening of sub-grade.
No. 7	• Compacted clay liner; • perched water in slopes not controlled; • basal heave due to use of inadequate factor of safety; • localised softening of compacted clay liner.

4.3.2 Case history No. 1

Summary

A mass movement occurred during waste placement. A translational type slide took place with the basal shear surface located in the compacted clay liner. The failure surface formed just below the geomembrane in a layer of softened clay. This material was a sacrificial layer that should have been removed prior to geomembrane installation. This low shear strength clay resulted in sliding occurring on a very shallow slope.

Site background

The site was originally used as a sand quarry extracting Moulding Sand for brass castings. The quarry excavated Tertiary sands, which are overlain by London Clay. Final waste depth in Cell 6 was expected to be 25 to 30 metres.

Site details

The site accepts commercial, industrial and domestic waste under the current waste management licence. The site was scheduled to have 17 Phases. The lining systems used in Cell 6 and the adjacent Cell 7 had the same design and construction details. The system comprised a 2mm HDPE geomembrane overlying a 1m layer of reworked clay derived from the on-site London Clay. Liner protection and drainage materials were then placed over the composite lining system.

The construction of Cell 6 took place over the period September 1998 to January 1999 and Cell 7 from July to August 1999. Waste placement commenced in Cell 6 during February 1999 but was stopped when significant mass movements were noted in September 1999. The body of waste involved in the failure had a maximum thickness of 15m and a plan area of 2.75 hectares (27,500m^2). The slipped waste body had a volume in excess of 200,000m^3. Figure 4.1 shows a plan of the adjacent Cells 6 and 7 and a cross-section through Cell 6. Initial observations of the failure included: i) A large tension crack developed in the waste along the boundary line of Cell 6 and the adjacent Cell 5. ii) The geomembrane liner pulled out of its anchor trench at the northern and eastern sides of the cell. iii) At the dividing bund with Cell 7 at the south-east corner of the cell, there was significant over-folding and wrinkling of the geomembrane. iv) Excavation of the geomembrane revealed that shearing had occurred within the clay immediately beneath the geomembrane.

Liner design and construction

The base of Cell 6 was designed and constructed with a fall of about 1 in 23 (2.5°) towards the south-west corner. The western and southern boundaries of Cell 6 are formed by clay bunds 2m in height with 1 in 1 (45º) side slopes, over which the geomembrane extends. On the northern and eastern boundaries of the cell the geomembrane liner extends up natural clay cut slopes about 8 metres in height. The geomembrane is assumed to terminate in an anchor trench. The CQA document supplied only summarised the original clay and geomembrane testing along with typical values from the literature. No relevant field and test records obtained during construction were included in the document. An important aspect of the design was that a 'sacrificial layer' of clay was to be placed on top of the 1 metre thick compacted clay liner. The reasoning being that this layer would protect the liner from variations in the weather (e.g. softening and drying out). The sacrificial layer was to be removed prior to installation of the geomembrane. The compacted clay liner was specified using a range of dry density values required to achieve the design permeability of 1 x 10^{-9}m/s. However, the minimum shear strength requirements of the liner were not specified and therefore the design did not specifically cover stability issues. A value was included in the preliminary design carried out by a different consultant.

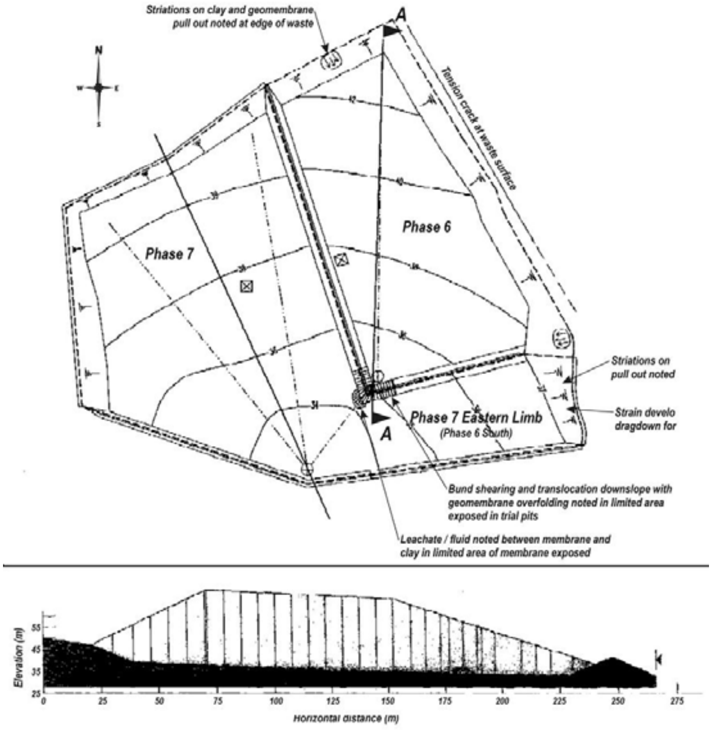


Figure 4.1 Plan and cross-section of Cells 6 and 7 of Case Study No 1

Investigation and site inspection

Investigation of the failure included exposing the clay and geomembrane basal liner components to make a visual inspection and taking samples to obtain a relationship between moisture content and undrained shear strength of the clay. A very soft layer of clay was found beneath the geomembrane liner. The shear surface was located approximately 1cm below the interface between the geomembrane and clay. Samples of clay taken post failure from immediately beneath the geomembrane were found to have moisture contents in the range of 50% to 55%. Based on the laboratory test, this would suggest undrained shear strengths in the range 10kPa to 15kPa. It was surmised that the softened sacrificial layer of clay had not been removed prior to geomembrane placement. It may have been assumed that leaving it in place would protect the underlying 1m thick clay liner. The consequence of the low shear strength was not considered. The lack of a specified minimum shear strength value for the clay liner meant that there was no trigger on shear strength to highlight potential problems.

Analysis

A total stress back analysis of the failure was carried out to obtain the undrained shear strength required for a factor of safety of 1.0. This showed that the presence of the layer of very soft clay produces failure conditions. It should be noted that the waste was not at full height. An effective stress analysis using drained strength parameters from the literature (i.e. $c' = 0$, $\phi' = 20^0$ for fully softened London Clay) and pore pressure conditions based on undrained loading, also gave a factor of safety of about 1.0. Estimates of residual shear strength parameters were used to assess the remedial measures and the stability of Cell 7 (i.e. the shear plane in the clay liner would be at residual as a result of the metres of slip movement). The stability analyses were sensitive to the assumed unit weight of the waste.

Liner integrity

The waste movement caused the geomembrane liner to pull out at the margin anchor trenches and also caused over-folding and wrinkling in the vicinity of the failed bund in the south-east corner of Cell 6. It is possible that other defects, tears and seam partings may have compromised the medium/long-term integrity of the liner. It was decided that as the failure occurred in the sacrificial clay layer, the 1m thick clay liner was unlikely to have been compromised. It was also concluded that works to remove the waste in order to examine and hence repair the geomembrane were likely to cause a higher risk of compromising the integrity of the composite lining system than leaving the geomembrane in place, even though it may have some defects.

Conclusion and recommendations made

During September 1999, Cell 6 of the landfill experienced a large translational slippage involving about 200,000m^3 of waste. The failure plane occurred just beneath the geomembrane in a very soft layer of sacrificial clay. This in turn was above a 1m thick compacted clay liner. The sacrificial layer had been intended as temporary protection to the clay liner and was to have been removed prior to geomembrane installation. As the sacrificial layer was to have been removed, no assessment was made of its engineering suitability (i.e. in terms of shear strength). In a deviation from the design, this sacrificial layer was left in place. Following failure, the sacrificial layer was found to have unacceptably high moisture content and thus it had very low shear strength. This allowed the waste to fail during placement on a very shallow slope angle. The fast rate of waste placement, combined with the low permeability of the clay beneath the geomembrane, meant that the soft clay layer did not have time to consolidate, and hence gain in strength, during filling (i.e. undrained loading conditions occurred). Following failure, the presence of a failure plane with a much reduced (i.e. residual) shear strength must be considered in design of remedial works and adjacent cells or other engineering.

4.3.3 Case history No. 2

Summary

A failure occurred during construction of the composite lining system of the north slope of Phase 2 of this site. The failure comprised movements of a clay liner that was being placed on top of a drainage geocomposite. The failure plane was located on an internal interface (geotextile/drainage core) of the geocomposite.

Site background

The site was originally a clay quarry supplying the onsite brickworks. The clay is from the Weald Clay formation with occasional lenses of sand. Construction of Phase 2 commenced in June 1996.

Site details

The site has been operational since 1993 and is licensed to accept commercial, industrial, domestic, inert and civic amenity wastes and it takes approximately 450,000 tonnes per annum.

Site investigation

An extensive hydrogeological study of the site was carried out as part of the site investigation, which included the installation of piezometers. The north slope of Phase 2 was 30m high and was therefore divided into three 10m sections separated by berms. The lower slope comprised engineered clay fill placed against in situ weathered mudstone/siltstone in the west, and in situ Weald Clay in the east. During the construction of the north slopes, it became evident that the overall height of the lower section of slope needed to be increased due to the unsuitable material found at the base of the slope. At this stage, extensive slope stability calculations were carried out and adequate factors of safety were calculated.

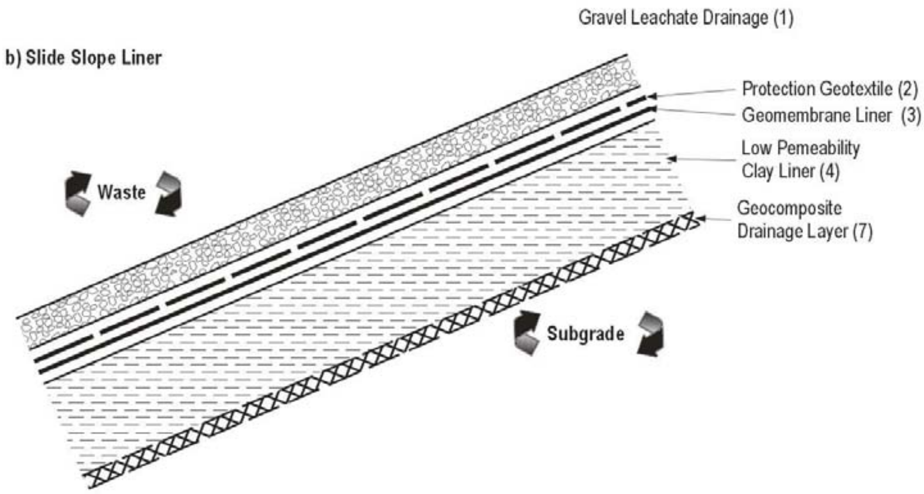


Figure 4.2 Schematic of the lining system of Case Study No 2

Liner design and construction

Figure 4.2 shows the details of the lining system. Stability of the lining system was considered in design. The critical failure mode was found to be sliding of the clay liner on the top of the geocomposite. The calculations were based on the 'infinite slope' method (conservative in this case) using parameters supplied by a manufacturer of the proposed drainage geocomposite. The designer concluded that the factor of safety was dependent primarily on the apparent cohesion between the upper surface of the geocomposite and the clay liner. The calculations were based on the assumption that a geocomposite drainage layer would be placed beneath the clay liner. It was also assumed in design that the geocomposite drain would have high internal shear strength (i.e. that it would be fully laminated and bonded). A different type of geocomposite was eventually chosen and delivered to site and this had an upper geotextile layer that was only glued to the drainage core in strips at 600mm wide centres. The material used on site was therefore much weaker than the material envisaged by the designer. The designers carried out direct shear tests on the geocomposite/clay interface, however these tests were carried out at normal stresses of around 400kPa. Such high normal stresses are only relevant for the long-term stability of the system when the site is full of waste. The actual normal stress at the time of failure was around 35kPa and the designer did not carry out any tests at this normal stress.

On the 16 October 1996 a large area of clay liner on the north face of Phase 2 failed by sliding. The failure was confined to the lower section of the northern slope (Figure 4.3). This area had been initially prepared, covered by the geocomposite drainage blanket, and was in the process of receiving the clay liner.

Analysis

It was evident that the failure occurred within the geocomposite on the weakly glued upper geotextile/drainage core interface. From extensive laboratory testing, peak and residual shear strengths were measured for all the interfaces. These results were used to back analyse the failure and it was clear that if peak shear strengths had been available then the failure would not have occurred.

Figure 4.3 Post failure conditions at the site of Case Study No 2

Post-peak near residual conditions were necessary for the failure to occur. Further examination of the construction procedures revealed that the contractor had used the clay liner as a diagonal haul road in the vicinity of the failure. The repeated loading of heavy dump trucks, breaking and accelerating very close to the geocomposite was seen as the likely cause of the mobilisation of post peak shear strengths.

Remedial works

Since the failure occurred during construction, remedial works comprised removing the geocomposite and clay liner, amending the design and installing counterfort drains as the under-drainage instead of the geocomposite and replacing the clay liner.

Conclusion and recommendations made

This case history demonstrates a number the important issues. Good site supervision is required to ensure that materials specified during the design are actually used on site. Strain softening interfaces (post-peak shear strengths) can be mobilised by construction activities (i.e. the loading and repeated trafficking on the diagonal haul road). Stability assessments should be made for all stages of construction and operation (e.g. during construction, during waste placement and post waste placement).

4.3.4 Case history No. 3

Summary

The final cell (Cell 3) of a redundant sand quarry was lined using compacted clay in a “Christmas tree” style construction. Towards the final stages of landfilling there was a toppling type failure of the clay away from the quarry face. The failure involved internal shearing through the mineral liner and resulted in large tension cracks appearing. The failure occurred during a 3m lift prior to waste placement.

Site background

The site was originally a sand quarry supplying building sand from the Lower Triassic which is also a major aquifer. The site was developed in the early 1970’s for its current use by the local authority.

Site details

Since 1988, the site has accepted domestic, inert, industrial, commercial, difficult, special (asbestos) and liquid waste. Originally there were 3 voids on site. Landfilling of the final cell commenced in the summer of 2000. The final cell being landfilled is approximately 5000m^2 in plan area with 30m high slopes. The slopes are inclined between 70^o and 80^o. Occasional bands of mudstone could be seen in the quarry face. Due to the large amount of waste arriving onto the site, filling was planned to take approximately 18 months.

Liner design and construction

A “Christmas tree” type construction was implemented with a minimum clay barrier thickness of 1.2m. The majority of overhanging sections were trimmed off to minimise the amount of clay overlying waste. The clay liner also had a nominally 1m wide inert fill layer placed against it for support and protection. As the quarry was approximately 30m deep, 10 equal lifts were planned. Figure 4.4 shows a schematic diagram of the liner system. A toppling type failure resulted in movement of the clay away from the rock face along a 30 metre length of the barrier. The failure occurred during construction of the eighth 3m lift.

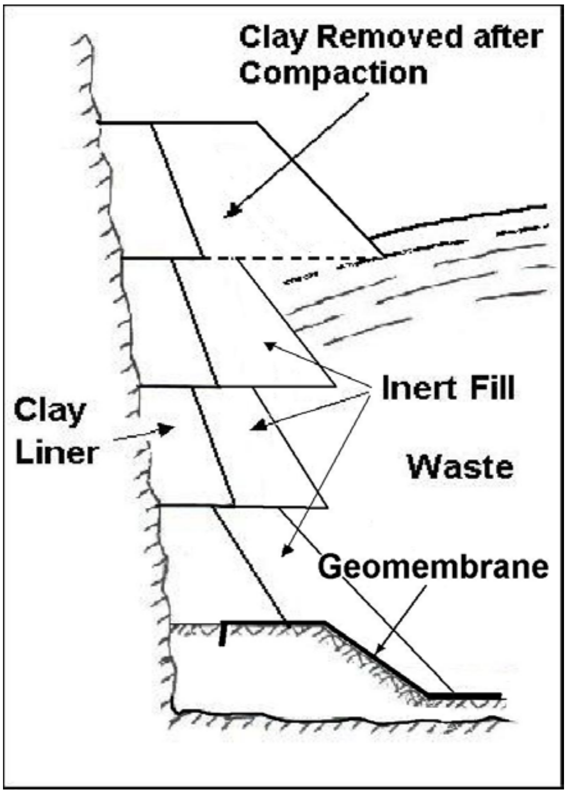


Figure 4.4 The lining system at Case Study No 3

Investigation and site inspection

Researchers have been monitoring the lining system at this site using inclinometers, extensometer and pressure cells since landfilling commenced in August 2000. This research project is funded by an EPSRC grant and is further supported by the Environment Agency, WRG, Shanks, Cambridge Insitu and Golder Associates. An element of the study involves the in situ measurement of waste stiffness.

The research has shown that significant amounts of movement in both the vertical and horizontal directions have taken place in the inert waste and clay lining materials. The monitoring is ongoing.

Liner integrity

The lining system follows that of a "Christmas tree" design, with inert waste material being placed against the liner for lateral support and protection. The phasing of this in conjunction with the general site landfilling is important as the clay lining system relies upon the lateral support from the inert material and MSW for its structural stability.

During the eighth lift, after the lining system had been trimmed there was a delay in placing waste against the liner system to provide lateral support. As a result, the clay liner moved in a toppling fashion away from the rock face (see Figure 4.5). A sub-vertical tension crack occurred (≤200mm wide) over a length of approximately 30m. It penetrated through the mineral liner from the top surface of the lift to the underlying quarry face. The rock face was clearly visible through the tension crack. This section of clay lining has been replaced.

Figure 4.5 View of the tension crack at the rear of the liner caused by toppling failure

Conclusion and recommendations made

Failure occurred due to a lack of support from the inert and waste material. Poor phasing of the waste placement meant that the clay barrier was left unsupported for a prolonged period. The use of the "Christmas tree" method of construction may have contributed to the failure by allowing a cantilever effect of the lift to take place. Compressible waste under the clay overhang of each lift is a poor founding material. The research project into performance of the lining system is ongoing.

Supporting evidence

The mode of failure is consistent with that observed at another similar site. This is also a near vertical "Christmas tree" type steep side slope lining system. A toppling type failure occurred during construction, resulting in tension cracks forming in the clay liner parallel to the quarry face. Over steepening of the liner, an excessive lift height and a delay in placing waste against the clay liner are all considered to be contributory factors. Taken together, the findings from these two case histories indicate that the stability and integrity of "Christmas tree" type mineral liners on steep side slopes is compromised by the flawed design and construction process.

4.3.5 Case history No. 4

Summary

Uncontrolled groundwater in the mudstone sub-grade resulted in softening of the clay liner and hence a reduction in its shear strength. Groundwater conditions were influenced by periods of heavy rainfall. Slumping occurred in a number of areas at the bottom of the side slope. Remedial works were conducted to stabilise the slopes. Under liner drainage was installed to control groundwater flow and to dissipate pore water pressures.

Site details

The landfill cell was constructed at the base of a 40 metre deep working quarry. The geology of the quarry is coal measures, sandstone and mudstone. The sandstone is present in the upper 20 metres of the quarry with the mudstone located in the lower 20 metres.

Lining design and construction

The lining system consisted of a 500mm thick compacted clay liner that was overlain by a geosynthetic clay liner (GCL) and a 2mm thick HDPE geomembrane. Bunds where constructed around the perimeter of the cell with side slope gradients of 1 in 2.5 and a vertical height of 5 metres. The bunds were constructed of re-worked mudstone. The clay used to line the base was a yellow brown site won glacial till. The optimum moisture content and maximum dry density of this material was determined from laboratory tests as being 10% and 2.08 Mg/m^3 respectively. The shear strength of the as placed clay was recorded as being in excess of 90 kN/m^2.

Analysis

The base of the quarry represents the lowest part of the site. During periods of heavy rainfall, surface water seeped under a relatively high hydraulic gradient into the mudstone forming both the sub-grade and the engineered bunds. The high hydraulic gradient, together with build up of surface water gradually saturated the overlying compacted clay liner. The increased moisture content resulted in reduction of the mineral liner shear strength through softening. This resulted in slumping of the clay liner at a number of locations at the bottom of the side slopes (Figure 4.6). Water seepages were observed at these locations.

Figure 4.6 View of slumped areas of softened compacted clay liner

Remedial measures

The CQA Engineer identified the location of all seepages. The mineral liner at the location of each seepage area was excavated. A 150mm diameter HDPE pipe together with drainage stone was installed within each excavated trench, with each pipe passing within the sub-base up the sidewall where it was connected to a ring main (Figure 4.7). The ring main was connected to an eductor pump. Each under drainage channel excavated was then covered with a separation geotextile and the mineral liner/GCL/FML placed. The water levels were monitored and pumping maintained until an adequate amount of drainage stone and waste had been placed in order to resist any hydrostatic uplift pressure. Upon the installation of the eductor system, no further instability/seepage was noted.

Figure 4.7 View of drainage works to control local groundwater

Conclusions and recommendations

Uncontrolled transient groundwater in the sub-grade resulted in softening of the clay liner and subsequent shear failure. Simple remedial measures were used to control the groundwater in the sub-grade. This case history demonstrates the importance of a thorough investigation of the sub-grade groundwater regime before detailed design. It should include possible transient conditions in response to climatic events. The cost of providing adequate under liner drainage would be significantly less if included at the design stage as apposed to carrying out remedial works.

4.3.6 Case history No. 5

Summary

Failure of a temporary waste slope in Cell 3 of this site occurred on 30th December 1994. The basal section of the shear surface was located within the 3 metres thick compacted clay basal liner. The inclined part of the shear surface was located along a former soil covered slope face that had been buried during filling to make the waste slope steeper. This former slope had reduced shear strength in relation to the waste because there was no waste reinforcement across it. Failure was triggered by formation of the over steep slope, although failure did not occur immediately after slope formation.

Site background

The site was originally used as a sand quarry extracting Moulding Sand for brass castings. The sand is from Tertiary sands, which are overlain by London Clay.

Site details

The site accepts commercial, industrial and domestic waste under the current waste management licence. The site was scheduled to have 17 Phases. The basal lining system in Cell 3 comprised a 3 metre thick layer of re-worked and compacted London Clay. The waste was placed directly on the clay. Waste filling in this cell was approaching the boundary of the site in late 1994 and a temporary slope face was constructed to allow construction of the permanent edge barrier. A cross-section through the temporary slope is shown in Figure 4.8. The clay bund at the toe of the slope was to stop leachate from contaminating the unfinished section of the liner. It is believed that the bund was not constructed using engineered fill (i.e. not compacted to a design density or strength). A temporary soil bund was formed at the top of the slope, presumably as a safety barrier. The waste forming the slope was primarily MSW material. The waste slope was approximately 1 in 1.5 and 20 metres high.

Failure of the temporary waste slope occurred on 30th December 1994 along a 90 metre length. A sketch of the post-failure cross-section is shown in Figure 4.8.

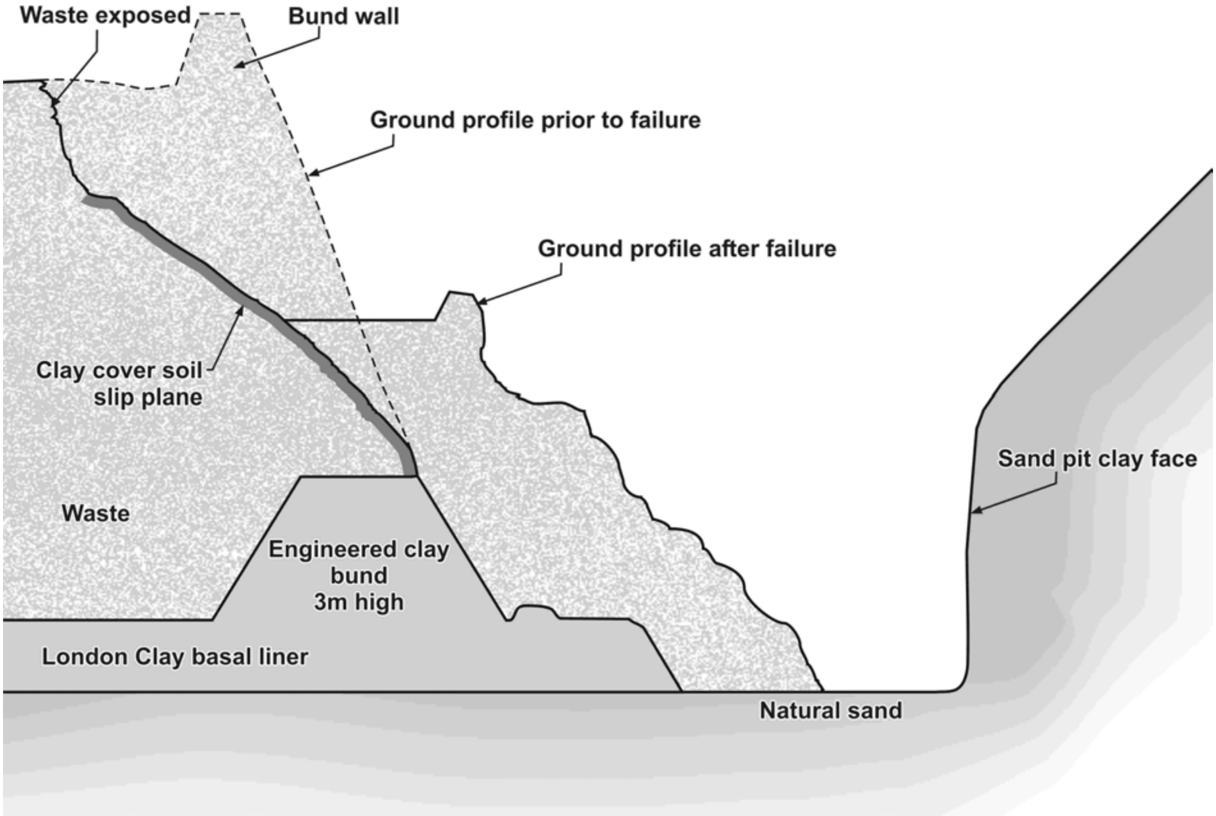


Figure 4.8 Sketch cross–section showing the pre- and post-failure slope profiles

Analysis

Waste involved in the failure was excavated and re-deposited in another part of the site. The basal liner was exposed to assess its integrity. The investigation showed that the basal section of the shear surface formed within the clay liner. A slickensided shear plane was clearly visible. The location of the inclined part of the shear surface was controlled by a former (i.e. relic) slope surface. Figure 4.9 shows a general view of the failure and it can be seen that the exposed part of the shear surface is relatively smooth. This is due to the high soil content of the waste, which is consistent with the former waste slope having a clay soil cover. It appears that the slope face was made steeper to increase the void space and the previous waste slope was buried. The presence of this relic slope is important because normally waste has relatively high shear strength due its reinforced nature. Figure 4.10 shows a near vertical waste slope adjacent to the failure and the reinforcement fibres (e.g. paper and plastic) are clearly visible. There would have been no waste reinforcement across the relic slope surface. It is also likely that the additional waste placed to extend the fill area would have been poorly compacted. The bund constructed at the crest of the slope would have decreased stability by loading the top of the slope. The role of the clay bund at the toe is unknown but if the clay was not compacted fully it is likely that the shear surface would have passed through it.

Figure 4.9 General view of the waste slope failure showing the soil covered relic slope

Figure 4.10 Steep slope in waste showing the reinforced nature of the material

Remedial measures

The waste mass involved in the failure was completely removed. The 3 metre thick clay liner was excavated to remove softened and sheared material (i.e. some areas had been left exposed for many months). A stepped profile was formed in the existing liner to enable a competent joint to be formed with the new section of basal liner. The final permanent waste slope was designed with a maximum angle of 33°.

Conclusions and recommendations

An over-steep temporary waste slope was constructed. A previous soil-covered shallower slope was buried. This relic surface in conjunction with the clay basal liner formed a preferential path for the shear surface. These factors caused the failure. Temporary waste slopes should be designed, they should not be allowed to just evolve. Phases of filling must be fully integrated to ensure that weak (i.e. un-reinforced) planes are not formed within the waste body.

4.3.7 Case History No. 6

Summary

A composite lining system constructed on shallow side slopes suffered distress as a result of uncontrolled surface water and groundwater at this site. Ponding of surface water adjacent to the cell resulted in failure of the southern bund. This allowed water to penetrate beneath the geomembrane and led to softening of the compacted clay liner. In addition, the ground displacements of the failed bund caused stresses in the geomembrane and wrinkling. Groundwater in the north-east corner formed a bubble beneath the geomembrane and softening of the compacted clay liner and sub-grade. Remedial works included replacement of softened material, construction of additional drains to control groundwater in the area of the bubble and reconstruction of the southern bund.

Site details

The site lies to the west of and immediately adjacent to an existing landfill site. The site occupies a section of a shallow sided valley. It is planned to develop it in six phases and provide 1.2 million cubic metres of void for mainly domestic waste. The site is underlain by the Tertiary deposits. A site investigation carried out in August 1998 indicated the strata to be soft to firm clay with sandy patches, silty lenses and some fibrous material. The depth of the strata is unknown. Elevated groundwater levels were recorded as part of the site investigation.

Lining design and construction

Cell 1 was constructed in late 1999. The lower slope is at 1 in 5 up to a height of 3 metres above the landfill base, and is lined with a composite compacted clay/textured geomembrane system. The compacted clay has a minimum thickness of 1m. A geotextile protection layer overlies the geomembrane and a 300mm thick layer of granular material forms a drainage layer. The upper slope is at 1 in 8 and is lined with a composite GCL/textured geomembrane system. The protection and drainage layers are as for the lower slope. To control groundwater during liner construction a series of trench drains were constructed in the side slopes. These were 600mm deep, 300mm wide trenches in the sub-grade with a 150mm perforated pipe on a 50mm thick layer of 10mm gravel with the trench backfilled with 20 to 40mm gravel. A blinding layer of sand was used at the surface of the trench. The number and spacing of these drains is unclear from the as built records. Due to the shallow angle of the side slopes and the rate of waste placement a significant area of the lining system was exposed throughout 2000.

Analysis

During the late autumn and early winter of 2000 water began to collect under the geomembrane liner in the north-east corner of the cell forming a pronounced bubble. In order to try and ascertain where the water was coming from, sand berms were placed on the liner to form a series of isolated bays. Initially the amount of water collecting in the bays was relatively small. It was proposed that the water was a result of condensation under the geomembrane, although this mechanism seemed highly unlikely. In early November the water quantity in the bays markedly increased, particularly in the bays at the southern end of the slope. It was noted that a rotational failure had developed in the southern bund of the landfill. It is possible that the water that had built up in the area had penetrated the bund causing the failure, and that this also led to the build up of water under the geomembrane liner. The source of the bubble of water under the liner in the north-east corner was not found. The slump in the bund and the ingress of water caused distress to the geomembrane. It was stressed and became wrinkled (see Figure 4.11). In addition, the compacted clay liner and sub-grade beneath the GCL were softened in some areas. There was concern that differential settlements could occur under loading from the waste thus leading to over stressing of lining components and a loss of integrity.

Figure 4.11 Distress of geomembrane following failure of the southern bund

Remedial measures

The geomembrane and GCL were removed in order to allow inspection of the compacted clay liner and sub-grade respectively. Where softened materials were found, defined by having a shear strength less than 50 kPa, it was removed and new material placed. In the area of the water bubble in the north-east corner of the cell a series of trench drains were constructed with the same specification as those in the original design. The only difference being the use of geotextile as the filter surround to the gravel (i.e. replacing the sand). The aim of these additional drains was to ensure that groundwater pressures could not build up. The original geomembrane and GCL material were inspected for damage and where deemed acceptable were reused. The southern bund was removed and reconstructed.

Conclusions and recommendations

Failure of the lining system occurred at this site due to inadequate control of both groundwater and surface water. Surface water external to the cell caused failure of the southern bund and this led to water flowing beneath the geomembrane. Build up of groundwater is the only likely explanation for formation of the bubble of water in the north-east corner of the cell. The exact source of the water was not found. However, the limited site investigation conducted indicated the sub-grade to have sandy patches and silty lenses. These will have higher permeability and would act as preferential flow paths. Although elevated groundwater pressures were noted as part of the site investigation, detailed knowledge of the groundwater pressure distributions in the sub-grade is unknown as is the seasonal variations in the water pressures. Trench drains were included in the original design to eliminate build up of pressure beneath the liner. Why these failed to work is not known. It is possible that spacing may have been too large to intercept all higher permeability layers and the blinding layer of sand on the surface of the gravel may have reduced their effectiveness. The drains installed as part of the remedial work appear to be working satisfactorily.

This case history demonstrates the importance of a thorough investigation of the sub-grade groundwater regime before detailed design. It should include possible transient conditions in response to climatic events. The cost of providing adequate under liner drainage would be significantly less if included at the design stage as apposed to carrying out remedial works. The importance of providing adequate control of surface water is also highlighted by this case history. Where compacted clay liners and cohesive sub-grade are liable to softening from groundwater, a minimum shear strength should be specified, and used to ensure that softened material is not left in place that could result in differential settlements and loss of liner integrity.

4.3.8 Case history No. 7

Summary

Uncontrolled groundwater in the sub-grade of this site has resulted in occurrences of slope instability and basal heave during construction of a number of cells. Slope instability resulted from perched groundwater in a sand layer within the clay sub-grade. Basal heave occurred during excavation for the basal lining system. Sand layers and lenses in the clay sub-grade contain groundwater under artesian conditions.

Site details

Development of the landfill has taken place in phases with a number of cells constructed per phase. The failures in this case history occurred in Phases 2 and 3. The ground conditions comprise superficial clays over a sandstone aquifer. The clay layer underlies the base and side slopes of the landfill and contains sand lenses and layers. Excavation levels in the clay result in artesian groundwater pressures in the sand layers beneath the base.

Lining design and construction

The lining system comprised a 1m thick compacted clay liner overlain by a 300mm drainage blanket. The clay liner is formed using material excavated at the base of the cell. Depths of each cell have been designed to maximise the void space for waste placement, with the potential for basal heave controlling excavation levels. Side slopes are up to around 10m in height and constructed at around 1 in 2. A sand layer within the clay daylights in the slope face.

Analysis

There is a history of slope and base heave failures at this site. Slope failures occurred in the western batter of Phase 2, Cell 6 during Spring 1998 and Summer 1998. The failures in spring 1998 involved a 150m section of 1 in 2.2 slope. Seepages were observed from the failed areas. The slope was reconstructed without any changes to the design or method of construction. In Summer 1998 the slope failed again with the same mechanism, shortly after construction. Investigation revealed the failure occurred half way up the slope corresponding with the interface between a band of sand within the clay sub-grade. There was perched groundwater within the sand layers around the site. During the construction of the adjacent Phase 1, a drain had been installed to control the groundwater within the sand layer and this had been pumped for some time.

Slope failures also occurred in the southern batter of Phase 3, Cell 1A in December 1998 and in the western batter of Phase 3, Cell 2 in Summer 1999. In both cases the failures were similar to those in Phase 2. Groundwater pumping from the sand layer had not been carried out as previously agreed.

Basal heave occurred in Phase 3, Cells 2 and 3 in May and August 2000 respectively. Failure occurred during construction of the basal lining system.

Depths of excavation for the base of the cells were controlled by artesian pressures in sand layers within the clay sub-grade and hence by assessing basal heave. A key aim of the design was to maximise the void space for waste placement. Calculations for base heave used factors of safety = 1.0 in the critical zones of the base and in some instances the base level required for stability included the compacted clay liner. Consideration was not given as to how the sub-grade was to be kept stable during excavation for, and placement of, the mineral liner. In case of failure by basal heave, remedial works were planned (i.e. installing drains beneath the liner). No consideration was given to the influence basal heave might have on the strength and compressibility of the sub-grade and hence to the long-term integrity of the mineral liner. As noted above, failures did occur and the lining system was disrupted.

Remedial measures

Drains installed to reduce groundwater levels in the sand layer daylighting in the clay side slopes proved effective in ensuring slope stability. To ensure stability against basal heave a pressure relief system was constructed beneath the mineral liner in the areas of concern.

This comprised a geocomposite drain feeding a pump sump backfilled with 20mm gravel. A higher factor of safety was required for basal heave calculations. Regular monitoring of the condition of the sub-grade was required by the EA during excavation.

Conclusions and recommendations

Slope instability occurred due to inadequate control of groundwater. Despite an initial failure and successful drainage solution to stabilise the slope, design of subsequent phases did not incorporate those drainage measures and failures occurred again. The failure to learn from experience demonstrates poor communication between parties involved. Although information was available on the location of sand layers beneath the base and values of porewater pressure within these layers, the level of information including seasonal variations in groundwater is unknown. A factor of safety = 1.0 should not be used in basal heave calculations as there is always some uncertainty in the input parameters used in the calculations. The factor of safety used must be justified in the context of the quality of the input data and consequences of failure. A strategy of allowing failure and then carrying out remedial measures could result in a disturbed sub-grade and hence compromise the long-term integrity of the lining system. This case history demonstrates inadequate design, poor communication and poor site practice.

4.4 Summary of Key Issues

A summary of the main factors contributing to the failures described above and the key issues identified are listed below.

4.4.1 Case history No. 1

- Material specifications need to include minimum shear strengths for mineral liners. Knowledge of this can be used to trigger QA decisions on site regarding acceptability of material. This should ensure that the as-placed material has shear strength equal to or greater than the value used in design.
- Analysis methods must be justified (i.e. whether to use drained or undrained conditions). The decision depends upon rates of loading and drainage path lengths. Guidance is required for the EA and designers.
- Residual shear strength conditions may be required post-failure in the design of remedial works.
- Analysis of stability is sensitive to unit weight of waste used in calculations. The design values must be justified and a sensitivity analysis conducted using the potential range of values.
- It should be noted that failure could occur on a very shallow slope with low driving forces.
- The failure occurred during waste filling, not at the final slope profile, hence temporary conditions must be analysed as part of the design process.

4.4.2 Case history No. 2

- Failure occurred during construction, the method of working, include plant loads, must be considered as part of the design process.
- Measurement of shear strength properties for use in design should use normal stress levels that are relevant for the stresses imposed on the materials in situ. This may require a series of tests covering the construction period (i.e. at low stresses) and a

series for the post waste placement condition (i.e. at higher stresses). Performance tests must be carried out (i.e. using site specific materials).

- The design process should take into consideration the possibility of mobilising post-peak shear strengths. These can result from plant loads and waste settlement.
- The CQA procedure must check that the materials used on site are in accordance with the material specifications, and specifically that the engineering properties meet the minimum values assumed in design. A particular problem is the quality of bonding between layers of geocomposite (i.e. internal shear strength).

4.4.3 Case history No. 3

- Failure occurred during construction. The method of construction, including the phasing of construction processes, must be specified and followed.
- As the liner is not self-supporting, deformation of lining system into the waste must take place if it is to achieve equilibrium. This raises serious questions about the long-term integrity of such lining systems.
- Material specifications need to include minimum shear strengths for mineral liners. Knowledge of this can be used to trigger QA decisions on site regarding acceptability of material. This should ensure that the as placed material has shear strength equal to or greater than the value used in design.

4.4.4 Case history No. 4

- Failure occurred during construction of the lining system. The temporary conditions that exist during construction must be assessed as part of the design process.
- A high quality comprehensive site investigation is required in order to obtain the worst-case conditions for use in design (i.e. seasonal fluctuations in groundwater). A detailed knowledge of the structure of the sub-grade is required. Permeable bands and discontinuities control groundwater flow, and weak layers and discontinuities control sub-grade stability.
- An adequate under liner drainage system is a key element of the design. Retro fitting a drainage system after stability problems have been encountered is likely to be significantly more expensive than including a suitable system in the original design.
- The shear strength of mineral liner material can change post construction (e.g. softening). Material specifications need to include minimum shear strengths for mineral liners. Knowledge of this can be used to trigger QA decisions on site regarding acceptability of in situ material.
- Consideration should be given to strains in a compacted clay liner resulting from changes in the engineering properties of the sub-grade (e.g. softening of the sub-grade).

4.4.5 Case history No. 5

- Temporary waste slopes must be designed, they should not be allowed to just evolve.
- Design of waste slopes should not rely on past experience as the constituents of MSW change with time.
- Phases of filling must be fully integrated to ensure that planes of weakness are not constructed into the body of the waste. These can form a preferential path for a failure

surface. Such surfaces will have lower shear strength due to the absence of reinforcement across the plane.

- The basal section of shear surface will usually form in the mineral liner, or at an interface between liner component.
- It is important to justify values of waste unit weight used in stability calculations and to carry out sensitivity analyses using the range of possible values.

4.4.6 Case history No.6

- Failure occurred during construction of the lining system. The temporary conditions that exist during construction must be assessed as part of the design process.
- A high quality comprehensive site investigation is required in order to obtain the worst-case conditions for use in design (i.e. seasonal fluctuations in groundwater). A detailed knowledge of the structure of the sub-grade is required. Permeable bands and discontinuities control groundwater flow, and weak layers and discontinuities control sub-grade stability.
- An adequate under liner drainage system is a key element of the design. Retro fitting a drainage system after stability problems have been encountered is likely to be significantly more expensive than including a suitable system in the original design.
- Consideration should be given to strains in a geomembrane resulting from changes in the engineering properties of the sub-grade (e.g. softening of the sub-grade).
- Surface water must be controlled to ensure that it does not influence the stability of earth structures or be allowed to flow beneath the liner. This can lead to softening of compacted clay liners and sub-grade and hence too differential straining of the lining system following waste placement.

4.4.7 Case history No.7

- Failure occurred during construction of the lining system. The temporary conditions that exist during construction must be assessed as part of the design process.
- A high quality comprehensive site investigation is required in order to obtain the worst-case conditions for use in design (i.e. seasonal fluctuations in groundwater). A detailed knowledge of the structure of the sub-grade is required. Permeable bands and discontinuities control groundwater flow, and weak layers and discontinuities control sub-grade stability.
- Control of groundwater is fundamental to ensuring slope and base stability.
- Factors of safety used in basal heave calculations must be justified in the context of the quality of the input data and consequences of failure.
- A strategy of allowing failure followed by remedial measures should not be used as the disrupted subgrade may result in loss of liner integrity in the long-term.
- Good communication between parties involved in the design, construction and operation of landfills allows lessons learned from failures to be incorporated in subsequent phases of the works.

5. LINING SYSTEMS AND THEIR COMPONENTS

5.1 Introduction

Lining systems are required to prevent uncontrolled release of leachate and landfill gas into the environment. Many materials, both natural and man made, are used in lining systems and these are described with particular emphasis on their properties relating to landfill stability in Chapter 6. This Chapter gives an overview of the different landfill lining systems currently used in the UK.

Components of lining systems can be divided into four main functions; barrier layers, protection layers, drainage layers and reinforcement layers. Figure 5.1 provides a summary of typical UK lining systems and these are discussed in turn in the following sections.

5.2 Barrier Layers

Barrier layers provide the means of preventing (or more accurately limiting) leakage of liquid and gases from landfills. The simplest form of barrier is a single liner system, i.e. a barrier comprising a single material type. In the UK many materials are used as single liners for example engineered clay, bentonite enriched soil (BES), colliery spoil, processed shale, polymeric geomembranes, geosynthetic clay liners (GCLs) and more recently dense asphaltic concrete (DAC).

Single clay liners comprise naturally occurring clay reworked to provide a low permeability seal by controlling various factors such as moisture content, density, stone content etc. Typical specifications require a minimum of 1 m thickness of clay with a measured hydraulic conductivity of $1x10^{-9}$ m/s. Clay is placed in layers to ensure compliance with a performance specification (usually hydraulic conductivity) which is controlled on site by measuring dry density and moisture content. A second performance specification, which is often not considered, is shear strength. Minimum shear strength requirements need to be given since the assumptions made in the stability assessments (Chapters 10, 11 and 12) have to be followed through in construction. It should be noted that the hydraulic conductivity of clay liners generally decreases with increasing moisture content whilst shear strength tends to decrease with moisture content. There is therefore a dichotomy between the performance specifications and a compromise must be achieved in the final solution.

BES liners are man made barriers made by mixing a controlled amount of bentonite clay with (typically) sand host material. This mixing can either be carried out in situ, or ex situ by mixing in a batching plant. A low hydraulic conductivity, typically $1x10^{-10}$ m/s is achievable and a very uniform material can be produced by careful control of the mixing process. Colliery spoil, together with mudstones and siltstones, can be broken down on site and used as barrier layers; they are specified and controlled in the same way as single clay liners. Typically, it is more difficult to achieve comparable hydraulic conductivity to a clay liner but provided there is sufficient clay sized particles in the material, suitable barrier layers can be formed.

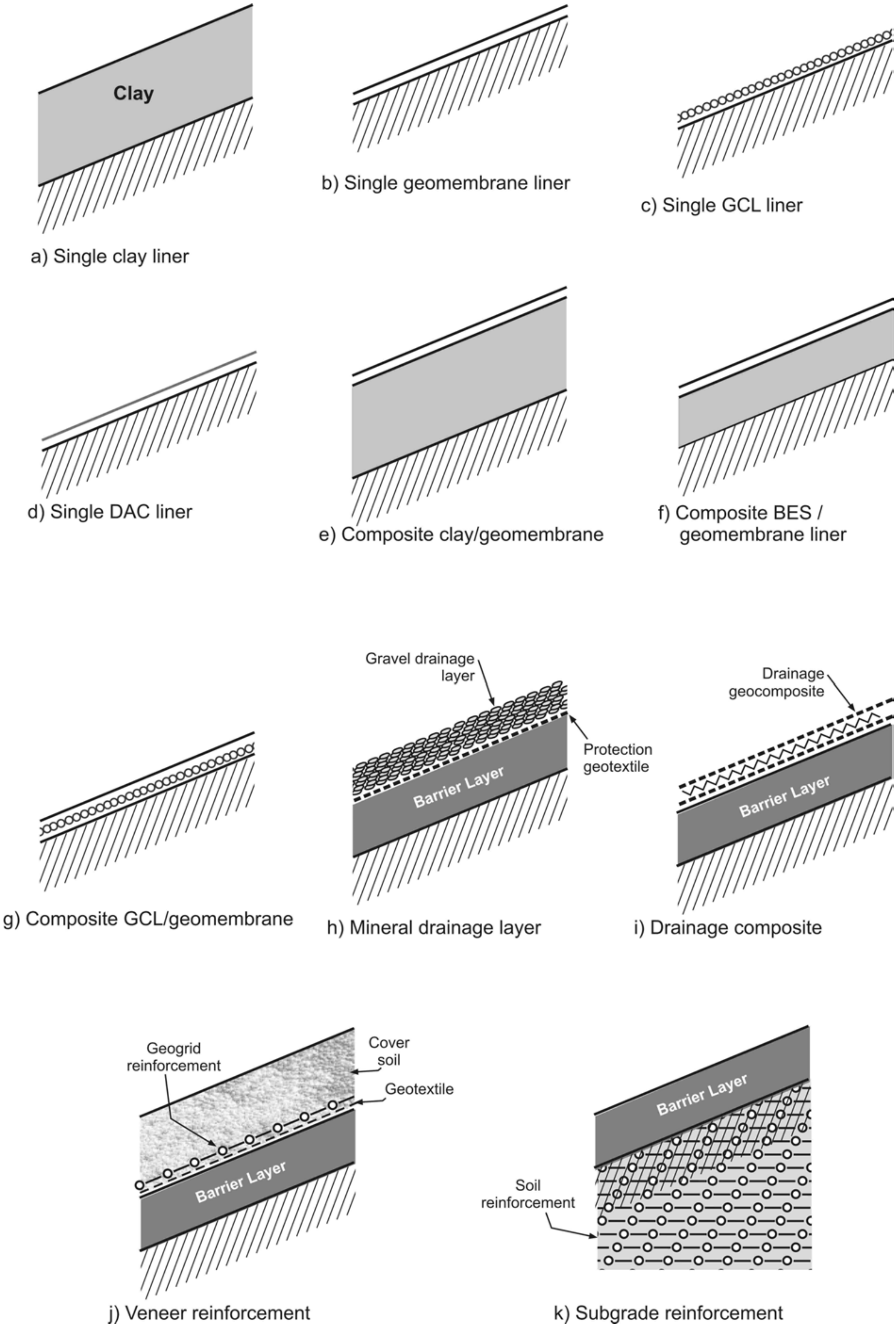


Figure 5.1 Typical UK landfill lining systems

Polymeric geomembranes are used extensively in the UK in landfill applications. Typically high density polyethylene (HDPE) geomembranes are used on the base and side slopes, and linear low density polyethylene (LLDPE), low density polyethylene (LDPE) and very low density polyethylene (VLDPE) have been used for capping applications. These low density polyethylene geomembranes are sometimes called very flexible polyethylene geomembranes (VFPE) due to their mechanical properties, and it is this ability to undergo large strains without failure that leads to their use as capping liners. Polymeric geomembranes are essentially impermeable to water, and leakage through geomembranes are typically considered to be due to defects such as tears and holes caused by installation activities and potentially through stress cracking. It should be noted however, that leakage through polymeric geomembranes is by diffusion through the liner.

Geosynthetic clay liners (GCLs) are increasingly used in landfills, particularly as single liners for capping applications. GCLs are a combination of geosynthetic materials (either a geotextile or geomembrane) and bentonite clay which provide an extremely low hydraulic conductivity (typically less than $5x10^{-11}$ m/s). The main GCLs used in the UK are:

- bentonite and adhesive between two geotextiles;
- bentonite between two geotextiles stitch bonded together;
- bentonite between two geotextiles needle punched together; and
- bentonite and adhesive on a geomembrane.

Further details of the construction of GCLs are given in Daniel & Koerner (1995). The properties of GCLs will be highly dependent on the type of product used and these are discussed further in Chapter 6.

Over the last few years dense asphaltic concrete (DAC) has been used as a barrier in UK landfills. DAC is a designed mix of various size aggregates and asphalt, placed to a typical thickness of 80 mm, and can have a hydraulic conductivity less than $1x10^{-12}$ m/s. DAC was originally developed as a barrier layer in dam construction and has been used in landfill applications in Europe for several decades. The major advantage of using DAC is that it is a very robust material in terms of physical damage with a comparatively high resistance to erosion and weathering.

In the correct circumstances all the barrier layers described in this section, except for single geomembrane liners can all be used separately as single lining systems, however there is often benefit from combining two or more of the components together to form a composite lining system. Using engineered clay and a geomembrane in intimate contact with each other, for example, will significantly improve the performance of the individual components as well as providing a double barrier.

5.3 Protection Layers

All barrier layers require some form of protection and these protection layers therefore form part of the lining system. Damage can come from a variety of sources such as:

- heavy plant operating close to the barrier;
- puncture and stressing from materials (e.g. drainage media) either above or below the barrier;

- placement of, and subsequent loading by, the waste above the barrier;
- erosion of mineral liners via surface runoff;
- heat and ultra violet radiation from the sun causing desiccation of mineral barriers and degradation of polylmeric barriers; or
- oxidation and chemical degradation of polymeric barriers.

It is therefore important to protect the barrier layers from potential damage. This protection is often afforded by mineral layers such as sand or gravel (note that gravel should not be placed directly next to a geomembrane), or by synthetic materials such as geotextiles, geocell mattresses or recycled rubber products. In the case of DAC, a mastic sealing layer is applied to the top surface of the barrier to promote runoff and protect from chemical damage.

Protection geotextiles are typically thick non woven needle punched materials that are placed directly above geomembranes. They protect the geomembrane by preventing puncture and excessive local strain from material above the lining system (usually drainage gravel). Assessment of the suitability of protection geotextiles can be carried out on a site specific basis using the proposed materials in accordance with the cylinder test (Environment Agency, 1998).

Other material that could be used for the protection of geomembrane barriers include geocell mattresses and recycled rubber products. Geocell mattresses are three dimensional geosynthetic materials that can be filled with sand or other materials to form a semi-rigid structure. Both filled geocells and rubber protection materials have been used in the UK to a limited extent, new products are regularly becoming available and are likely to be used as protection materials in the future.

5.4 Drainage Layers

Leachate must be removed from landfills to limit the head (i.e. water pressure) on the barrier in order to minimise leakage. Drainage layers are therefore used on the base and side slopes of landfills to act as conduits for the leachate to the extraction points. They are also used beneath lining systems to reduce groundwater pressures in the sub-grade. In addition, surface water run-off needs to be controlled above, and landfill gas removed from beneath, capping barriers. Therefore drainage layers are also used in capping systems. Materials used as drainage layers include sand or gravel layers, typically 300 mm thick (although this may rise to 500 mm with the introduction of the Landfill Directive) and geosynthetic drainage materials such as geonets and drainage geocomposites. In addition, pipes are often used to transfer liquid through and from these layers.

Sand has historically been used to form drainage layers above barrier systems, however it has become evident that the performance of sand and other fine grained material drainage blankets significantly reduce with time due to clogging (Brune *et al.*, 1991). Sand is therefore becoming less popular as a leachate drainage layer on the base of landfills, although it is still used on side slopes and for under-drainage beneath the liner.

Gravel has a higher initial hydraulic conductivity than sand and it is less likely to suffer from clogging.

Geonets have a planar polymeric structure consisting of a regular dense network of elements whose openings allow the flow of liquids and gases. They are often used with separation geotextile either placed, or bonded, above and below the geonet. The drainage geocomposites used in UK landfills tend to have cuspated HDPE drainage cores sandwiched between geotextile layers, and these often have much higher flow capacity than geonets. A key design issue with both these drainage geosynthetics is the strength of the bonding between the various elements and this is described in more detail in Chapter 6.

Perforated pipes are used in landfills for the collection of leachate from the base of the landfill, and solid pipes are used for the transfer of leachate out of the landfill.

5.5 Reinforcement Layers

Reinforcement layers comprising either geogrids or geotextiles are becoming increasingly common in UK landfills. They can be used in two ways. Firstly, they are use to aid the stability of veneer slopes (Hall & Gilchrist, 1995) where they are placed in the plane of the slope. Secondly, they can be used to create reinforced soil walls to form the sub-grade for geomembrane placement in steep side slopes (e.g. Di Stefano & Needham 1994, Jones 1996). In both applications, the stability of the slope is improved by the presence of the reinforcing materials.

6. STABILITY PROPERTIES OF LINING SYSTEM COMPONENTS

6.1 Introduction

The description and definition of barrier materials and other ancillary components used in landfill engineering have been well documented and explained in several key publications, e.g.:

- Environment Agency CWM 106/94C (Guidance on Good Practice for Landfill Engineering); and
- CIRIA Special Publication 124 (Barriers Liners and Cover Systems).

The aim of this chapter is to provide an introduction to the key material properties of common barrier components that are needed to assess stability and integrity. Information on other material properties such as permeability and chemical resistance is not included. Each family of materials is considered separately and information is provided on material definitions and the key parameters controlling stability and integrity. Existing guidance documents, technical reports and research papers are referenced where appropriate to direct the reader to more detailed information. The materials are grouped by the task they perform into barrier materials and ancillary materials. Ancillary materials have the role of drainage, filtration, separation and reinforcement. The main barrier materials considered are:

- mineral materials;
- geomembranes;
- geosynthetic clay liners;
- bentonite enriched soils; and
- dense asphaltic concrete.

Ancillary materials considered are:

- natural drainage materials; and
- geotextiles, geonets and drainage geocomposites.

6.2 Mineral Liners

6.2.1 Description

A mineral liner is a naturally occurring clay or shale reworked to provide a low permeability seal by controlling various factors such as moisture content, density, particle size distribution etc. In areas where there are no suitable sources of clay, Bentonite Enriched Soil (BES) can provide an alternative mineral liner. BES comprises between 6% and 10% bentonite mixed thoroughly with the host sand to provide a very permeability liner. The mixing can be done in situ, but preferably by the use of a batching plant in which the exact dosage of bentonite and water can be controlled.

6.2.2 Density and moisture content

The main property required of a mineral liner is low hydraulic conductivity (permeability coefficient) with values of $1x10^{-9}$ m/s and $1x10^{-10}$ m/s commonly being used as required values for clay and BES respectively. Laboratory permeability testing in accordance to BS 1377: Part 6: 1990 is used to confirm the specified requirements. Due to the low hydraulic conductivity of the mineral liner, laboratory testing typically takes several weeks to perform. Therefore other tests have to be employed on site as control tests for the works.

The relationship between dry density and moisture content for mineral liners can be established by carrying out compaction tests in accordance with BS 1377 Part 4: 1990. Figure 6.1 (after Mitchell *et al.* 1965) shows typical trends of dry density with moisture content and of dry density with compactive effort. Also shown is the relationship between hydraulic conductivity and moisture content. Lower values of hydraulic conductivity are obtained wet of optimum moisture content and therefore it is usual for moisture contents wet of the optimum to be specified by designers to ensure that the required value of permeability coefficient is achieved. However, compacting wet of optimum results in a reduction in the dry density and hence in the shear strength of the soil. It is important that designers appreciate the wider implications of specifying compaction moisture contents. If the soil is too wet this may give rise to a variety of problems related to low strength.

6.2.3 Strength properties

Shear strength of mineral liners can be measured in a five main ways. Firstly, the shear strength of a mineral liner can be measured in a direct shear test (BS 1377: Part 7:1990, Tests 4 and 5). In the direct shear test a sample of soil is laterally restrained and sheared along an induced horizontal plane while subjected to a stress applied normal to the plane. Both undrained and drained tests can be conducted. Secondly, unconfined compression tests (BS 1377: Part 7:1990, Test 7) can be carried out where the specimen is subjected to an axial compression without any lateral stress being applied. Only undrained tests are carried out.

Triaxial compression tests can simulate in situ stress conditions better than both direct shear and unconfined compression tests. Measurements of stresses due to imposed deformations, volume changes and pore water pressure can be made in a triaxial test. In an unconsolidated undrained (UU) triaxial test (BS 1377: Part 7:1990, Test 8), the sample is not allowed to drain during application of confining stress or axial load. In a consolidated undrained (CU) test (BS 1377: Part 8:1990, Test 7), the sample is allowed to consolidate fully under the applied confining stress but drainage is not permitted during axial loading and pore water pressures may be measured. In a consolidated drained (CD) test (BS 1377: Part 8:1990, Test 8), drainage is allowed during the full test. Undrained and drained parameters can be obtained depending upon the test set up (i.e. drainage conditions or pore pressure measurement).

The fourth method of shear strength measurement is the ring shear apparatus. Residual (large strain) shear strength of a soil is measured using a ring shear apparatus (BS 1377: Part 7:1990, Test 6). In this test an annular sample of remoulded soil is subjected to rotational shear while a normal stress is applied. Drained tests are normally carried out. Finally, the shear strength of a mineral liner in the field can be tested using a field vane (BS 1377: Part 9:1990, Test 4.4). This method consists of inserting a steel vane into the liner and rotating at a specified velocity, the torque of the device is related to the shearing resistance of the soil. This test is only used in clays having undrained shear strengths less than 100 kPa and appropriate corrections must be applied.

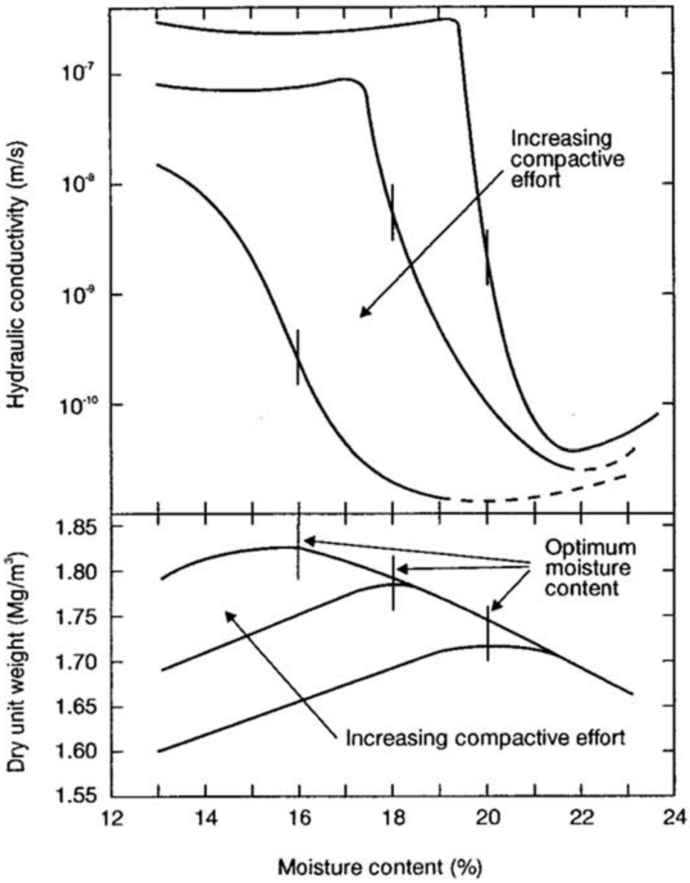


Figure 6.1 Effect of moisture content and compactive effort on hydraulic conductivity (after Mitchell *et al.*, 1965)

The shear strength of mineral liners will vary with moisture content, density, clay mineral content as well as the method of measurement. The selection of shear strength parameters will depend on the analysis to be carried out. For an assessment of the short-term stability (e.g. during and immediately after construction) the total stress (or undrained) shear strength applies. The undrained shear strength is normally given by c_u, or s_u, and is measured in kilo Pascals (kPa). For long-term stability analysis where drained conditions have been established (see Chapter 9), the effective stress shear strength is used given by the cohesion c′ (in kPa) and the friction angle, ϕ' (in °).

6.2.4 Swelling and shrinkage properties

Cohesive soils will change in volume in response to a change in moisture content. The relationship between moisture content and volume is dependent upon the type of clay minerals present. Minerals with a high swelling potential (e.g. montmorillonite) result in the largest volume changes. Plasticity index is used as a measure of a soil's plasticity, i.e. its ability to deform plastically over a range of moisture content, and hence provides an indication of the type and percentage of clay minerals.

Soils with a high swelling potential can undergo increases in moisture content that results in a marked decrease in shear strength, and an increase in permeability. The same soils, if subjected to a decrease in moisture content will shrink forming desiccation cracks and fissures. A general increase in strength of the material can result, but the mass strength may be reduced by the tension cracks. This affects the integrity of the soil as a barrier and increases the permeability.

Small moisture content changes in BES can have a significant influence on both strength and permeability. This is because it is the moisture content of the clay fraction that controls behaviour (Jefferis, 1998).

6.2.5 Stiffness properties

Stiffness of a soil is dependent upon grading, density, confining stress and drainage conditions (i.e. drained or undrained). Measurement of stiffness is normally carried out as part of triaxial testing. A number of types of stiffness can be measured (e.g. Young's modulus, shear modulus, constrained modulus). Stiffness of soils is not a constant as it is related to strain level. Linear elastic behaviour can be assumed where the change in applied stress is small, otherwise a relationship between strain level and stiffness is required. Sample disturbance and material variability mean than it is difficult to measure and/or specify values for a stratum. Ranges of possible values should therefore be assessed in design.

6.3 Geosynthetic Clay Liners

6.3.1 Description

Geosynthetic clay liners (GCLs) are a combination of geosynthetic materials (either a geotextile or geomembrane) and bentonite clay which provide an extremely low hydraulic conductivity (typically less than $5x10^{-11}$ m/s). During installation on site, the rolls are laid out and overlapped with additional bentonite powder added to help provide the inter-panel seal.

6.3.2 Thickness, mass per unit area and moisture content

Both the thickness and mass per unit area of GCLs are highly dependent on moisture content due to the water absorption properties of the bentonite, and they are both measured in accordance with ASTM D5993. Since bentonite is a very hydrophilic mineral, it will generally have a measurable moisture content at all times, for example in very humid areas its as-received (or "dry") condition it could have a moisture content as high as 20%. Its moisture content is measured using ASTM D4643, which is a standard measurement.

6.3.3 Strength properties

Internal strength

The internal strength of GCLs can be considered in several ways. Firstly, the internal tensile strength of a wide width sample can be measured using ASTM D 4595. Since the tensile strength of the bentonite content is essentially zero, this test merely tests the combination of geotextiles or the geomembrane carrier of the GCL. However, the tensile strength of GCLs has little impact on assessment of stability, since it is the transfer of stress in shear through the GCL that is of greater importance.

Secondly, the internal shear strength of GCLs can be measured using the conventional direct shear apparatus normally used for interface shear testing (ASTM D5321). The GCL is placed in the shear apparatus with the top geotextile fixed to the upper section of the shear box and the bottom geotextile fixed to the lower part of the shear box. The geotextiles are typically fixed along the full surface area using bent nails or carpet gripper rods to prevent stretching during the shearing process.

The internal strength of a GCL can also be measured by the peel test (ASTM D6496) in which the top and bottom layers of a GCL are gripped individually in tensile grips and pulled at a constant rate of extension by a tensile testing machine until the top and bottom layers separate. This is an index test and the results can only be used to evaluate the quality of the bonding process.

The results of the internal shear testing will depend on the type of GCL (whether it is reinforced by needle punching or stitching for example) and also the confining stress and moisture content of the bentonite.

Interface shear strength

The shear strength at the interface between GCLs and other geosynthetics or soils is often the critical part of a landfill side slope. The interface shear strength is normally measured in the direct shear apparatus, however when measuring interface shear strength, the whole GCL sample is fixed to either the top of bottom of the shear box, and the other geosynthetic (or soil) is placed either above or below it. The interface shear strength is often lower than would be expected from testing the geotextile component of the GCL against another geosynthetic or soil due to hydrated bentonite intruding into non-woven needle-punched geotextiles or extruding out of the woven geotextiles and into the interface being tested. The extremely low shear strength of bentonite leads to lower interface shear strength than would otherwise be the case (e.g. Lalarakotoson *et al.*, 1999, Heerten *et al.*, 1995).

6.4 Geomembranes

6.4.1 Description

A geomembrane is a very low permeability synthetic membrane liner used as a barrier layer. Polymeric geomembranes, the most common used in the UK for landfill applications, are made from relatively thin continuous polymeric sheets, with high density polyethylene (HDPE) and linear low density polyethylene (LLDPE) the most popular materials. Other polymeric geomembranes such as various low density polyethylene's, flexible polypropylene and plasticised vinyl chloride have also been used. Bituminous geomembranes, made from the impregnation of geotextiles with bituminous sprays and multilayered bitumen geocomposites are not widely used in the UK at present, and so this section deals with the stability properties of polymeric geomembranes.

The required characteristics for geomembranes and geomembrane related products used in solid waste storage and waste disposal sites have recently been formalised by CEN in draft format (pr EN 13493).

6.4.2 Strength properties

Internal strength

The tensile strength of a HDPE geomembrane is normally measured in an index test such as ASTMD D638. In these tests a dumbbell shaped sample of geomembrane is tested in tension at a given rate.

For assessing the performance of a geomembrane in the field, index tests should not be used. For example when considering the tensile stress induced in a geomembrane on a side slope from a stability analysis (see Section 11.3), the assumption of plane strain conditions (i.e. a unit width of an infinitely long slope) is normally made. It is the results of wide width tensile testing (ASTM D4885) which should be compared to the calculated stress. It should be noted that the wide width test results of textured geomembranes vary significantly for different manufacturing methods, performance testing should be carried out at design stage and should also be included in the conformance testing requirements.

For applications where the geomembrane will undergo three dimensional strains, for example basal liners used over compressible formations, or capping liners, the strength test of most relevance is the axi-symmetric test (ASTM D5617).

Other strength properties of geomembranes such as the puncture resistance (ASTM D4833, D5494) and tear resistance (ASTM D1004) are also important, however these parameters do not play a significant role in the stability of landfill sites.

Interface shear strength

The shear strength between various geosynthetics and between soils and geosynthetics is a key factor in the stability of landfill lining systems. There are currently three standards in use that provide guidance on testing procedures; BS 6906: 1991, ASTM D5321-92 and a German recommendation for landfill design GDA E 3-8 of 1997. A fourth standard, the final version of a preliminary European standard (pr EN ISO 12957-1: 1997) is imminent. Further details of interface shear strength is given in Chapter 7.

6.5 Dense Asphaltic Concrete

6.5.1 Description

Dense Asphaltic Concrete (DAC) comprises a controlled mixture of bitumen, aggregate, different sands, filler and other additives, placed and compacted to uniformly high density. DAC liners normally have a higher percentage of mineral filler and binder (usually 6.5 to 9.5%) and have much lower voids content (normally less than 3%) than highway paving asphalt mixes. DAC has been used as a landfill liner for over two decades in parts of mainland Europe and is becoming more popular in the UK.

6.5.2 Strength properties

Due to the well graded nature of DAC and the amount of compaction it undergoes on site, internal strength is unlikely to be an issue. Instead, it is the interface shear strength between the top of the DAC and any materials placed against it that will be the critical factor in design.

It is known that DAC will undergo creep movement on steep slopes (greater than 1 in 1 say), however there is insufficient information currently available to provide guidance on this aspect. It is clear that the performance of DAC on steep slopes under field conditions (i.e. buttressed by waste) needs further investigation.

6.6 Geotextiles, Geonets and Drainage Geocomposites

6.6.1 Description

Geotextiles, geonets and drainage geocomposites are used for a variety of functions in landfill applications including separation, filtration, protection, and drainage. Geotextiles are either woven or non-woven synthetic materials that can be made from a number of different natural materials (e.g. jute) and man-made polymeric materials (e.g. polyester, polyethylene, polypropylene). Geonets consist of integrally connected parallel sets of ribs overlying similar sets at various angles that allow the drainage of liquids and gases. Drainage geocomposites typically consist of a drainage core with a geotextile laminated to one or both sides and are designed for in plane flow over a large surface area. The central drainage cores can be biplane geonets (two layers of ribs superimposed over each other), triplanar geonets (two layers of inclined ribs separated by thick vertical ribs creating a wide flow channel) or cuspated core (regular pattern of cusps with large voids between).

The required characteristics for geotextiles and geotextile related products used in solid waste disposal sites have recently been formalised by CEN (BS EN 13257: 2001)

6.6.2 Strength properties

Internal strength

The basic test is to place the geotextile (or geonet) within a set of clamps, place the assembly in a mechanical testing machine and stretch the geotextile in tension until failure occurs. Both the load and the deformation are measured in the test. There are various variations on this test, e.g. ASTM D751, ASTM D4632, ASTM D4595 and BS EN ISO 10319: 1996. The latter two methods are wide width tests and are considered to be performance orientated tests.

The internal strength of a drainage geocomposite can be considered in two ways. Firstly, the internal tensile strength can be measured as above. Secondly, the internal shear strength of the geocomposite can be measured using the conventional direct shear apparatus normally used for interface shear testing (ASTM D5321, BS 6906: Part 8: 1991, pr EN ISO 12957-1: 1997). The geocomposite is placed in the shear apparatus with the top geotextile fixed to the upper section of the shear box and the bottom geotextile fixed to the lower part of the shear box. Consideration needs to be given to fixing the geotextiles to prevent stretching during the shearing process. An indication of the internal strength of geocomposites can be obtained by carrying out peer tests such as the ASTM D6496 for GCLs. While these tests are relevant for an assessment of stability, additional tests may be required to demonstrate performance suitability.

Interface shear strength

The shear strength between various geosynthetics and between soils and geosynthetics is a key factor in the stability of landfill lining systems. There are currently three standards in use that provide guidance on testing procedures; BS 6906: 1991, ASTM D5321-92 and a German recommendation for landfill design GDA E 3-8 of 1997. A fourth standard, the final version of a preliminary European standard (pr EN ISO 12957-1: 1997) is imminent. Further details of interface shear strength is given in Chapter 7.

6.7 Granular Material

6.7.1 Description

Granular materials (both sand and gravel) are typically used for drainage and protection materials in landfills. It is the shear strength of these materials that needs to be considered in the stability assessment.

6.7.2 Strength properties

Shear strength

The shear strength of granular materials will depend on the particle strength (see below) and the grading, particle shape, density of the material and confining stress. Shear strength is normally measured in the direct shear apparatus (BS 1377: Part 7: 1990, tests 4 and 5). The small shear box (Test 4) can be either 60 mm or 100 mm square and 20 mm to 25 mm high and therefore can only be used for sands due to limitations on the maximum allowable particle size. The large shear box (Test 5) is 300 mm square and 150 mm high and can be used for soils containing particles up to 20mm in size. Material must be placed in the test device at a density equivalent to the value expected on site (i.e. dependent on method of placement and degree of compaction). Confining stress applied to the sample should be consistent with those on site for the stability condition being considered. Internal shear strength of a granular material is dependent upon stress level; the material will increase in strength with increasing overburden pressure (e.g. depth of waste).

Particle strength

The physical strength of an aggregate is indicated by a test such as the ten per cent fines test (BS 812: Part 111: 1990). In this test, a specimen is compacted in a standard manner into a steel cylinder fitted with a freely moving plunger. The specimen is then subjected to a load applied through the plunger, which crushes the stone to a degree which is dependent on the crushing resistance of the material. The degree of crushing is assessed by a sieving test on the crushed specimen. The procedure is repeated with various loads to determine the maximum force which generates a given sieve analysis; this force is taken as the ten per cent fines value(TFV).

6.8 Other Materials

A range of other materials may be used as components to lining systems particularly as drainage materials and as cover soils. Drainage materials include tyres (both whole and shredded) and secondary aggregates such as bricks, concrete, glass etc. The stability properties of these other materials should be assessed as part of the landfill design process.

6.9 Summary of Key Points

Material properties related specifically to strength and deformation must be measured and used in the assessment of lining system stability and integrity. Many of the tests currently carried out are to obtain index values for quality control. These values cannot be used in design. Performance tests using site specific materials and boundary conditions are required to obtain material parameters for stability assessment.

7. INTERFACE SHEAR STRENGTH

7.1 Introduction

The stability of a geosynthetic landfill lining system is controlled by the shear strength between the various interfaces, i.e. geosynthetic/soil and geosynthetic/geosynthetic interface shear strengths. The importance of interface shear strength was illustrated by the slope failure in Phase IA of Landfill B-19 at Kettleman Hills in the USA. This significant failure instigated a major investigation carried out by the University of California at Berkeley (Seed *et al.*, 1988).

A full explanation of shear strength theory can be found in standard soil mechanics text books. Only a brief summary is provided below. The shear strength (τ) of a soil at a point on a particular plane was originally expressed by Coulomb as a linear function of the normal stress (σ_n) on the plane at the same point:

$$\tau = c + \sigma_n.\tan\phi \qquad \text{Equation 7.1}$$

where c and ϕ are the shear strength parameters, now described as the cohesion intercept and the angle of shearing resistance respectively. In accordance with Terzaghi's fundamental concept that shear stress in a soil can be resisted only by the skeleton of solid particles, shear strength can be expressed as a function of effective normal stress:

$$\tau = c' + \sigma_n'.\tan\phi' \qquad \text{Equation 7.2}$$

where c' and ϕ' are the shear strength parameters in terms of effective stress. This defines a straight line in the $\sigma_n' : \tau$ stress plane. It is assumed that, for sliding to occur on any plane, the shear stress has overcome a frictional resistance, $\sigma_n'\tan\phi'$, which is dependent on the effective normal stress σ_n' acting on the plane and on a friction angle ϕ', together with a component c', which is independent of the normal stress. This component c' is often called cohesion but is more usefully regarded merely as an intercept on the shear stress axis which defines the position of the Coulomb strength line.

This failure criterion can be extended to interface shear strength with the strength line rewritten as:

$$\tau = \alpha' + \sigma_n'.\tan\delta' \qquad \text{Equation 7.3}$$

where the friction angle of the soil ϕ' is replaced by an interface friction angle δ', and the shear stress axis intercept is replaced by the interface adhesion intercept, α'. Figure 7.1 shows typical results from shear tests on a geosynthetic vs. soil interface. The shear stress/shear displacement curves are used to obtain the peak shear strength (i.e. the maximum shear stress the interface can withstand) and the residual shear strength (i.e. the minimum shear strength at large displacements) for each of the tests at a different stress level. Note that the tests data could also be plotted as shear stress vs. shear strain.

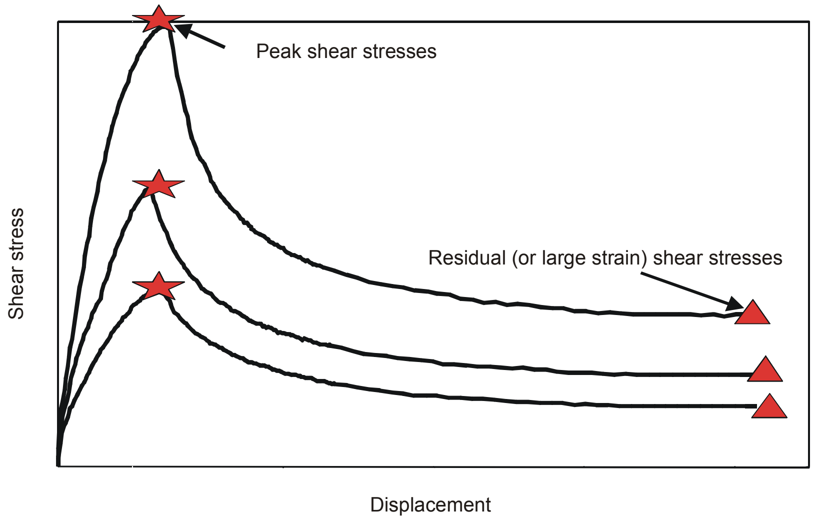


Figure 7.1 Typical shear stress/displacement curves for a geosynthetic vs. soil interface

These values are then used to define the peak and residual shear strength envelopes as shown in Figure 7.2, from which the shear strength parameters are obtained from the best-fit-straight lines through the sets of points.

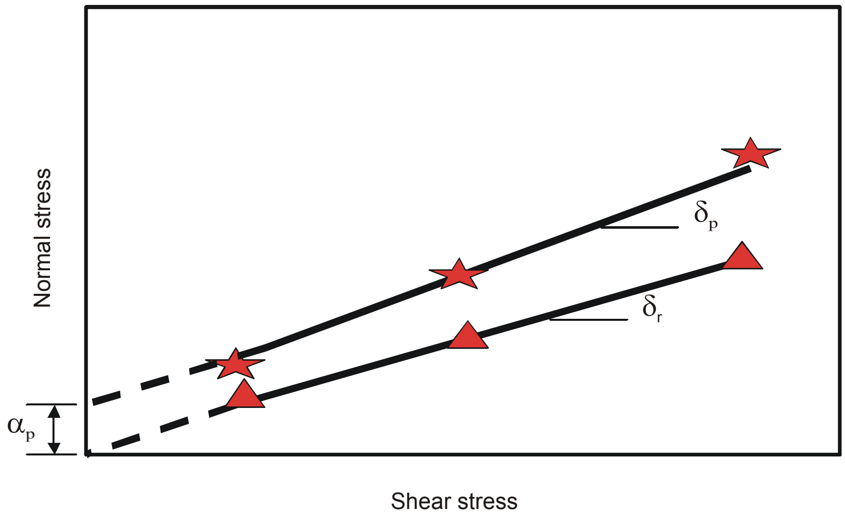


Figure 7.2 Peak (p) and residual (r) failure envelopes and derivation of the shear strength parameters α and δ

For soils, the failure envelope may show some slight curvature, particularly under low normal stresses. The same is true for some geosynthetic interfaces, however, a straight line approximation can still be taken over the stress range relevant for design and the interface shear strength parameters determined for that range. This potential curvature of the envelope means that it is important to carry out tests at the relevant stress level and not to extrapolate the failure envelope from tests carried out at appreciably higher or lower stresses.

7.2 Methods of Measurement

7.2.1 Introduction

The measurement of geosynthetic interface shear strength can be carried out by three main methods; direct shear testing, ring shear testing and testing with a tilting table. Direct shear testing can be carried out in standard soil shear boxes with dimensions of 60 mm x 60 mm and 100 mm x 100 mm which can be regarded as index testing, or can be more performance-related using larger 300 mm x 300 mm and 300 mm x 400 mm direct shear apparatus (DSA). All direct shear apparatus have limited displacements and it has been shown (Jones, 1999) that even displacements of 100 mm may not mobilise the true residual interface shear strengths. Figure 7.3 shows a schematic cross-section through a direct shear device.

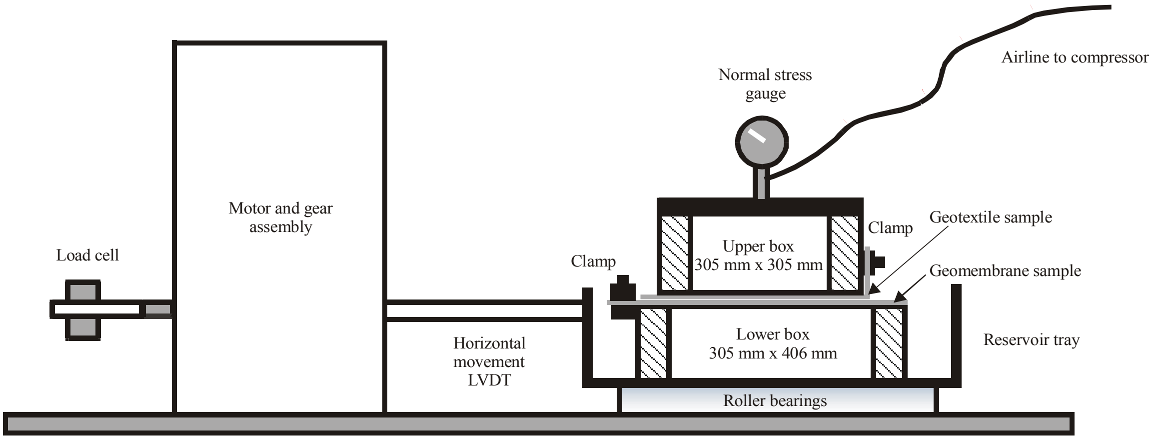


Figure 7.3 Typical details of a direct shear device

Ring shear testing can be carried out to investigate the true residual strengths since the apparatus can produce unlimited displacements. It should be recognised, however, that the direction of shearing in a ring shear test is not comparable to the field and thus true residual shear strengths may only be of academic interest and the large strain strengths obtained from a direct shear test in a 300 mm x 400 mm apparatus may be sufficient for design applications. In addition, ring shear testing should not be used to measure peak interface shear strengths due to non-uniform strains across the shear surface (Dixon & Jones, 1995). The third main method of measurement is the use of a tilting table which has been used predominantly in Europe. There is currently no consensus on the size of apparatus required to provide performance results and its use is limited to low normal stresses. However, the tilting table may be more accurate in determining the behaviour of geosynthetic interfaces at low confining stress and assessing creep behaviour. Publication of a European standard is imminent. There are currently no UK operators of a tilting table.

Parameters for use in design are obtained by carrying out performance tests. This means using site specific materials and relevant boundary conditions; such as direction of shearing in relation to manufacturing process (i.e. roll direction), using site specific cover soils and appropriate moisture conditions. There are currently three standards in use that provide guidance on testing procedures; BS 6906:1991, ASTM D5321-92 and a German recommendation for landfill design GDA E 3-8 of 1997. A fourth standard, the final version of a preliminary European standard (prEN WI 00189015) is imminent. In addition, a significant number of research papers have been published on this topic in the past 15 years.

It would appear therefore that there is adequate information and guidance to ensure high quality testing is carried out. However this is not the case. There is growing evidence that tests specified to obtain parameters for design, and those reported in the literature, often lack sufficient control on the key factors affecting the measured values. This is resulting in uncertainty regarding the likely variability of measured shear strengths, and in some instances is leading to the use of un-conservative (i.e. high) interface strengths in design (see Section 7.4). A brief summary of some of the key factors controlling measured interface strength is given in Section 7.2.4, which is taken from a technical paper by Stoewahse *et al.* (2002).

7.2.2 Test procedure

The four standards listed in the introduction are available to provide guidance on testing procedures and evaluation of measured data. These standards provide some useful guidance for both the designer and operator. The ASTM gives guidance for performance testing of soil vs. geosynthetic and geosynthetic vs. geosynthetic interfaces. BS6906 Part 8 essentially covers only index tests on these two types of interface, although limited guidance on performance testing is provided in Appendix A. The proposed European standard is restricted to index tests on standard sand vs. geosynthetic interfaces. The BS and ASTM are in the order of ten years old and therefore do not include recent developments, and the proposed CEN document is of limit use for designers, as it only covers index testing. GDA E3-8 is specifically devoted to landfill design and gives detailed recommendations for performance testing of all kinds of interfaces for liner systems and covers, although it is presently not available in English.

Table 7.1 summarises the scope and guidance provided by the test standards. This section provides a brief summary of aspects of the guidance given by each of these standards and comments on key elements of the test procedure, including references to papers detailing relevant research. Example results are given showing the influence of selected factors of the test procedure.

None of the guidelines specifies the construction of the testing device although detailed specification of the DSA exists in all the standards for direct shear tests on soils. A comprehensive study by researchers at Hanover University (Bluemel & Stoewahse 1998, Bluemel *et al.* 2000 and Stoewahse 2000) has shown that the design of the shear box has a controlling influence on the results obtained. They conclude that a device where the top box is allowed to move vertically, but not rotate, gives correct and consistent results. Boxes with a fixed top box were found to produce un-conservative (i.e. high) shear strengths for some interfaces. The difference is related to variations in the normal stress acting on the interface. At present the fixed top box is the most common design used in the UK, and also in Germany and the USA. Therefore, it is important that a full description of the testing equipment is provided together with the test results. The investigating laboratory should comment on the key question of how the effective normal stress on the interface is calculated or measured during shearing. While the issues included here are important for the assessment of all geosynthetics, there are specific additional considerations for the testing of geogrids, geonets and geosynthetic clay liners (GCL) that are not covered.

Table 7.1 Key elements of interface test standards

Standard	BS6906:1991	PrEN WI00189015	ASTM D5321.92	GDA E 3-8
Scope	Index tests + some guidance on performance testing	Index tests only	Performance tests	Performance tests
Test Apparatus	DSA 'about 300mm square'.	DSA minimum shear area 300mm square.	DSA minimum shear area 300mm square.	DSA minimum shear area 300mm square, for geosynthetics without surface structure and fine grained soil 100 mm square.
Specific requirements of DSA	σ_n applied through rigid load plate. Measure vertical deformations. Design of box not specified.	Design should allow for sand dilation, $\sigma_n \pm 2\%$ Fluid filled membrane systems allowed for application of σ_n. Measure vertical movement of loading plate at end of test.	σ_n applied by device that maintains a constant uniform σ_n for duration of test $\pm$ 2% Design should allow for soil deformation during shearing.	Design of DSA not specified Measurement of normal and friction stresses and of vertical movement Calibration measurements recommended to determine the stress acting in the friction plane.
Number of Tests conducted	9 tests in total, σ_n = 50, 100 and 200, kPa (3 tests at each σ_n) Highlights need to conduct tests in different directions and on different sides of geosyn.	4 tests in total, σ_n = 50, 2 x 100 and 150 kPa.	Minimum of 3 σ_n, user defined. Test different directions and sides.	3 tests with 3 different normal stresses and 2 repeating tests with the mean value, which should match the expected normal stress in situ.
Material Conditioning	Sand and geosyn. 20° ± 5°C	Sand and geosyn. 20° ± 2°C Humidity 65% ± 2% if applicable.	Soil and geosyn. 21°C + 2°C Humidity 65% ± 5% if applicable	Soil mechanical laboratory conditions
Method of fixing geosynthetics	Clamp or glued to rigid sub-stratum	Fix geosyn. to rigid support to prevent any relative dis-placement between specimen and support (e.g. glue, friction support in shear area or clamped outside area).	Clamping outside shear area or gluing to rigid sub-stratum.	Recommendations about support and fixation of geosynthetics depending on the individual test case.
Soil Properties	Complying with fraction B (1.18mm to 600 μm) BS 4550 Compacted dry ρ_d = 1.65 → 1.7Mg/m^3 Performance tests, compact soil at w_{nat} to 92 ± 2% ρ_{dmax}.	Standard sand in accordance with EN 196-1 (1.6mm to 0.08mm) Compacted w of 2% to ρ_d=1.75Mg/m^3	User defined Take care not to damage geosyn. during placement. Measure ρ and w after test.	Cohesive soils with not more than 95% $\rho_{Proctor}$ 'on the wet side' or as proposed by the landfill designer. Not less than 24 h preconsdolidation time under normal stress equal to the test. Noncohesive soils compacted to medium density or as proposed by the landfill designer.
Maximum particle size and Gap size (top/bottom base)	Sand vs. geosyn. (index) not specified. Soil vs. geosyn. (performance) gap is ρ_{85}/2 or 1mm for fine grained soils. Maximum particle size < 1/8th box depth. Geosyn. vs. geosyn. gap not specified.	Maximum particle not applicable. Gap size = 0.5mm.	Maximum particle size < 1/6th box depth. Soil vs. geosyn. gap ≥ d_{85} of soil. Geosyn. vs. geosyn. gap not specified.	Maximum particle size d_{85}< 1/15th of box length. Gap size is depending on test materials and has to be chosen so that there cannot develop additional normal forces by the frame and secondary friction planes; chosen gap size has to be reported.
Location of materials in DSA	Geosyn. vs. geosyn. rigid sub-stratum (i.e. not soil) Soil vs. geosyn. Either rigid sub-stratum, geosyn. Or soil in top box. Depth of soil layer not specified.	Sand vs. geosyn., rigid sub-stratum in bottom box and sand in top box. Depth of sand layer = 50mm.	Geosyn. vs. geosyn. rigid sub-stratum (i.e. no soil). Soil vs. geosyn., geosyn. supported by rigid sub-stratum. Soil either in top or bottom box. Depth of sand layer not specified.	Geosyn. vs. geosyn. rigid sub-stratum (i.e. normally no soil). Soil vs. geosyn., geosyn. supported by rigid sub-stratum. Soil either in top or bottom box. Depth of soil layer not specified.
Shearing rate	Geosyn. vs. geosyn. and sand vs. geosyn. (index) 2mm/min. Soil vs. geosyn., variable rate depending on drainage.	Sand vs. geosyn. 1mm/min.	Geosyn. vs. geosyn., 5mm/min if no material specification. Soil vs. geosyn., slow enough to dissipate excess pore pressures. If no excess pore water pressures expected use 1mm/min.	Geosyn. vs. geosyn and non cohesive soil vs. geosyn., 0.167 to 1 mm/min. Geotextile vs. cohesive soil 0.167 mm/min. Geosyn. liner vs. cohesive soil 0.005 mm/min.
Derivation of shear strength parameters	Obtain δ_p, δ_r from best fit straight line through all 9 points. Disregard any apparent adhesion (α) values.	Best fit straight lines through all points (peak and residual) to obtain, δ_p, δ_r, α_p and α_r	Failure envelopes defined by best fit straight lines to obtain strength parameters δ_p, δ_r and Y intercepts.	Tests should be performed independently by a second institution. Best fit straight lines through all points (peak and residual) to obtain test values of δ_p, δ_r, α_p and α_r. Derivation characteristic values. Disregard any apparent adhesion (α) values for noncohesive soils and for cohesive soils in special construction cases.
Specific reporting requirements	All plots and calculations. Describe failure mode. Report φ′ of sand.	'For comparison of index test results, all graphs and data have to be submitted to judgement of an engineer.' Description of 'post peak behaviour observed in each test'.	All plots and calculations	Detailed report about the test equipment, procedures and observations during testing, about the measured data and the further evaluation.

7.2.3 Test set up

In fixed top box DSA the gap between the top and bottom boxes must be set prior to shearing. Advice from the test standards is both ambiguous and outdated. The gap size must be so small that no soil particles can migrate out of the box but it must also be large enough so that no constraints are induced. Bemben and Schulze (1998) demonstrated that the gap size has a significant affect on the measured strength. A gap size of d_{85} was shown to be too small, resulting in high peak and residual strengths. This means that use of the gap sizes specified in ASTM D5321 and BS6906 can lead to significant errors. Unfortunately they did not describe the type of DSA they used. In the tests with the vertically movable and tilting top boxes the box heaves up to 1 mm during shear. In this type of box there is a immediate relief of constraints if the gap is too small. It should be considered that the accuracy with which the gap can be adjusted is not less than 0.5mm in a 300mm square DSA. This does not take into account the compressibility of a geosynthetic in the lower box.

The thickness of a soil layer placed in the top box also has a direct influence on measured strengths when using a fixed box design. This layer thickness is not specified in the BS and ASTM, and no guidance is given. In fixed top boxes, the soil layer thickness will have a direct influence on the change in normal stress at the interface that occurs during shearing (Bemben and Schulze, 1998). The thicker the layer of soil, the larger the influence. Conversely, thin soil layers can restrict the correct development of interface shear strength. General dimensions of acceptable soil layers cannot be given because they are dependent upon the particle size distribution of the soil. In tests on a sand-geotextile interface Stoewahse (2000) varied the thickness of the sand layer in the top box. The tests were performed in a 300mm square DSA with a fixed top box.

Tests were conducted using both a rough and a smooth load plate and the results are shown in Figure 7.4. When a rough load plate was used, no significant differences in the test results can be seen for a soil layer thicker than 50mm. When the sample was thinner than 50mm, no peak values were observed. Rotation of the load plate also affects the results. At a thickness of less than 20mm the load plate rotated significantly. These effects are more obvious if the load plate is smooth. In general, a sample thickness of at least 50mm is sufficient for non-cohesive soils. To reduce consolidation times in drained tests, for cohesive soils the sample thickness can be reduced to 30mm. A rough load plate with ribs is recommended in every case.

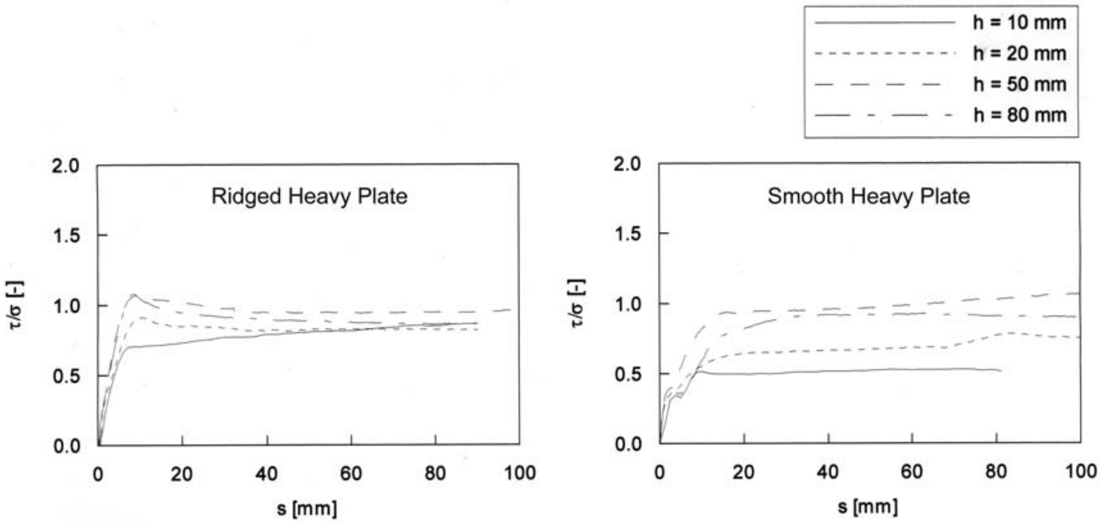


Figure 7.4 **Shear stress vs. displacement curves for a sand vs. geotextile interface using different sample thickness and load plate roughness (Stoewahse 2000)**

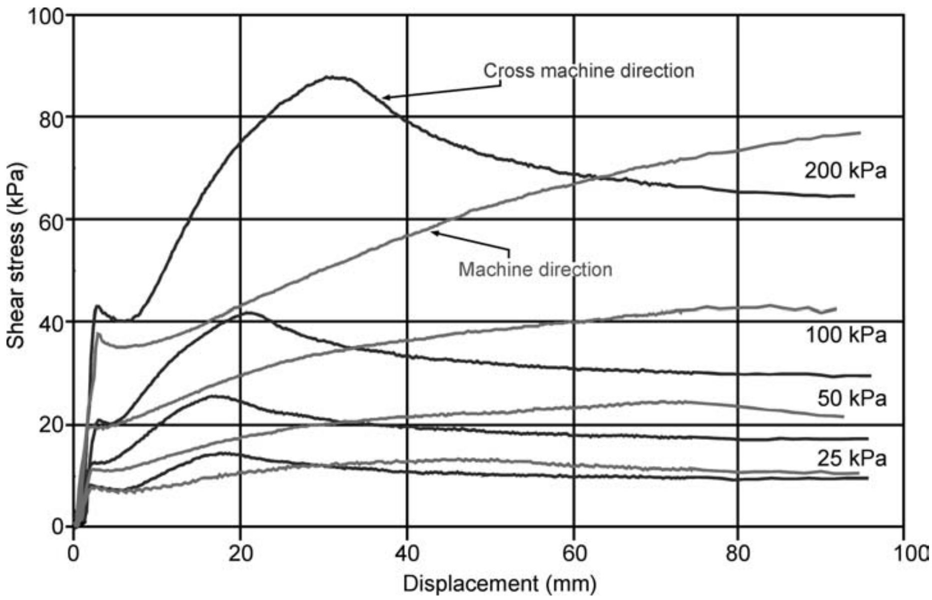


Figure 7.5 **Influence of material stretching and direction of testing on shear test results**

Geosynthetic elements must be restrained in order to ensure that stretching does not occur. Figure 7.5 shows examples of tests on geotextile vs. geomembrane, in one set the geotextile was sheared in the machine direction and in the other in cross machine direction. The different behaviour in the two directions is a result of the material having different tensile stress vs. strain relationships caused by the manufacturing process. The post-peak shape of the stress/strain curves (i.e. strain hardening) and the magniture of the measured peak and residual values are modified in all tests due to stretching of the geotextile.

Tests must be carried out in a temperature controlled environment (20°C ± 2°C) and using materials conditioned in this temperature range to ensure consistency of results. Pasqualini *et al.* (1993) demonstrated that temperature has an affect on the shear resistance of geosynthetic interfaces, with increased temperatures leading to increased interface shear strength. Thus although temperatures in landfills may exceed 20°C, it is likely that in situ interface shear strengths will be greater than those measured in the laboratory.

Although water is likely to be present at all geosynthetic interfaces at some time in their design life, it is common for dry DSA tests to be specified. Pasqualini *et al.* (1993) present results that show interface shear strengths between geomembranes and geotextiles are reduced in the presence of water. This is demonstrated in Figure 7.6 using results for a textured geomembrane vs. non-woven geotextile. From these results it could be concluded that all testing should be carried out submerged. However, the results in Figure 7.7 for a smooth geomembrane vs. non-woven geotextile show that submerging this interface (i.e. introducing water at a low pressure head) increases the measured shear strength. For this interface, specifying submerged tests as a standard would result in un-conservative high strengths being used in designs for dry conditions.

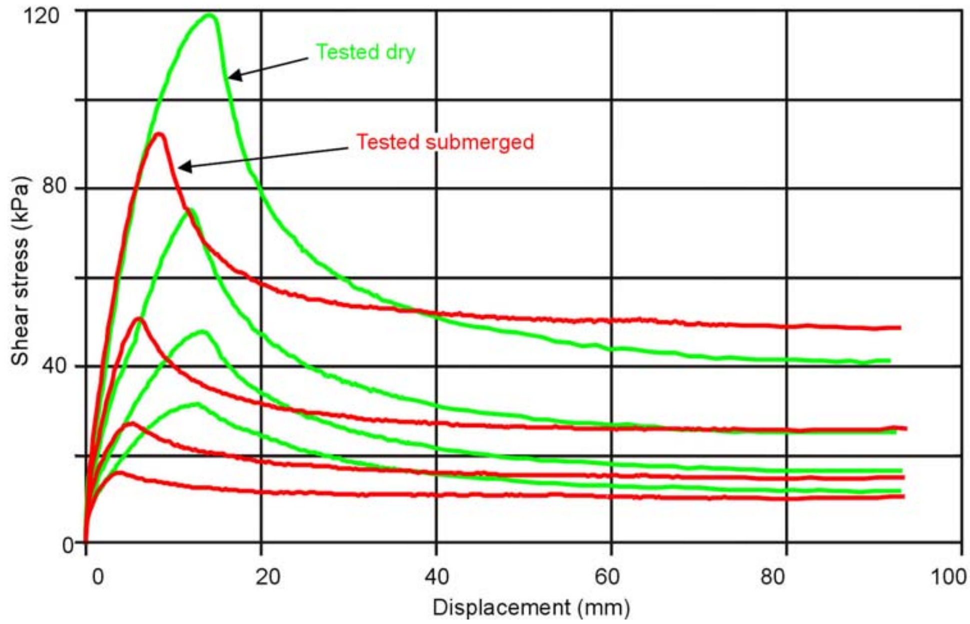


Figure 7.6 Shear tests on textured geomembrane vs. geotextile under dry and submerged conditions

In performance testing of geosynthetic vs. geosynthetic interfaces it is important to use site specific soils in the top box (i.e. overlying the upper geosynthetic). For example, Jones and Dixon (1998a) showed that grading, particle size and particle shape have a direct influence on the shear strength of a geomembrane vs. non-woven geotextile interface. The soil overlying the geotextile controls the distribution and value of the normal stress at the contact between the two geosynthetics. The soil in the top box also controls stretching of the upper geotextile.

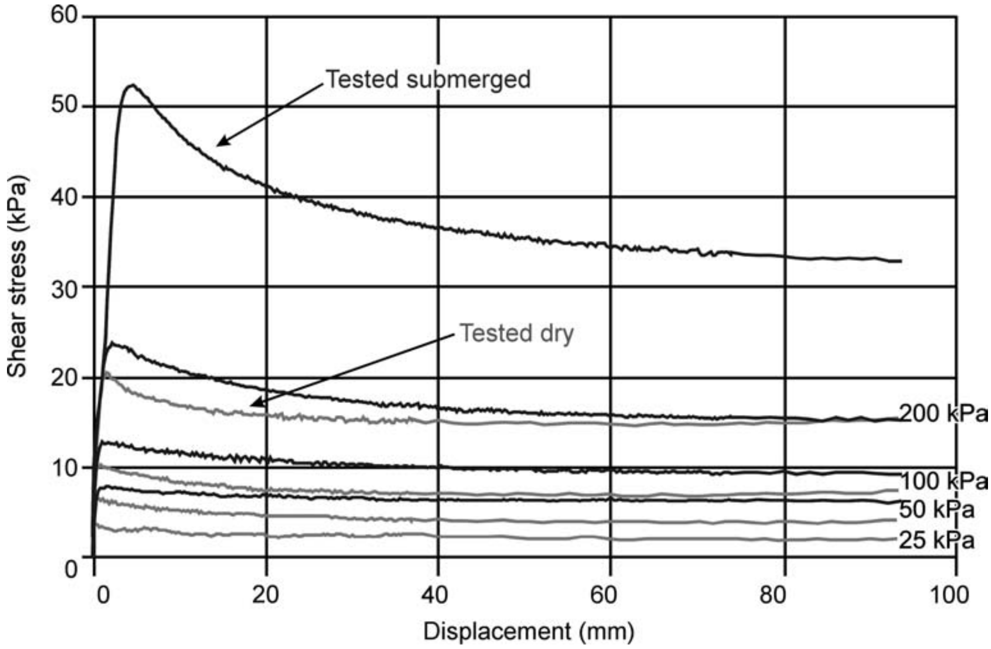


Figure 7.7 Shear tests on smooth geomembrane vs. geotextile under dry and submerged conditions

The important role of the material used in the top box is demonstrated by the results shown in Figure 7.8. Tests were conducted to measure the shear strength between a cuspated drainage core and a heat bonded geotextile filter. The test with a nylon block used to apply the normal stress to the interface produced a low initial peak value, and a shear stress vs. displacement distribution typical of geotextile stretching. This result is inadequate even for an index test. When sand was used above the interface, clearer and higher peak and residual shear strengths were measured. There is no evidence of stretching. This result is a reliable index test. The third test is for clay compacted in the top box onto the geotextile. This reproduces the field conditions of the particular site being studied.

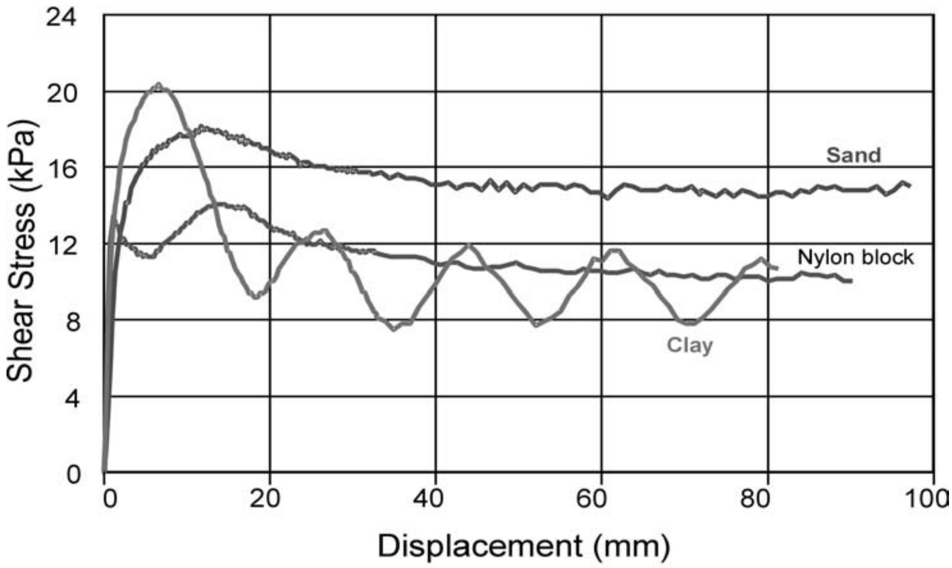


Figure 7.8 Shear test results for a cuspated core material vs. heat bonded geotextile with different cover material

Figure 7.8 shows the higher peak and lower (oscillating) large displacement strengths of the compacted clay test. This is a performance test and can be used to obtain strength parameters for use in design. Use of the sand overburden results would under-estimate the peak strength and over-estimate large displacement strength.

The shearing rates specified in the standard tests for geosynthetic vs. geosynthetic and sand vs. geosynthetic tests are appropriate. Figure 7.9 shows peak shear stresses for a sand vs. geotextile interface obtained at different shearing rates. The test were conducted in a DSA with fixed top box at a normal stress of 100 kPa at shearing rates between 0,0167 and 2 mm/min (Stoewahse 2000). Stark *et al.* (1996) performed ring shear tests on a geotextile vs. geomembrane interface at shearing rates between 0,03 and approximately 40 mm/min at a normal stress of 96 kPa. For both interfaces no significant effect of shearing rate on the shear stress was observed.

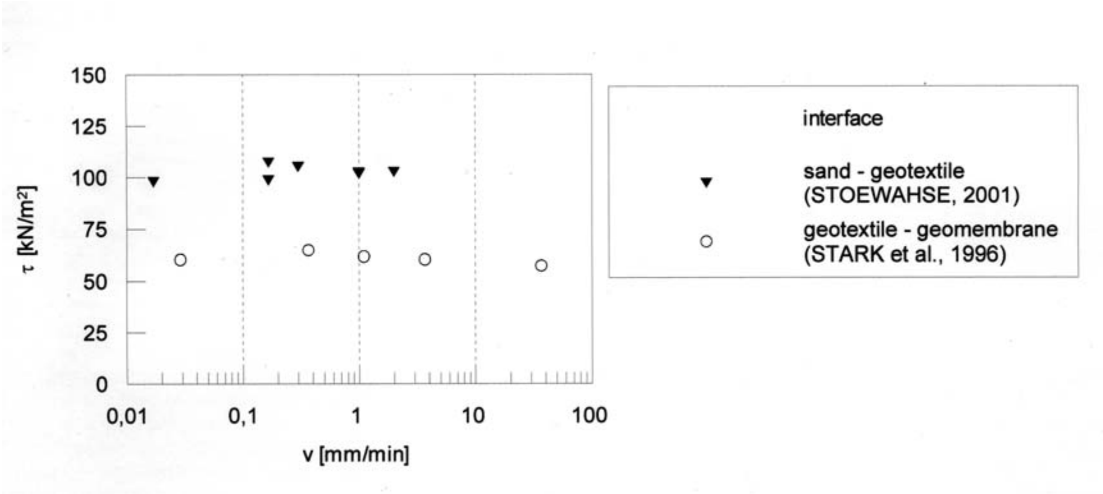


Figure 7.9 Peak shear stresses against shear rate for a sand vs. geotextile

However, for performance testing, appropriate shearing rates must be specified according to the critical conditions expected on site (i.e. drained or undrained). Drained tests can take many hours or even days when involving cohesive soils and therefore are seldom carried out, although effective strength parameters are often required in design. In the German guideline on friction tests for cohesive soil vs. geomembrane interfaces drained tests are compulsory. Results for interfaces involving cohesive soils published in the literature rarely provide adequate information to interpret the drainage conditions during shearing. Soil mechanics principles must be considered and followed in geosynthetic interface testing.

7.2.4 Summary of factors influencing test results

Key factors influencing measured interface shear strengths include:

- design of the direct shear device (i.e. fixity of top box, method of applying normal stress);
- test set up (e.g. method of clamping and restraining the geosynthetics, gap size between the top and bottom boxes, dry or submerged conditions, type of material used in the top box to transmit the normal stress to the interface, shearing rate, temperature and normal stress range);
- material variability (i.e. direction of shearing, number of tests required to obtain representative values – see Section 7.5); and

- soil mechanics principles (density of soil, maximum particle size, consolidation properties, drained or undrained shearing, value of pore water pressures, volume changes).

7.3 Typical Values

A summary of interface shear strengths from the literature for the most common interfaces was presented by Jones & Dixon (1998b), and this is reproduced below. The following summary is presented as interfaces with smooth geomembranes, textured geomembranes and non-woven geotextiles. These values from the literature should not be used in design. Performance tests must always be carried out using site specific materials and conditions.

7.3.1 Smooth HDPE geomembrane

The results of testing on smooth HDPE geomembranes are presented in Figure 7.10 and a summary is given in Table 7.2 below.

Table 7.2 Summary of results for smooth HDPE geomembrane

Interface	Interface shear strength parameters					
	Peak			Residual		
	δ (°)	α (kPa)	R^2	δ (°)	α (kPa)	R^2
Geonet	9.0	1.0	0.74	6.9	1.8	0.80
Non-woven geotextile	9.8	-0.8	0.88	5.8	0.3	0.88
Sand	26.9	-4.0	0.90	16.2	0.0	0.95
Clay – undrained	10.3	7.1	0.48	2.3	15.0	0.09
Clay – drained	21.5	2.1	0.86	17.1	-6.1	0.97

The summary plot of shear stress vs. normal stress for a smooth geomembrane/geonet interface (Figure 7.10a) shows a scatter in data points with a poor straight line fit for both peak and residual conditions with R^2 values of 0.74 and 0.80 respectively. This linear regression gives a peak friction angle of 9.0°, which reduces to 6.9° at large displacements. This interface has low adhesion intercepts for both peak (1.0kPa) and residual (1.8kPa) conditions. For the smooth geomembrane/non-woven geotextile interface, a peak interface friction angle of 9.8°, reducing to 5.8° for residual conditions (Figure 7.10b) is calculated; there is negligible adhesion intercept for this interface. Both peak and residual conditions give strong straight line fits both with correlation coefficient values of 0.88, however there is still a degree of scatter in the results (Figure 7.10b).

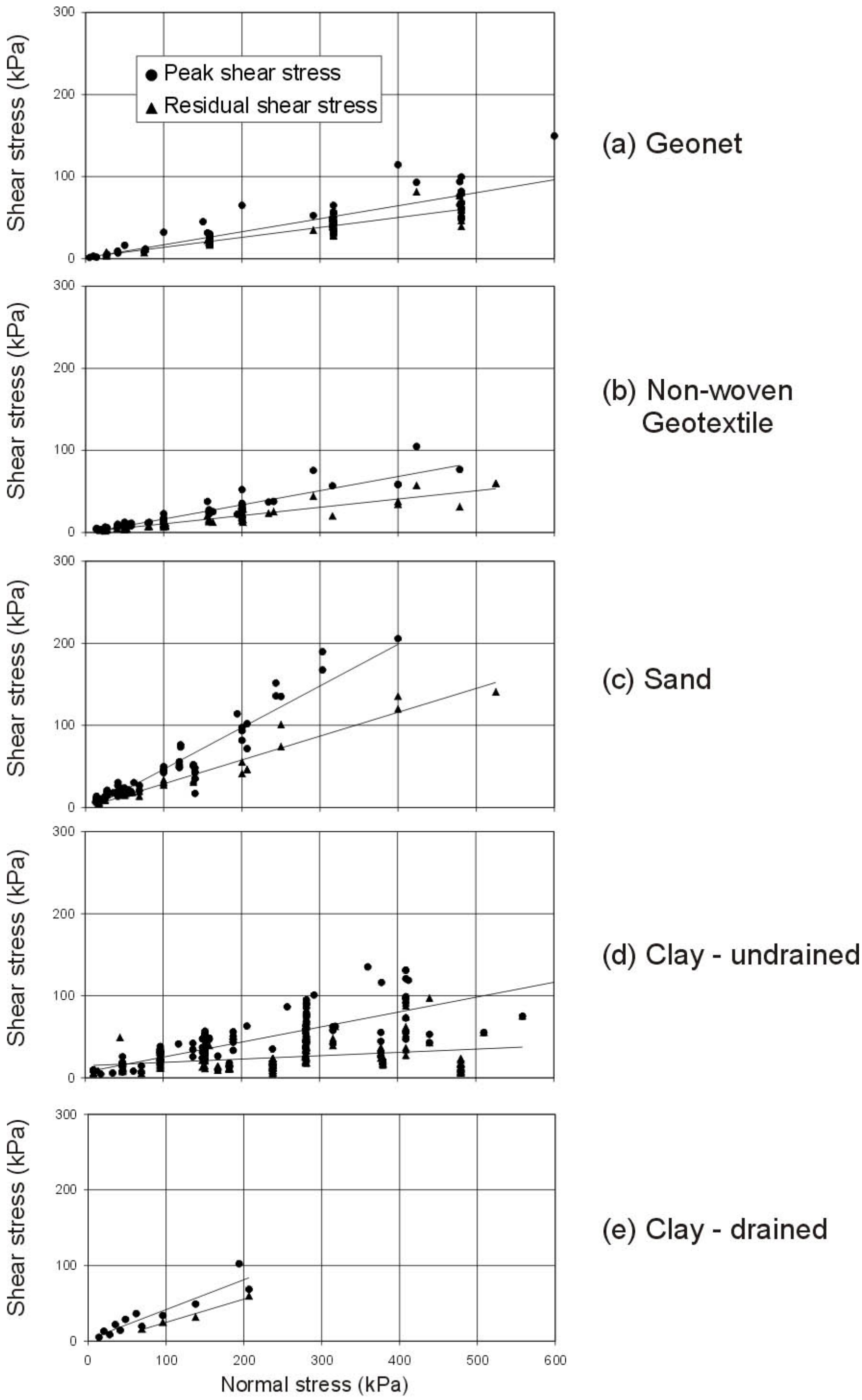


Figure 7.10 Summary of interface shear strength results – smooth HDPE geomembrane

The smooth geomembrane/sand interface has much higher shear strength than the two interfaces discussed above. The peak interface shear strength using linear regression is δ = 26.9° and α = -4.0 kPa, and there is a good straight line fit with R^2 = 0.90 (Figure 7.10c). The residual values give slightly less scatter and thus a higher correlation coefficient of 0.95, and a residual friction angle of 16.2°.

Testing of the interface shear strength between geosynthetics and cohesive soil is more difficult than the testing of geosynthetic/geosynthetic or geosynthetic/granular interfaces, since there is the possibility of pore water pressures at the interface during shearing. Such pressures may be positive or negative (suctions) and will lead to an increase or decrease in effective stress at the interface thus making the assessment of interface shear strength more difficult. The assessment of whether the results quoted in the literature are based on undrained or drained conditions is based on either the various authors' descriptions or on an interpretation of the shearing rates used by the current authors. It is considered that the results presented may not be true undrained or drained conditions and thus caution is required when assessing the results.

For undrained tests it may be that the interface shear strength will be dependent on the undrained shear strength of the clay and/or the moisture conditions at the interface. However, not all authors reported the clay strength and this makes any accurate assessment of the results difficult, if not impossible. The scatter in results for smooth HDPE geomembrane/clay interface (Figure 7.10d) is not unexpected. Correlation coefficients of 0.48 and 0.09 for the peak and residual envelopes respectively demonstrate this scatter. There is a clear increase in shear strength with increasing normal stress with a peak interface shear strength parameters of $\delta = 10.3°$ and $\alpha = 7.1$ kPa. However, the friction angle of the residual envelope is negligible ($\delta = 2.3°$) and the adhesion intercept is 15.0 kPa.

For the drained case the smooth geomembrane/clay interface has less scatter than the undrained conditions (Figure 7.10e). This may be associated with no pore pressures at the interface or may be due to the lower number of data points available. Both peak and residual envelopes have strong correlation coefficients of 0.86 and 0.97 respectively, and the peak interface friction angle of 21.5° reduces to a residual value of 17.1°. The adhesion intercept reduces from 2.1 kPa for the peak to -6.1 kPa for the residual shear strength. Since the residual envelope is only based on four data points it is not considered to be representative.

7.3.2 Textured HDPE geomembrane

The results of testing on textured HDPE geomembranes are presented in Figure 7.11and a summary is given in Table 7.3 below.

Table 7.3 Summary of results for textured HDPE geomembrane

Interface	Interface shear strength parameters					
	Peak			Residual		
	δ (°)	α (kPa)	R^2	δ (°)	α (kPa)	R^2
Geonet	11.0	3.0	0.98	9.1	9.2	0.96
Non-woven geotextile	25.8	6.9	0.88	13.1	3.6	0.88
Sand	27.4	6.9	0.96	25.5	15.5	0.90
Clay – undrained	4.4	36.0	0.13	3.1	34.0	0.21
Clay – drained	10.7	26.7	0.93	-	-	-

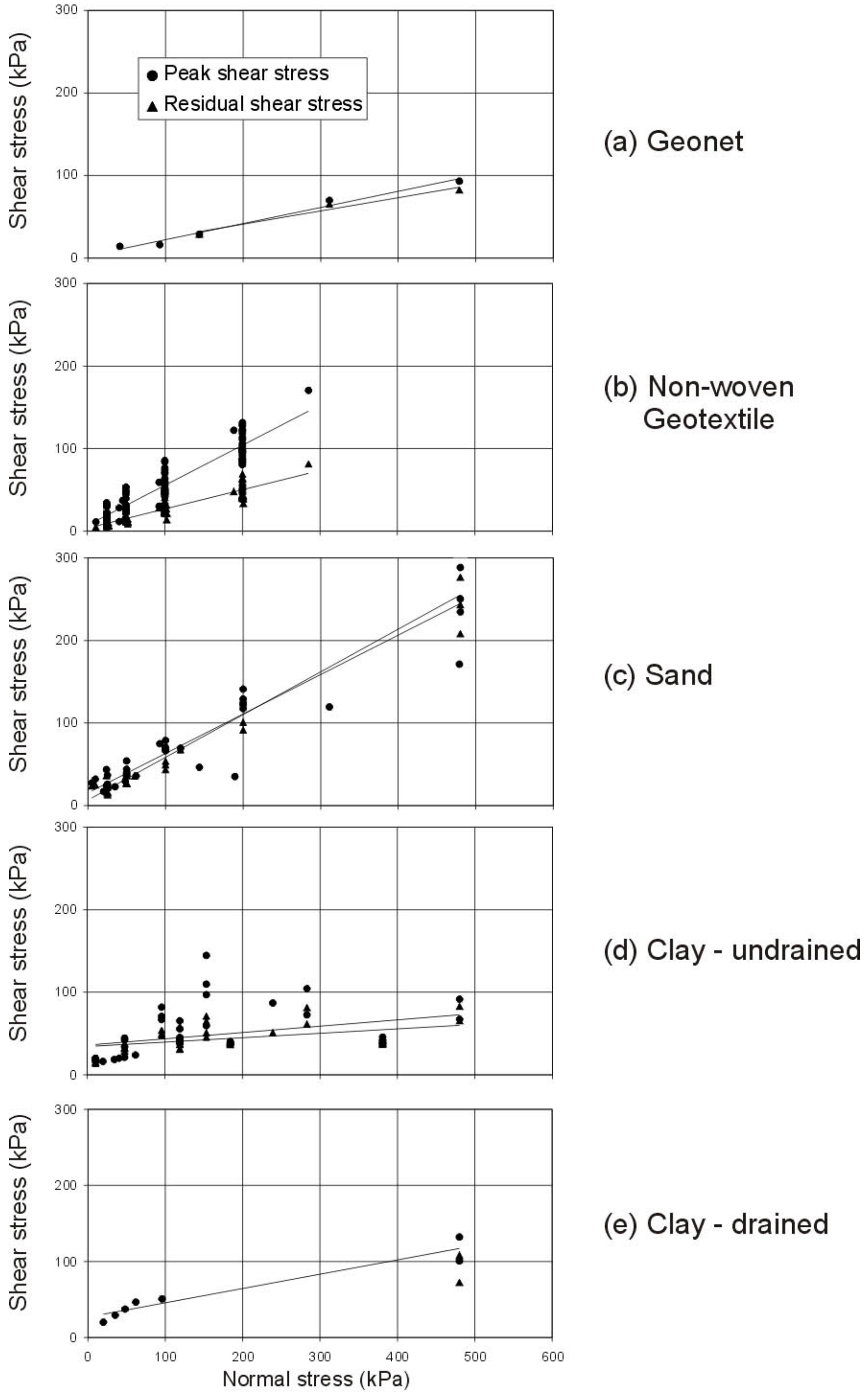


Figure 7.11 Summary of interface shear strength results – textured HDPE geomembrane

The information available on the interface shear strength between textured HDPE geomembranes and geonets is limited and this may be because the increase in interface shear strength over and above the smooth geomembrane is marginal. Figure 7.11a summarises the available information, although there are only five data points for the peak strength and three points for the residual strength. The peak interface shear strength based on this data is $\delta = 11.0°$ and $\alpha = 3.0$ kPa with a coefficient of determination of 0.98, which compares with a friction angle of 9.0° for the smooth geomembrane case (Figure 7.10a). The residual interface shear strength for the textured geomembrane (δ = 9.1° and α = 9.2 kPa) needs to be treated with care since it is only based on three data points.

The majority of data presented for the shear strength of textured geomembrane/non-woven geotextile interfaces is from the results of the testing carried out by the authors (Jones & Dixon, 1998a), although other information from the literature has been used to develop Figure 7.11b. A peak friction angle of 25.8° is obtained together with a adhesion intercept of 6.9 kPa, which reduces to residual values of δ = 13.1° and α = 3.6 kPa, although there is a significant range of values, with R^2 values of 0.88 for both the peak and residual case.

The interface shear strength results for the textured geomembrane/sand interface are shown on Figure 7.11c which give peak parameters of δ = 27.4° and α = 6.9 kPa with a correlation coefficient of 0.96. This interface, although strain softening, does not seem to exhibit a large reduction in shear strength with increased displacement since the residual friction angle is 25.5° with a relatively high adhesion intercept of 15.5 kPa.

From the results of undrained tests on textured HDPE geomembrane against clays (Figure 7.11d), it can be seen that the dependency of shear strength on normal stress is limited with peak and residual friction angles of 4.4° and 3.1° respectively. Adhesion intercepts for both peak and large strain conditions are similar with a peak value of 36.0 kPa and a residual value of 34.0 kPa, however both envelopes give poor linear relationships with R^2 values of 0.13 and 0.21. The shape of the envelopes suggest that the shear strength between textured geomembrane and a clay tested without an allowance for the dissipation of pore pressures is almost independent of normal stress, and is likely to be related to the undrained shear strength of the clay. Since the data shown on Figure 7.11d has been obtained from eight separate references with different clay at different remoulding conditions, the extent of the data scatter is not surprising.

The results shown on Figure 7.11d compare well with the observations made by Orman (1994), who found that failure of a textured HDPE geomembrane/silt interface occurred within the silt along the line of the asperities on the geomembrane sheet. Thus it is to be expected that the undrained interface shear strength of a textured geomembrane/clay is independent of normal stress and probably equal to the undrained shear strength of the clay.
There is little information on geomembrane/clay interfaces tested at strain rates slow enough to dissipate pore water pressures and the data available indicates that the shear strength of this interface is dependent on normal stress (Figure 7.11e). Again the small amount of data available means that caution is required when analysing the results, however, linear regression gives a peak interface shear strength corresponding to δ = 10.7° and α = 26.7 kPa. Closer inspection of the plot reveals that a non-linear fit may be more representative for the peak shear strength envelope, possibly curving downwards at lower normal stresses and passing through the origin. There is insufficient data to determine the residual shear strength for this interface.

It should be noted that the type and degree of texturing can vary significantly between products and that this will influence interface shear strengths with other geosynthetics and with soils. The size, shape and number of asperities influence the interaction with soil grains and geosynthetics fibres.

7.3.3 Non-woven geotextile

The results of testing on non-woven geotextiles are presented in Figure 7.12 and a summary is given in Table 7.4 below.

Table 7.4 Summary of results for non-woven geotextile

Interface	Interface shear strength parameters					
	Peak			Residual		
	δ (°)	α (kPa)	R^2	δ (°)	α (kPa)	R^2
Geonet	13.1	17.9	0.76	15.4	4.1	0.92
Gravel	35.0	-1.0	0.87	19.9	30.1	0.99
Sand	33.0	-1.3	0.93	28.7	7.7	0.92
Clay - undrained	25.3	5.3	0.91	17.7	55.6	0.98
Clay - drained	32.5	4.4	0.98	-	-	-

The results of shear strength testing on non-woven geotextile/geonet interfaces are plotted in Figure 7.12a and linear regression of the all the data points give peak interface shear strengths of δ = 13.1° and α = 17.9 kPa with an R^2 value of 0.76. For the range of normal stresses considered, the residual envelope is similar to the peak in terms of its mobilised shear strength, however the friction angles and adhesion intercept are different. The best fit line through the residual data points is given by δ = 15.4° and α = 4.1 kPa, i.e. a higher friction angle but a lower adhesion intercept with a correlation coefficient of 0.92.

The non-woven geotextile/gravel interface has a high shear strength with some values in the literature reported as high as 48°. Most of the results available are for tests carried out at normal stresses less than 200 kPa (Figure 7.12b) and linear regression gives a friction angle of 35.0° with a adhesion intercept of -1.0 kPa. This reduces to a residual shear strength corresponding to δ = 19.9° and α = 30.1 kPa. The peak shear strength envelope shows a reasonable strong straight line fit with a correlation coefficient of 0.94, while the residual envelope has a very strong fit with R^2 = 0.99, however the residual is based on a small number of data points.

There is much more information available in the literature on the interface shear strength between sand and non-woven geotextiles, and this is also a high strength interface with a peak friction angle of 33.0° and an adhesion intercept of -1.3 kPa (Figure 7.12c). The residual shear strength for this interface is reduced to a value of δ = 28.7° and α = 7.7 kPa. The peak interface shear strength envelope has been generated from over a hundred data points and the scatter is minimal with an R^2 value of 0.91. Fewer data points were available for the residual plot, however the amount of scatter is less with a correlation coefficient of 0.98.

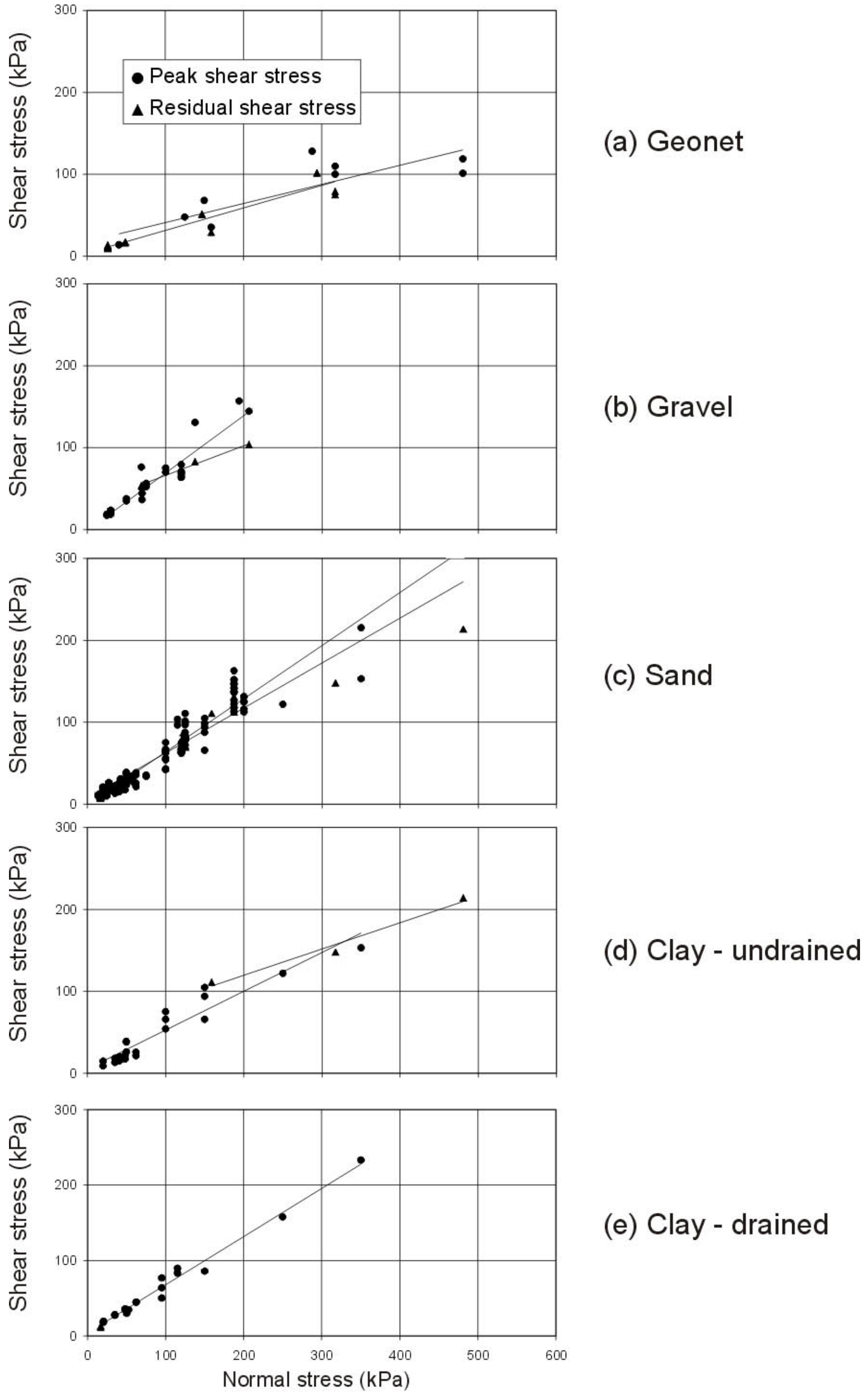


Figure 7.12 Summary of interface shear strength results – non-woven geotextile

The results of undrained tests on non-woven geotextile/clay interface shown on Figure 7.12d. Peak interface shear strengths of δ = 25.3° and α = 5.3 kPa are obtained with a correlation coefficient of 0.91, which reduce to δ = 17.7° and α = 55.6 kPa for large strains. The residual envelope is based on three data points, has an extremely high adhesion intercept and has an R^2 value of 0.98. The peak interface shear strength is predominantly frictional in nature however the high adhesion intercept of the residual envelope could be indicative of dependence on the undrained shear strength of the clay. In particular it may be that the failure plane exists in the outer layer of the geotextiles' fibres which are clay filled, and thus the shear strength is a combination of the fibres' frictional (and possibly tensile) strength together with the clay's strength.

A higher shear strength is obtained for drained tests on non-woven geotextile/clay interfaces, as shown on Figure 7.12e. The summary plot of all data points gives a good straight line fit ($R^2 = 0.98$) for the peak interface shear strength with a high friction angle of 32.5° and a adhesion intercept of 4.4 kPa. There is insufficient information to generate a residual interface shear strength envelope.

7.4 Variability of Results

The variability of geosynthetic interface shear strength parameters is discussed in detail by Stoewahse *et al.* (2002). Research has been conducted to quantify the likely variability of test results and to identify the key factors that control measured strengths. As part of the development of the new European geosynthetic test standard, inter-laboratory comparison tests were conducted in an effort to quantify the likely scatter in measured strengths resulting from the use of different operators and test equipment (Gourc & Lalarakotoson, 1997). Tests were carried out in seven commercial and research laboratories (two each in France, Germany and UK and one in Italy) using geosynthetic materials supplied by the co-ordinator and obtained from one source. The interface shear strengths between a range of geosynthetic materials and standard sand were measured.

Two similar, and complementary, inter-laboratory comparison test programmes were conducted by a working group of the German Society for Geotechnical Engineering in 1995 and 1996, the latter programme with a more detailed specification of the testing procedure, as part of their response to development of the European standard (Bluemel and Stoewahse, 1998). These programmes, each involving approximately twenty laboratories, produced a range of measured strengths that is similar to the European study. Results for a non-woven geotextile vs. sand interface from the German studies are given in Figure 7.13; the significant variability of the curves is typical. The different laboratories produced a range of peak and large displacement shear strengths, and widely varying stress vs. displacement relationships. Figure 7.14 shows the distribution of peak failure envelopes obtained by the laboratories. In addition to the large variation of results, of particular concern is that some laboratories produced high, and hence unsafe, shear strengths.

There are three categories of factors that lead to variability of measured interface shear strength: Test apparatus design; operator/test procedure; and variability of both geosynthetic and soil materials.

Both the European and German test programmes involved the use of a clearly defined common test standard and samples from a common source, but involved different operators and a range of different DSA designs. Hence all three factors are included in the results.

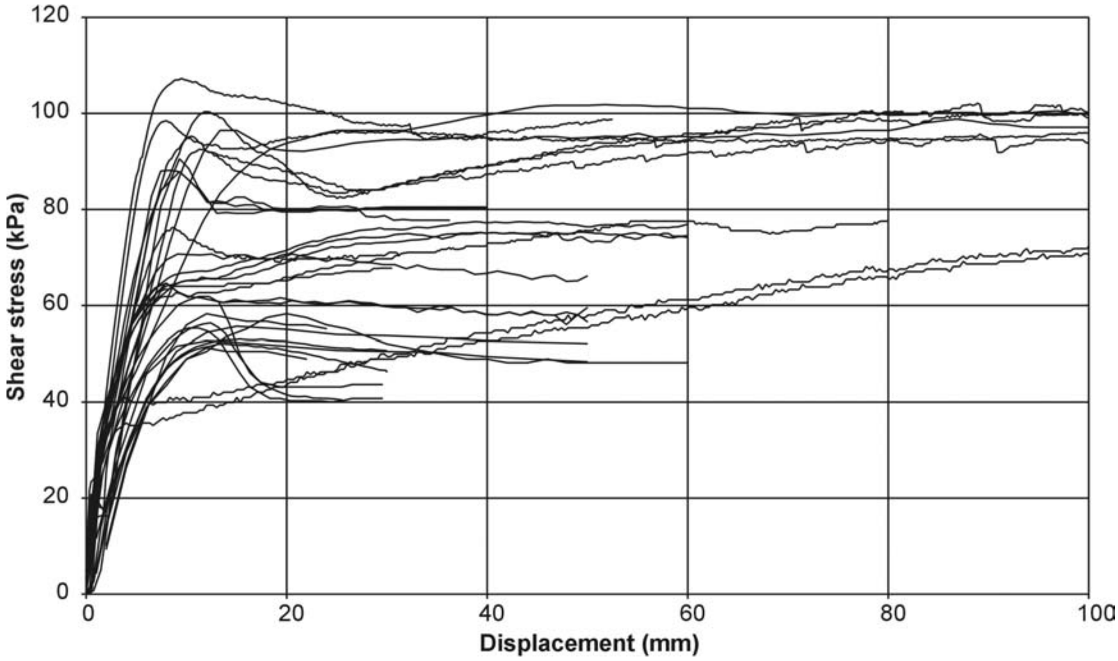


Figure 7.13 Results from German inter-laboratory comparison tests on non-woven geotextile vs. sand interface (Bluemel and Stoewahse 1998)

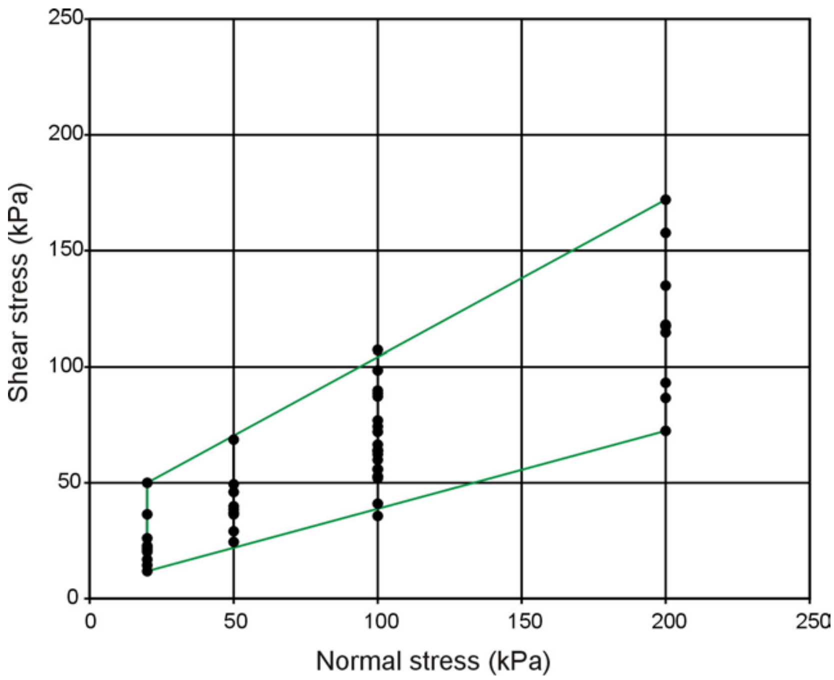


Figure 7.14 Distribution of peak shear strength failure envelopes obtained from test results in Figure 7.13 (Bluemel and Stoewahse 1998)

Repeatability can be improved and the material variability investigated by using one design of DSA and one operator. Test programmes have been carried out under these conditions at Hanover University (Bluemel and Brummermann 1996) and Loughborough University (Dixon *et al.,* 2000). Scatter of results from these tests "under conditions of repeatability" would be primarily due to variation in the geosynthetic and soil test materials. Some results of these studies are shown in Figures 7.15 and 7.16 together with the results of inter-laboratory tests as coefficient of variation (standard deviation/mean) vs. normal stress for interfaces between a sand and a geotextile, as well as between a geotextile and a geomembrane. Each point represents a number of tests on materials from the same source conducted at the same normal stress. The two important trends that can be observed are, reduced scatter of data is obtained if tests are carried out in one laboratory (not surprisingly), and the coefficient of variation increases with decreasing normal stress for all repeatability testing. The latter trend is of practical importance to the design of landfill cover systems. The increased uncertainty in measured interface strengths at low normal stresses should be taken into consideration when deriving design parameters from the test data. Unfortunately, rather than provide confidence in the ability of laboratories to undertake reproducible tests, the results cast doubt on the applicability of aspects of current test procedures.

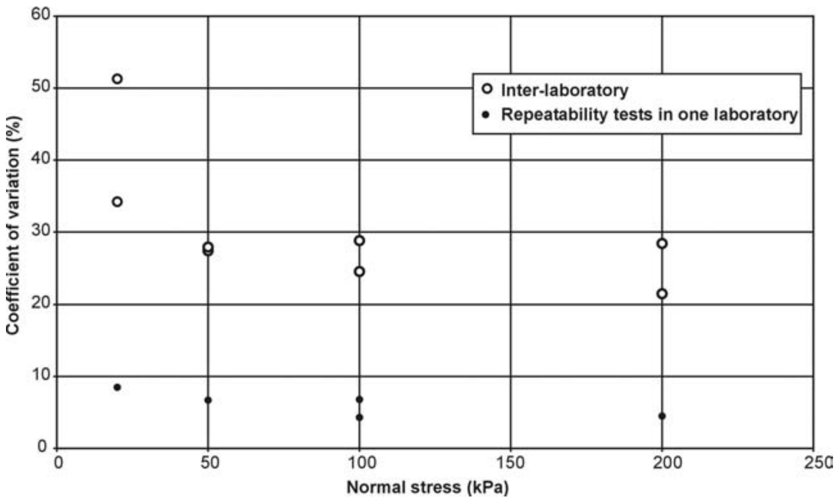


Figure 7.15 Results from inter-laboratory tests and repeatability tests at one laboratory for a sand vs. geotextile interface

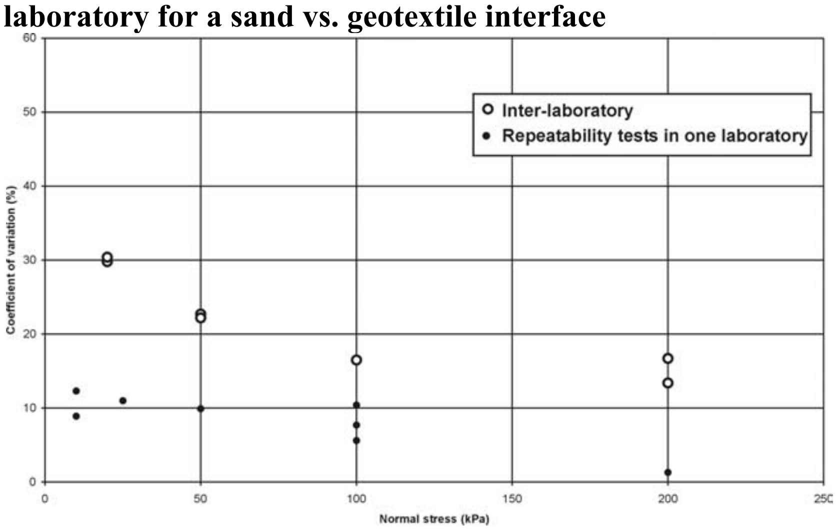


Figure 7.16 Results from inter-laboratory tests and repeatability tests at one laboratory for a geotextile vs. geomembrane interface

7.5 Selection of Characteristic Values

7.5.1 Introduction

Interface shear strength parameters are required for design calculations for assessing the stability of geotechnical structures incorporating geosynthetics. These include the design of landfill barriers and reinforced earth structures. Limit equilibrium calculations can be carried out using a global safety factor (traditional approach) and using partial factors on both resisting and disturbing forces (limit state approach defined in Eurocode 7, 1997). In the global safety factor approach it is necessary to obtain *conservatively chosen mean values* of shear strength. The Eurocode 7 (EC7) approach is to obtain *characteristic values* of shear strength. In both cases a limited number of site-specific laboratory tests is usually supplemented by subjective experience. For practical purposes it can be assumed that *characteristic value* (EC7) and the *conservatively chosen mean value* (traditional) are equivalent (Schneider, 1997), and therefore the recommendations in this paper can be applied to both design approaches. This section is taken from the paper by Dixon *et al.* (2002) and provides guidance on obtaining characteristic values of the interface shear strength parameters (apparent adhesion, α_k and friction angle, δ_k) for use in design calculations.

7.5.2 Characteristic values

Selection of characteristic values of soil and geosynthetic properties must take account of:

- inherent variability of soil;
- inherent variability of manufactured geosynthetic materials;
- measurement errors; and
- extent of zone governing behaviour of limit state being considered.

Measurement errors are a significant factor and are caused by equipment, procedural, operator and random test effects. Some of these factors have been discussed in Section 7.2 and typical variability of measured strengths is considered in section 7.4 and also in detail below.

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). Assessment of an interface between a geosynthetic and soil requires characteristic values of the shear strength parameters that produce a cautious calculated shear strength that allows for variability over the area of the interface involved in the potential failure. Eurocode 7 advises that: *'If statistical methods are used, the characteristic value should be derived such that the calculated probability of a worse value governing the occurrence of a limiting state is not greater than 5%'*.

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):

$$X_k = X_m - 0.5\sigma_m \qquad \text{Equation 7.4}$$

This equation has been in use in Switzerland for several years and has been proven to produce values that are in close agreement with values estimated by experienced geotechnical engineers (Schneider, 1997).

The process of obtaining design parameters is typically: selection of *representative samples* $\rightarrow$ *measured values* (e.g. results of laboratory direct shear tests - peak and residual shear strengths at specific normal stress levels) $\rightarrow$ calculated *derived values* based on theory, empirical relationship or correlations (e.g. obtaining α_m and δ_m values that describe the best fit straight line through the measured strengths) → calculated *characteristic values* α_k and δ_k (a cautious estimate of α_m and δ_m as discussed above) → calculated *design values* α_d and δ_d obtained by applying partial factors to α_k and δ_k.

7.5.3 Derived interface shear strength parameters

Interface shear strength parameters are obtained by plotting peak and residual shear strengths measured in direct shear apparatus on a shear stress vs. normal stress graph. Coulomb failure criteria are defined by best-fit lines through sets of peak (p) and residual (r) data measured at normal stresses relevant to the design problem. Shear strength parameters are used to describe these lines (intercepts α_{pm} and α_{rm}, and slope angles δ_{pm} and δ_{rm}). 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. Given the inevitable scatter of measured interface strengths (see section 7.4), this approach provides insufficient information to enable characteristic strength parameters to be selected. If only one or two tests are conducted at each normal stress, it is not known whether the measured shear strengths are high, low or in between values and the potential scatter of measured strengths is also unknown. Depending upon the position of the measured strengths within the possible range at each normal stress, the best-fit line can have a variety of positions, and hence a wide range of shear strength parameters could be obtained. Figure 7.17 demonstrates possible strength envelopes that can be obtained if a limited number of tests are conducted. The results are from a series of drained repeatability tests conducted on a smooth geomembrane vs. non-woven needle punched geotextile at low normal stresses. The scatter of measured peak shear strengths at a given normal stress is typical of the results obtained in other repeatability test programmes (e.g. textured geomembrane vs. geotextiles).

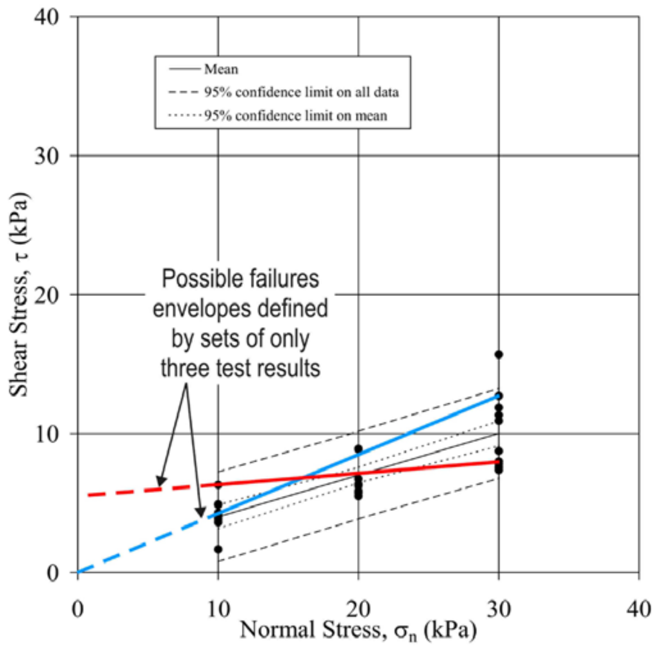


Figure 7.17 Possible interface strength envelopes based on a scatter of data for a smooth geomembrane vs. non-woven geotextile interface

Shear strength envelopes are defined by pairs of apparent adhesion (α) and slope angle (δ) parameters. While it is common practice in soil mechanics to ignore apparent adhesion values in design, this approach is not recommended for geosynthetic interfaces. Apparent adhesion values can be taken into consideration in design of structures incorporating interfaces when they are:

- a measure of true strength at zero normal stress (e.g. the Velcro affect between non-woven needle punched geotextile and textured geomembranes and internal strength of a laminated geocomposite);
- used to define a failure envelope over a range of normal stresses (i.e. assuming a linear failure envelope) when the full envelope curves towards the origin at lower normal stresses; and
- used to define a best-fit straight line through limited variable test data (see Figure 7.17).

In these cases it would be over conservative to assume $\alpha = 0$, especially for design cases with low normal stresses (e.g. design of cover systems). Therefore, as the quantification of interface shear strength requires two parameters (α and δ) it is not appropriate to obtain characteristic values for the shear strength parameters derived directly from the best-fit straight line through the measured values. A methodology is proposed where by characteristic shear strengths are calculated for each normal stress and then these 'corrected' strengths are used to derive characteristic shear strength parameters α_k and δ_k.

7.5.4 Example of interface test data variability

An assessment has been made of the variation in peak strength parameters that can be obtained based on the repeatability data shown in Figure 7.17. A Monte Carlo simulation has been carried out to obtain the distributions of peak strength parameters (α_p, δ_p) that are calculated when sets of three strengths are selected randomly (i.e. one from each normal stress) and a best-fit straight line calculated. The measured distributions of shear strength for each normal stress form the input data for the simulation. These typically can be represented by a normal distribution. A total of 1000 trials were conducted. An example of results from the Monte Carlo simulation for the smooth geomembrane/geotextile test data are shown in Figure 7.18 for the intercept (α_p) and slope (δ_p) values. Table 7.5 contains a summary of the results from simulations in terms of mean and standard deviation of the calculated parameters. In addition, the pairs of shear strength parameters that define each best-fit line have been used to calculate the shear strength for a normal stress of 20 kPa (i.e. typical for a cover system). A summary is given in Table 7.5, also in terms of mean and standard deviation.

Table 7.5 Mean and standard deviation (σ_m) of peak shear strength parameters (α and δ) and shear strength (τ)

Interface	Smooth geomembrane/ geotextile interface	Textured geomembrane/ geotextile interface
Mean α_p (kPa)	1.0	3.7
σ_m (kPa)	2.4	1.9
Mean δ_p (°)	16.2	34.5
σ_m (°)	7.8	3.5
Mean τ (kPa) @ σ_n = 20 kPa	6.9	17.5
σ_m (kPa)	1.1	1.4

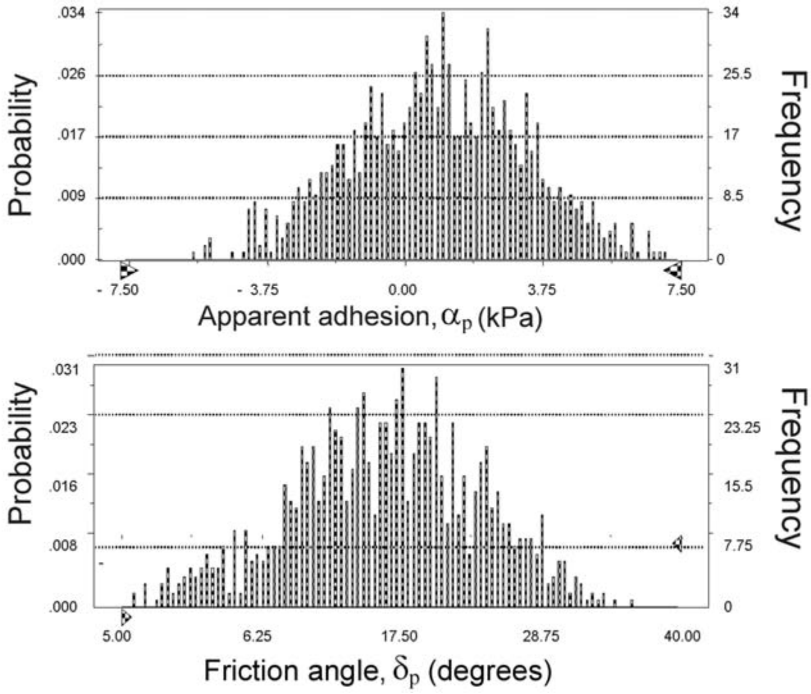


Figure 7.18 Results of Monte Carlo simulation carried out on results from a repeatability test programme

The magnitude of variation in measured shear strengths (Figure 7.17) leads to a wide range of possible failure envelopes and hence the calculated values of shear strength using these failure envelopes (Table 7.5) also have a significant range. This is demonstrated further by Figures 7.19a and 7.19b that show the results of Monte Carlo simulations (as described above) carried out on the results from each of five extensive repeatability/inter-laboratory test programmes. The coefficient of variation of calculated shear stresses (i.e. using generated shear strength parameters defining best-fit straight lines through sets of randomly selected data points) are plotted against the normal stress used in their calculation. Figure 7.19a shows the results from tests on geomembrane vs. geotextile interfaces, and Figure 7.19b the results from sand vs. geotextile interfaces. It can be seen that significant variation of calculated shear strength occurs. Of note is that a larger variation is shown for the sand vs. geotextile tests. This is due to there being additional variation in the test materials, such as resulting from the compaction process used to form the sand test specimens and the wide range of different geotextiles available.

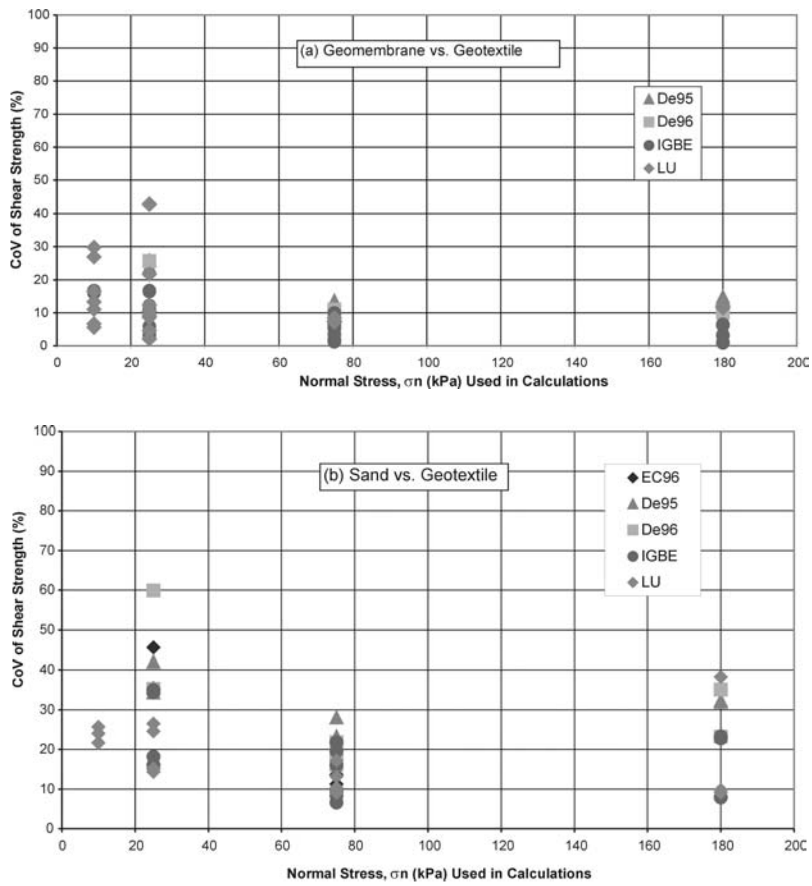


Figure 7.19 Coefficient of variation of calculated shear strengths from Monte Carlo simulations on repeatability and inter-laboratory test programmes a) geomembrane vs. geotextile, and b) geotextile vs. sand

The above analyses show that un-conservative high shear strengths can be obtained from limited test data. This has important implications for selection of characteristic values (α_k, δ_k), as these must provide a cautious estimate of interface shear strength. It is clear that the present common practice of requesting one test at each normal stress is insufficient to calculate a mean value or to assess the variability of measured shear strengths. Hence current practice is inadequate to obtain characteristic interface shear strength parameters. Guidance on selection of characteristic values is provided below.

7.5.5 Guidance on selection of characteristic values

Three approaches for obtaining characteristic shear strength parameters from laboratory test data are summarised below. They are listed in order of preference.

Generation of site-specific statistical data

Selection of characteristic values using a site-specific statistical analysis of test data is the most rigorous approach. It requires multiple performance tests to be conducted at each normal stress to enable the mean (X_m) and standard deviation (σ_m) of measured strengths to be calculated for each stress level. The characteristic shear strengths (X_k) can then be calculated from equation (1). The process is demonstrated in Figure 7.20.

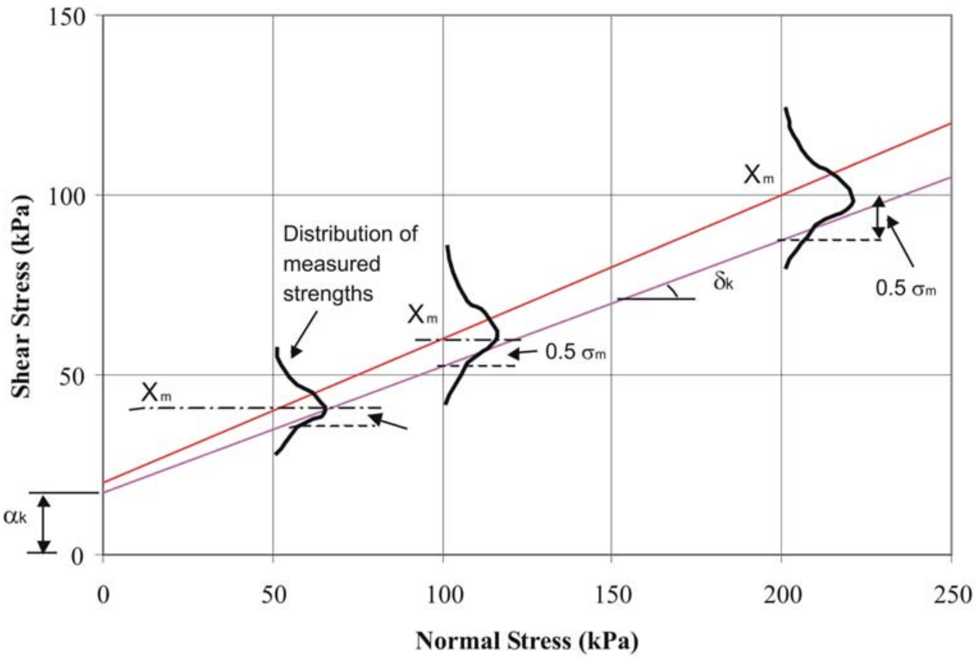


Figure 7.20 Demonstration of the process for evaluation of characteristic shear strengths

Characteristic shear strength parameters (α_k and δ_k) are obtained from the best-fit straight line through the characteristic shear strengths. As outlined above, this approach is based on assessing the variability of measured shear strengths and not the derived shear strength parameters. A sufficient number of tests should be carried out to allow a valid statistical analysis. It is proposed that a minimum of four tests should be conducted at each of three normal stresses (i.e. a minimum of 12 tests in total). However, the number of tests required is also dependent upon the level of existing information relating to the shear strength of the interface being tested. Although, it should be noted that variability of geosynthetics and soils could result in significant differences in shear strength for what appear to be similar interfaces. The level of experience of the engineer interpreting the test results should also be taken into consideration. This approach may appear an expensive option due to the large number of tests required, however experience indicates that significant errors can result from carrying out an inadequate number of tests.

Lower bound of limited repeatability test data

Present recommendations provided by the Germany Geotechnical Society related to the design of water-front structures involving soils is for three tests to be conducted at each of three normal stresses (EAU 1990). The failure line defining the characteristic shear strength parameters is taken as the best fit straight line through the lowest measured strength at each normal stress (i.e. a lower bound to the test data). The selection of three tests is consistent with the guidance in Eurocode 7 (1999) Part 2, Table A.9.2, which suggests carrying out three tests in cases where the results exhibit significant scatter and there exists a medium level of comparable experience. While a smaller number of tests can be carried out than in the preferred method given above, it can lead to over conservative (i.e. low) strength parameters being calculated.

Method based on statistical data from inter-comparison tests

A method of obtaining cautious characteristic values using a limit number of site-specific tests is proposed. The approach is based on an analysis of the variability of measured interface shear strengths from the extensive repeatability and inter-comparison test programmes outlined in Section 7.4. These studies have been analysed to provide statistical information on the magnitude of scatter of measured shear strengths. Two commonly used interfaces are considered: a) non-woven needle punched geotextile vs. geomembrane and b) non-woven needle punched geotextile vs. sand. The first includes results from tests using textured (both co-extruded and blown film types) and smooth high-density polyethylene and low-density polyethylene geomembranes. Results from tests on textured and smooth geomembranes were combined as the coefficients of variation were in the same range. The studies conducted take into consideration the affects of both measurement errors (i.e. equipment, procedural, operator and random test affects) and inherent material variability.

For each of the series of repeatability tests the standard deviation of the measured peak shear strengths has been calculated for each normal stress. The results from these individual tests series have then been combined to calculate a weighted average standard deviation for a range of normal stress levels (i.e. weighted in proportion to the number of tests conducted in each series). The weighted standard deviation data are presented in Table 7.6 for the two interfaces. The data are plotted in Figure 7.21 as weighted standard deviation vs. normal stress. It shows clear linear relationships of increasing variability of measured strengths (indicated by increasing standard deviation) with increasing normal stress. This is demonstrated by the results of tests on both interface types. Significantly less scatter is exhibited by the geomembrane vs. geotextile interface. As discussed above, this is because the interface does not incorporate soil with its inherent variability.

Table 7.6 Statistical analysis of measured shear strengths from repeatability test series conducted on two common geosynthetic interfaces

Geotextile/sand interface						
Normal stress (kPa)			10/20/25	50	100	200
Number of tests			46	45	77	44
Weighted standard deviation, σ_m (kPa)			7.9	12.1	14.4	27.8
Geotextile/geomembrane interface						
Normal stress (kPa)	10	20	25/30	50	100	200
Number of tests	104	76	74	44	70	56
Weighted standard deviation, σ_m (kPa)	1.6	3.0	2.5	7.1	6.6	12.7

It is proposed that where there is insufficient test data for the shear strength of an interface to undertake statistical analysis of its variability, the characteristic values of shear strength for each normal stress should be calculated using the mean measured value of strength (X_m) in conjunction with standard deviation values (σ_m) obtained from the relationships shown in Figure 7.21. The normal stress dependent standard deviation of test data (σ_m) can be used to calculate characteristic values from site-specific testing programmes using Equation 7.4. For the geomembrane vs. geotextile type interfaces σ_m (kPa) is given by the relationship:

$\sigma_m = 0.054.\sigma_n + 1.9$ Equation 7.5

and for the geotextile vs. sand interface

$\sigma_m = 0.106.\sigma_n + 5.8$ Equation 7.6

where σ_n is the normal stress in kPa.

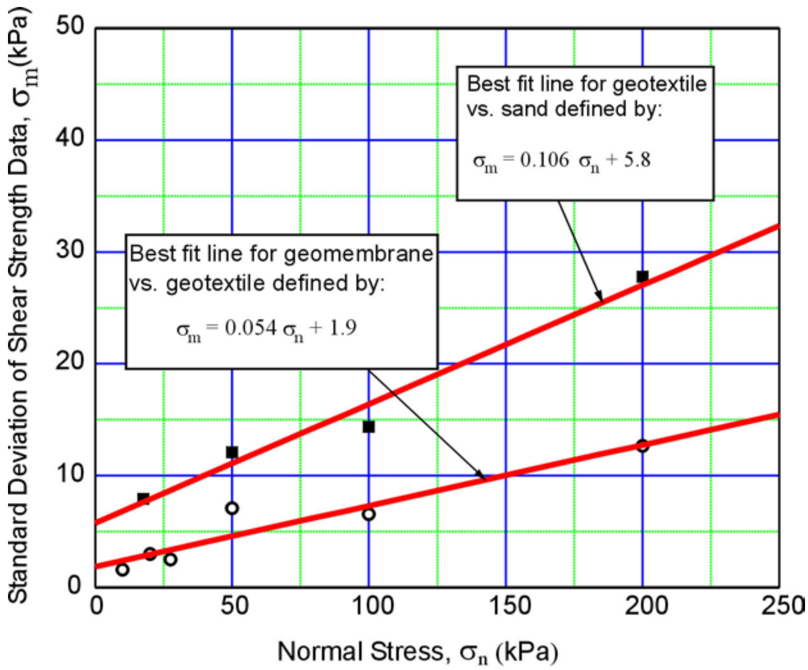


Figure 7.21 Results from the standard deviation of measured peak shear strengths for each normal stress obtained from the repeatability test programmes

A limitation of this approach is that there is an inherent assumption that the measured strengths at each normal stress represents approximate mean values. As shown in Figure 7.17, this may not be the case and they could be significantly higher or lower than the mean. Therefore, the engineer must use his/her experience and personal judgement, backed by published data (e.g. Section 7.3 after Jones & Dixon, 1998b), to decide whether the measured strengths approximate to mean values. If they are considered to be high or low, then further tests should be conducted.

Examples

Figure 7.22 shows the results of three direct shear tests (each at a different normal stress) conducted to obtain peak shear strength parameters of the interface between a textured high-density polyethylene geomembrane and a polypropylene non-woven needle punched geotextile. The best-fit straight line through the three measured shear strengths is defined by the parameters α_m = 6.9 kPa, δ_m = 25.8°. These are considered to be typical for the type of interface tested and hence the measured strengths can be taken as mean values. As only one test has been conducted at each normal stress, it is not possible to assess the variability of the data and hence characteristic shear strength parameters cannot be calculated directly. Characteristic values can only be obtained by either a) carrying out more tests and undertaking a statistical analysis or, b) using the approach based on typical variability of test data from this type of interface. If the later approach is taken, the measured shear strength at each normal stress (X_m) is corrected (i.e. reduced) to a characteristic value by the amount of $0.5\sigma_m$, where σ_m is obtained from equation (7.5). Figure 7.22 shows the best-fit straight line through the characteristic shear strength values. It is defined by the characteristic shear strength parameters α_k = 6.0 kPa, δ_k = 24.3°. These are the parameters that are used in design, either with partial factors applied or a global safety factor calculated. The correction proposed is equivalent to applying the following partial factors to the measured shear strength parameters:

$\alpha_k = \alpha_m / 1.15$ and $\tan \delta_k = \tan \delta_m / 1.07$

These are considered to be reasonable and consistent with factors typically used by designers based on experience.

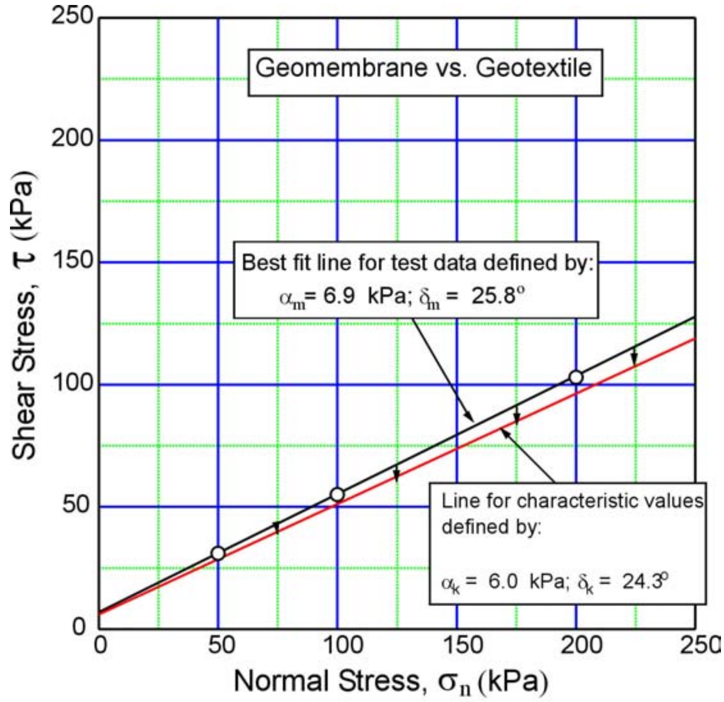


Figure 7.22 Best-fit straight lines through the test data and calculated characteristic shear strengths

7.6 Summary of Key Points

A knowledge of interface shear behaviour between lining system components is of fundamental importance in the design of landfills. While the concepts of measurement are well understood and there is significant literature on methods of measurement and typical values for combinations of materials used in lining systems, there is still much uncertainty regarding test practices. Incomplete, out of date and conflicting information is given by the existing standards. Tests should be specified and interpreted by experienced geotechnical engineers. Particular attention should be given to the design of the shear device, issues of test set up, the stress range specified and the number of tests required to obtain values for use in design. Soil mechanics principles must be considered in full when testing soil vs. geosynthetic interfaces.

A summary of tests from the literature has been provided to give a guide to the range of shear strengths that can be expected for a given interface. These values should not be used in detailed design. Performance tests using site specific materials should always be conducted.

Possible variability of test data is demonstrated. Guidance is provided on methods that can be used to obtain characteristic shear strength parameters for use in design. Conservative estimates of shear strength are required and therefore test values should not be used directly in design. A single test conducted at each of three normal stresses (i.e. current practice) is not acceptable practice for obtaining characteristic values.

8. WASTE PROPERTIES

8.1 Introduction

8.1.1 Scope

Behaviour of the waste body is a controlling factor in both the stability and integrity of engineered landfill structures. Figure 8.1 summarises modes of landfill failure in which the waste body plays a role. Knowledge of engineering properties of waste is required to assess each mode and hence to design against their occurrence. While it is not possible to fully characterise the engineering properties of waste due to its heterogeneous nature, it is important that its basic behaviour is understood and that likely ranges of the key engineering properties are known. Table 8.1 lists the properties required to perform an analysis of each of the failure modes summarised in Figure 8.1.

This Chapter concentrates on the engineering behaviour of municipal solid waste (MSW). A brief summary is provided of each of the main engineering properties. References are made to key publications, and methods of measurement and calculation are summarised. Where possible, typical ranges of values are given.

Table 8.1 Engineering properties of MSW required for design

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

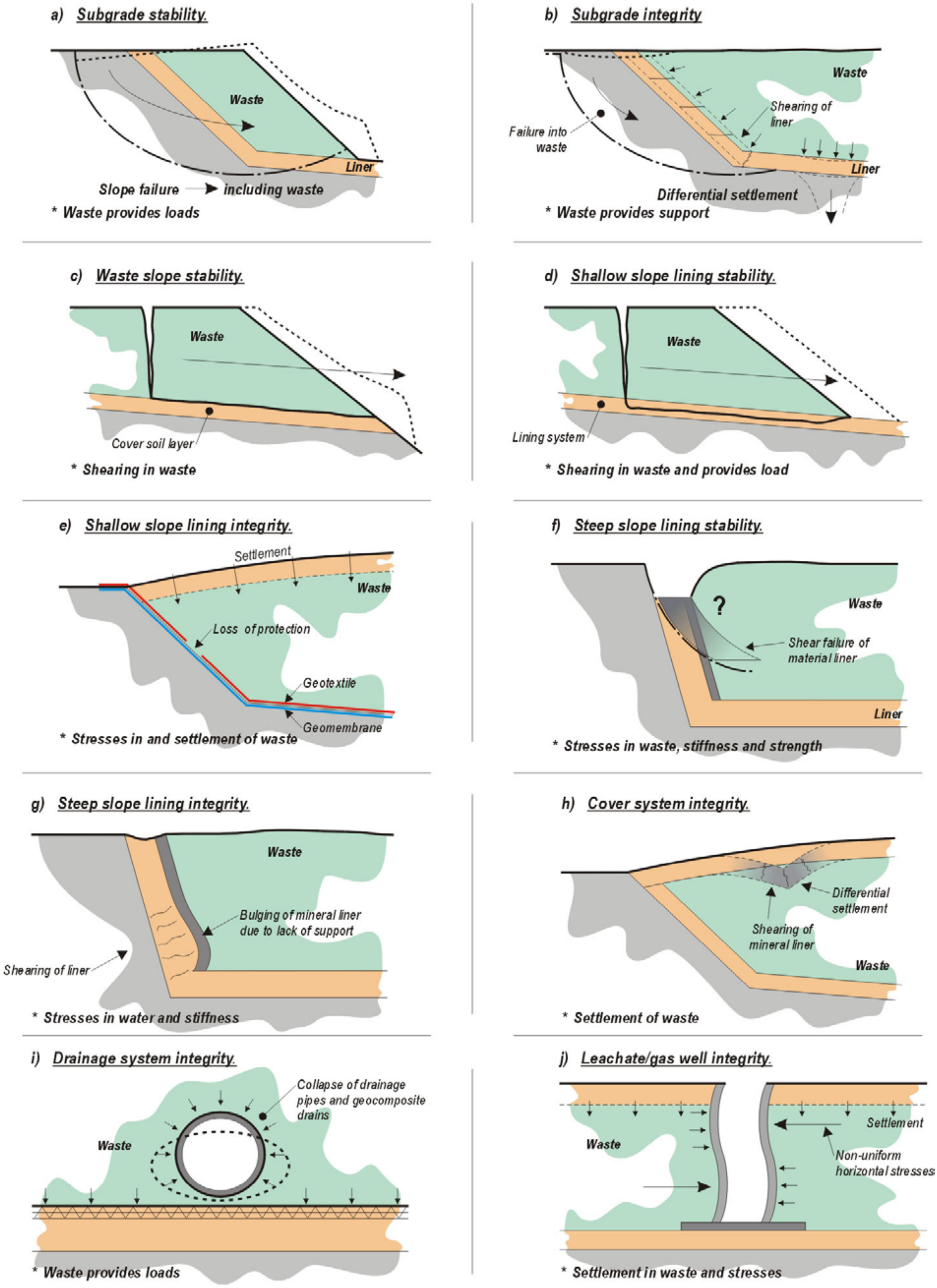


Figure 8.1 Potential landfill infrastructure failure modes: stability and integrity

8.1.2 Material description

MSW is a mixture of wastes that are primarily of residential and commercial origin. Typically, MSW consists of food and garden wastes, paper products, plastics, rubber, textiles, wood, ashes, and soils (both waste products and material used as cover material). A wide range of particle sizes is encountered ranging from soil particles to large objects such as tree stumps and demolition waste (reinforced concrete and masonry). The proportion of these materials will vary from one site to another and also within a site. Life style changes and legislation result in a changing waste stream over time. Examples are increasing plastic and decreasing ash content over the past few decades, and reduction in the amount of inert waste landfilled following introduction of the Landfill Tax. In addition, the EU Landfill Directive will see a reduction in biodegradable waste in landfills through the introduction of Biodegradable Municipal Waste (BMW) diversification targets.

8.1.3 Waste mechanics

The current understanding of waste behaviour is far from being complete. For engineering the disposal of waste, researchers and practitioners have relied on their knowledge of the behaviour of soils. Although this has been helpful to some extent, there is an increasing realisation among the landfill community that behaviour of waste should be considered in the context of a separate discipline of waste mechanics. As a starting point, it is appropriate to compare some of the results from preliminary and novel studies on waste properties available in literature with those of geological materials e.g. soil, sand, peat etc. Similarities and differences in measured behaviour can then lead to the development of laboratory and field tests specifically for obtaining engineering properties of MSW. The topic of waste mechanics is growing rapidly. An increasing number of international researchers are investigating the engineering behaviour of waste and its interaction with engineered containment systems. This work is set to expand as changes in waste composition occur over the next decade in response to the EU Landfill Directive (1999). This will have a significant impact on the engineering behaviour of the waste body. Past experience may provide little insight into future behaviour.

Evaluating the engineering properties and hence behaviour of MSW is challenging due to the variety of materials present. It is preferable to undertake testing on real materials in an undisturbed state. However, this is not always possible. Undisturbed samples cannot be taken and therefore laboratory tests have to be on disturbed material that is re-compacted into the test apparatus. MSW can be highly structured material and this structure will be destroyed. In addition, variation in composition between samples can be extreme, making it difficult to quantify the contribution to behaviour of the different components of waste or mechanisms of behaviour. Also, it is difficult to systematically change the proportion of waste constituents in order to investigate the role each plays. This is required in order to evaluate the impact of future changes in waste composition.

Additional considerations are the very large size of test apparatus required to accommodate large particles, and health and safety requirements that dictate tests on real waste have to be carried out in a controlled laboratory environment. These are both expensive to construct and operate. An example of such a facility is the 2 m diameter and 3 m high Pitsea test cell that is being used to research compressibility and permeability of MSW (Beaven & Powrie, 1995). An additional major factor is that engineering properties of waste vary with time due to the degradation process. At present there are no standard testing procedures for waste materials.

8.1.4 Waste classification

There are a number of waste classification systems in common use, and these have been developed to provide information for specific end uses, e.g. re-cycling/waste minimisation, assessment of bio-degradation potential and calorific value. However, for assessment of engineering behaviour a classification is required that groups waste constituents in terms of their mechanical properties (e.g. compressible, incompressible and reinforcing particles). In a typical landfill there will be three distinct phases present – solid, liquid and gas. There may also be a need to distinguish between mobile liquid in large drainable pores and liquid in small pores (inter-particle), and liquid that is trapped, absorbed or otherwise bound to the solid fraction (intra-particle). Grisolia *et al.* (1995a), Kölsch (1995) and Thomas *et al.* (1999) have proposed classifications based on mechanical properties.

The system proposed by Grisolia *et al.* (1995a) has three categories: inert stable, highly deformable and readily biodegradable. This system has limitations because degradation potential does not necessarily influence mechanical properties of the waste at a given instance in time. Properties may change in time as a result of degradation but this will not occur over the time span of a test or during construction (i.e. wood and paper will be compressible in the long-term as they degrade, but in the short-term would reinforce the waste). Thomas *et al.* (1999) propose using two categories: soil like and non-soil like. The disadvantage of this system is that reinforcing type materials are not specifically differentiated. Kölsch (1995) has produced the most comprehensive classification system. This is based on 7 material groups, particle size and particle shape (i.e. grain, sheet, box and fibre). The main disadvantage is the large number of variables. Development of waste mechanics as a subject will be problematic until a universally accepted engineering classification system is in place. This will enable test data to be related to a specific waste type and hence will allow the use of data from tests on similar (in an engineering sense) waste.

8.1.5 Literature on MSW engineering properties

There is a growing body of literature on the measurement of engineering properties of MSW. The majority is published in specialist conference proceedings, although papers are starting to appear in established refereed geotechnical journals. Unfortunately, due to the lack of both an agreed classification system and agreed test standards it is difficult to interpret published results. Often the nature of the waste tested is not described in any detail and the test boundary conditions are rarely given. This makes it difficult to amalgamate the results into a common framework or to apply findings to other sites.

The majority of the available data is for shear strength and settlement, which has generally been obtained from laboratory tests on disturbed (i.e. processed and re-compacted) samples, or tests subjected to sample size limitations. Although laboratory tests can provide useful information related to the general mechanisms of waste behaviour, they cannot represent field conditions. This chapter concentrates on the key parameters of unit weight, settlement, shear strength, lateral stiffness and in situ horizontal stress. Where possible, the variation of these parameters with time is also considered. This brief review is base on the following publications, each of which was originally written to summarise the state-of-the-art, Fassett *et al.* (1994), Van Imp & Bouazza (1996) and Ng'ambi (2000).

8.2 Unit Weight of MSW

As shown in Table 8.1, knowledge of unit weight is required for all aspects of design. The specific roles that waste plays in assessment of stability are highlighted in Chapters 10, 11 and 12. It is surprising that so few detailed studies have been conducted. Unit weight values vary significantly both between sites and within a single site. MSW has highly variable components, types and amounts of cover soil differ between sites, the percentage of inert and industrial wastes varies and placement procedures play an important role, as do environmental conditions (e.g. rainfall). Common difficulties in assessing MSW unit weight have been summarised by Fassett *et al.* (1994) as:

- separation of the contribution of daily soil cover;
- assessing the changes in unit weight with time and depth; the majority of reported values reflect waste near or at the surface; and
- obtaining data on the moisture content of the waste.

Fassett *et al.* (1994) considered that the following factors should be recorded along with measured unit weights:

- MSW composition including daily cover and moisture content;
- method and degree of compaction;
- the depth at which the unit weight was measured; and
- the age of the waste.

The form of the unit weight measurement should also be recorded and noted by those using the data. Values can be given as dry unit weight (for no moisture present, sample could have been artificially dried), bulk unit weight (some moisture present but waste not saturated) and saturated unit weight (all voids filled with liquid i.e. below leachate level). In most studies, it is the bulk unit weight that is measured and reported (i.e. the value includes both solids and liquid). Some studies report dry unit weights and measured moisture contents but this is rare.

8.2.1 Unit weight estimation methods

Unit weight can be estimated and measured using several techniques. Methods based on direct field measurements are considered to be the most reliable. Field tests include: large-scale replacement density measurements from the surface of the waste (e.g. Gotteland *et al.* 2000); replacement density measurements carried out in boreholes (e.g. Kavazanjian *et al.* 1993); in situ unit weight logging with gamma rays; and calculation from direct measurement of vertical stresses within the waste body (e.g. Gourc et al. 2001, Ng'ambi *et al.* 2001). Traditionally, measurements of landfill volume and the weighing of incoming waste and cover material have been used to calculate mean unit weights. This method is fundamentally flawed, as it does not take into consideration the depth dependency of unit weight or the influence of changes in moisture content resulting from precipitation.

Laboratory samples have been used in some studies but the results from these are often of limited use. Field placement conditions cannot be reproduced in the laboratory and in most cases pre-treated and sorted samples are used. Measurements using large test cells (e.g. Powrie and Beaven 1999) are the most reliable of the tests on disturbed samples. Measuring the unit weight of individual components of the waste and making an estimate of the overall unit weight by using percentages of each component has also been used but is unlikely to provide useful results. Most of the information in the literature relates to recently placed waste. There is little data on the unit weight of older, degraded, waste materials.

8.2.2 Factors affecting unit weight of waste

As with soils, the unit weight is affected by the compaction effort and layer thickness, the depth of burial (i.e. overburden stress) and the amount of liquid present (moisture content). Unlike soils, the unit weight also varies significantly because of the large variations in the waste constituents, state of decomposition, and degree of control during placement (such as thickness of daily cover or its absence). It is generally believed that initially the unit weight of waste is very much dependant on waste composition, the daily cover and the degree of compaction during placement. But as the waste becomes older the unit weight becomes more dependent on the depth of burial, the degree of decomposition and climatic conditions. Although unit weight can vary significantly over short distances, this is not necessarily a major concern in design. Unit weight is used to calculate vertical stress. Average values of stress acting on a plane (e.g. a basal liner system) are used in design calculations and hence average values of unit weight are acceptable in most design scenarios.

8.2.3 Waste components

Waste components have a controlling influence on the average unit weight of the waste mass. Individual waste components have a wide range of particle unit weights and these can change with time. Components may have voids within them in addition to those between components (intra-particle and inter-particle voids). This results in a significant percentage of waste particles behaving differently to soil particles due to their high compressibility. This is demonstrated by considering the states a metal container may experience through the landfilling process. A container will have a high void ratio and a unit weight varying from 2 kN/m^3 to 12 kN/m^3 depending on whether it is filled with liquid or is empty. If the same container is crushed flat, its unit weight can be as high as 80 kN/m^3, which is the unit weight of sheet steel. The mechanical properties of the container in the different states (i.e. empty, liquid filled and flattened) are significantly different.

Degradation of components with organic content will result in a loss of mass, changes in size and alteration of the mechanical properties (i.e. compressibility and shear strength). It will also change the unit weight of the component. As a waste body degrades void ratio reduces and hence a volume reduction occurs. Volume reduction due to degradation is responsible for a large proportion of long-term settlements. Although there are few field measurements in degraded waste it is believed that degradation results in an increase in waste density, and hence unit weight.

8.2.4 Compaction

Since MSW is a particulate material and a large proportion of the components have a high void ratio and a high compressibility, compaction processes will reduce the voids within an individual component (intra-particle voids) as well as voids between various components (inter-particle voids). The unit weight of compacted waste will depend upon the waste components, thickness of layer, weight and type of compaction plant and the number of times the plant passes over the waste. A layer thickness of 0.5 to 1.0 m will facilitate the achievement of good compaction and hence high unit weights. Present practice in the UK varies significantly both between sites and within individual sites. The later is due to varying rates of waste inputs and different operators. It is not untypical for waste to be placed in layers of 2 to 3 m thick. This results in poor to moderate compaction. Fassett *et al.* (1994) conducted a detailed survey of bulk unit weight data from the international literature (including from UK sites). A statistical analysis of the data is shown in Table 8.2.

Table 8.2 Statistical summaries of bulk unit weight data (Fassett *et al.* 1994)

Parameter	Poor Compaction	Moderate Compaction	Good Compaction
Range (kN/m^3)	3.0 to 9.0	5.0 to 7.8	8.8 to 10.5
Average (kN/m^3)	5.3	7.0	9.6
Standard Deviation (kN/m^3)	2.5	0.5	0.8
Coefficient of Variation (%)	48	8	8

The degree of compaction was derived from an assessment of individual site practices. Poor relates to little or no compaction, moderate to 'old' practices and good to 'current' (1994) practices. The assessment was in most cases subjective but provides a useful guide. An important result is the large variation in unit weight when little or no compaction is used. Landva & Clarke (1990) and Oweis & Khera (1986) report similar ranges of bulk unit weights.

Watts and Charles (1990) report values measured at a UK site. MSW compacted in 2 m lifts using a steel wheeled 21 tonne compactor achieved bulk unit weights of 6 kN/m^3, and for waste placed in 0.6 m thick layers achieved 8 kN/m^3. Van Impe and Bouzza (1996) report bulk unit weight values ranging from 5 to 10 kN/m^3 for Belgian landfills. Ng'ambi *et al.* (2001) and Gourc *et al.* (2001) have measured bulk unit weights in the order of 7kN/m^3 in the upper layers of fresh (non-degraded) waste using in situ techniques. These results indicate that current practice is still only achieving 'moderate' levels of compaction.

Kavazanjian (2001) summarises values measured at a number of US landfills. These show typical bulk unit weights between 14 and 20 kN/m^3. These are significantly higher than much of the data in the literature. However, this can partly be explained by the age of the waste (i.e. advanced stages of degradation) and the high percentage of soil like material present. Kavazanjian also reports initial waste unit weights upon placement of 6 to 7 kN/m^3.

8.2.5 Depth

Unit weight of waste varies with effective stress, which is a function of depth, and hence unit weight should vary with landfill depth. Figure 8.2 produced by Powrie and Beaven (1999) shows the variation in dry density and wet density at field capacity with vertical effective stress.

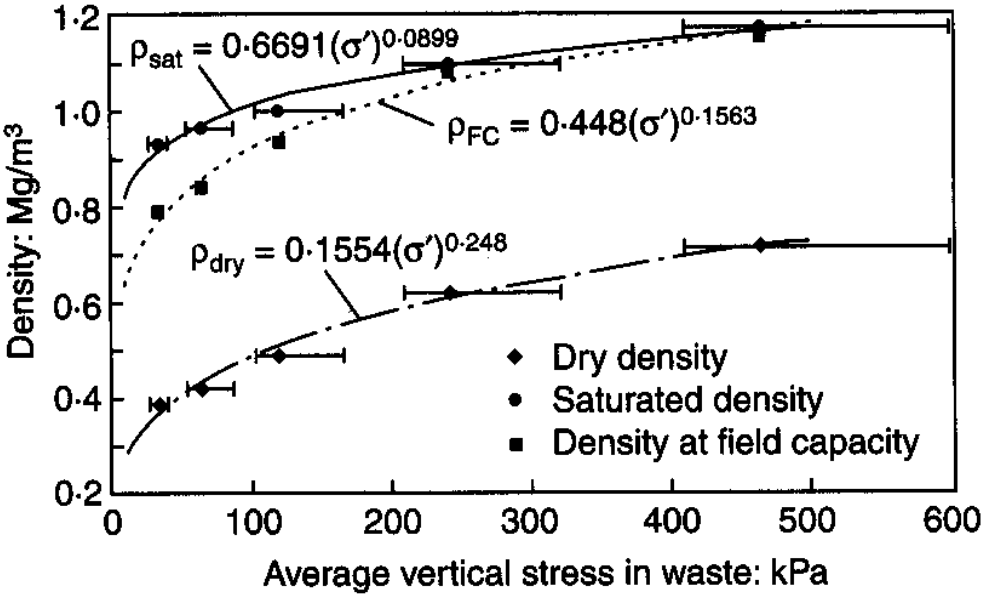


Figure 8.2 Relationship between density and average vertical stress (Powrie & Beaven 1999)

The data was obtained by compressing samples of waste in a large diameter cylindrical test chamber. The work was part of a study to investigate the effective stress/density/permeability relationship of waste. One of the implications of this work, in terms of the waste density achieved, is that compaction at the tipping face can have a similar effect to the burial of the waste by several metres of overburden (Powrie *et al.* 1998). Due to the difficulties and costs involved there are few field measurements of unit weight variation with depth. Gourc *et al.* (2001) present initial data obtained during filling of the Torcy landfill in France. The results shown in Figure 8.3 as bulk unit weight against overburden stress demonstrate a clear trend of increasing unit weight with stress level. Dixon *et al.* (2001) have obtained similar data for a site in the UK.

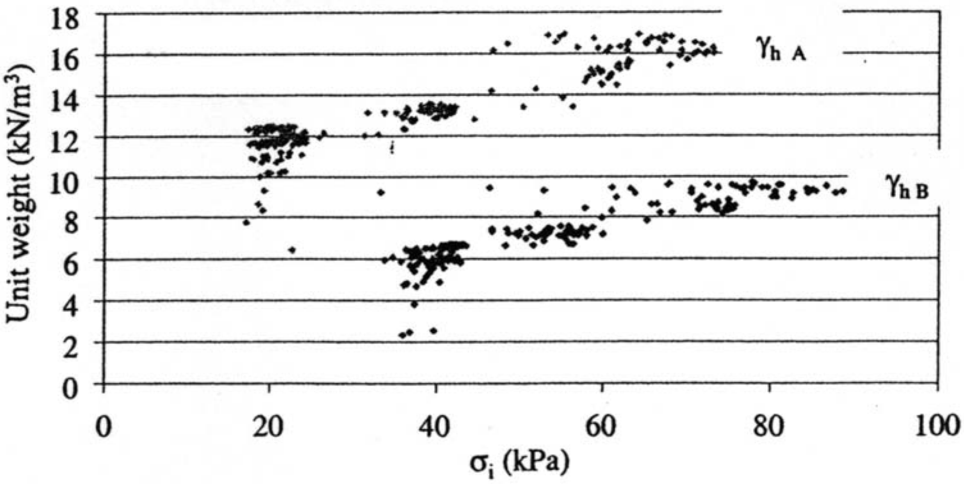


Figure 8.3 Relationship between bulk unit weight and vertical stress (Gourc *et al.* 2001)

8.2.6 Moisture content

Moisture content of waste depends on a wide range of factors including the initial waste composition, local climatic conditions, operating conditions, rate of decomposition and organic content. On exposure to water, the unit weight of any constituent absorbing water would increase (e.g. that of food waste, garden refuse, paper, textiles, wood, ash etc) due to increased moisture content of the intra-particle voids. These increases in individual particle unit weight are added to the increase in bulk unit weight resulting from increased leachate in the void spaces between particles of waste (inter-particle voids) to produce increases in the bulk unit weight of the waste mass. Therefore, older waste would be expected to have a higher bulk unit weight than fresh waste. Although there is limited field evidence to support this proposed mechanism, the data from investigations such as those described by Kavazanjian (2001) provides some corroboration.

Daily cover soils play an important role in controlling the amount and distribution of precipitation that enters waste. They result in highly structured waste bodies (i.e. horizontal layers of waste bounded by often low permeability layers of cover soil) and this can cause large spatial variations in the moisture content of waste. The phasing of final cap construction also influences the evolution of moisture content changes. Addition of liquid wastes and re-circulation of leachate will both have a fundamental influence on the magnitude and distribution of moisture contents, and hence on the magnitude and distribution of bulk unit weight.

8.3 Settlement

The compressibility of MSW has been studied for many decades. The earlier work focused on the behaviour and suitability of landfills for construction sites, however researchers now study settlement to improve the efficiency of waste placement, predict final settlement profiles for the cap and to enable assessment of interaction between side slope barrier systems and the settling waste body. Differential settlement of the capping system and settlements affecting side slope barrier performance are important in the context of this report. The aim of this Chapter is to provide a brief summary of methods used to calculate settlements and, where possible, ranges of typical values. Readers interested in more detailed information on settlements are recommended to review the key technical papers by Fassett *et al.* (1994), Van Impe & Bouazza (1996), Oweis & Khera (1998) and Gourc *et al.* (1998).

8.3.1 Calculation of vertical stress

It is usually assumed that traditional principles of soil mechanics theories of settlement can be applied to solid waste. The unit weight, γ, of a deposit increases with depth as discussed in Section 8.2. The overburden pressure, σ, at a given depth, z, is:

$$\sigma = \int_0^z y.dz \qquad \text{Equation 8.1}$$

To take into account stress dependent unit weight, the overburden pressure can be calculated using:

$$\sigma = \sum_1^n y_n z_n \qquad \text{Equation 8.2}$$

where unit weight is assumed to be constant within a given layer and *n* is the number of layers.

8.3.2 Settlement components

Mechanisms resulting in compression of waste have been summarised by Van Impe & Bouazza (1996) as:

- Physical compression and creep due to mechanical distortion, bending, crushing and reorientation;
- ravelling settlement due to migration of small particles into voids between large particles;
- collapse of containers and bridging components due to physical/chemical changes such as corrosion oxidation; and
- decomposition settlement due to biodegradation of organic components.

Factors affecting the magnitude of settlement (including due to self weight) are complex and interrelated. They include:

- Initial composition of waste (grading, particle shape, particle material properties);
- initial density and void ratio;
- layer thickness;
- type, thickness and number of layers of cover soil;
- stress history (pre-and post filling mechanical treatment);
- leachate levels and fluctuations;
- environmental factors (e.g. moisture content, temperature, gas generation); and
- settlement of sub-grade under applied waste loading.

It can be assumed that the total settlement, δ_t, (excluding any contribution from the sub-grade) is made up from two main components; primary settlement (δ_p) and secondary settlement (δ_s)

$$\delta_t = \delta_p + \delta_s \qquad \text{Equation 8.3}$$

Primary settlement includes physical compression (distortion, bending, crushing and particle orientation) and consolidation. Consolidation (i.e. time dependent dissipation of excess liquid pressures) is only relevant for saturated waste bodies. In most wastes, physical compression will occur immediately on application of load (i.e. in response to placement of overlying layers of waste). Therefore, primary compression will occur in a period of a few days to a few weeks and hence can be considered to be short-term. Incrementally linear compression models can be used to calculate primary settlements (see Section 8.3.3).

Secondary compression includes all creep effects (i.e. mechanical compression under constant stress) and those relating to degradation (both chemical and biological). Creep effects include time dependent particle distortion, bending, crushing, particle reorientation and ravelling. Degradation includes collapse of containers due to a change in strength (e.g. corrosion) and degradation of organic compounds. Biodegradation is the main component of secondary compression in MSW landfills. Many methods have been proposed to characterise and predict secondary compression. The degradation process is influenced by a range of interrelated factors, all of which vary spatially within a landfill and with time (e.g. moisture content, temperature and stress level). Present methods of prediction are simplistic and many rely on curve fitting techniques. A brief introduction to calculation methods is provided in Section 8.3.4. Secondary compression occurs throughout the active life of the landfill and is usually the main component of the total settlement.

8.3.3 Primary settlement

The principal source of loading is self-weight causing landfill settlement to occur during construction. Waste placement can be considered to be a one-dimensional compression problem (e.g. waste is placed over a large area in relation to the thickness of the deposit). An increment of vertical effective stress $\Delta\sigma'_v$, produces an increase in vertical strain $\Delta\varepsilon_v$. Stresses are assumed to be effective for fresh waste due to its typical low moisture content and hence strains are assumed to occur immediately on application of stress. A constrained modulus D, can be defined as:

$$D = \frac{\Delta\sigma'_v}{\Delta\varepsilon_v} \text{ (units kN/m}^2\text{ or MN/m}^2\text{)} \qquad \text{Equation 8.4}$$

The settlement during construction can be computed using:

$$\delta_p = \sum_{1}^{n} \frac{H_n \Delta\sigma'}{D_n} \qquad \text{Equation 8.5}$$

where:

$\Delta\sigma'_v$ is the change in vertical effective stress, H_n is the thickness of the sub-layer of waste, D_n is the constrained modulus of the layer.

The compression index (C_c) can also be used to relate increments of strain to increments of stress change (see Fassett *et al.* 1994).

Primary settlement will occur during waste placement. As the thickness of the waste increases the stiffness of the waste will also increase with depth. Constrained modulus is therefore not a constant but depends upon the level of mean stress in the layer under consideration. The compression of each layer is calculated separately using the relevant D value and the total primary settlement is calculated as the sum of the individual layers (Equation 8.3). If the waste layer is saturated, the final primary settlement will still be calculated using D but the settlement will take place over an extended period, controlled by the permeability of the waste layer and length of the drainage path (i.e. standard consolidation theory). Note that $D = 1/m_v$ where m_v is the coefficient of compressibility in m^2/kN.

Figure 8.4 shows a summary of constrained moduli values for MSW related to stress level. The most reliable data is provided by a small number of field studies. The data shown in Figure 8.4 can be used to estimate primary compression. Fassett *et al.* (1994) provide a useful summary of the literature up to 1994, detailing constrained moduli and compression index values obtained from a large range of studies, both field and laboratory based.

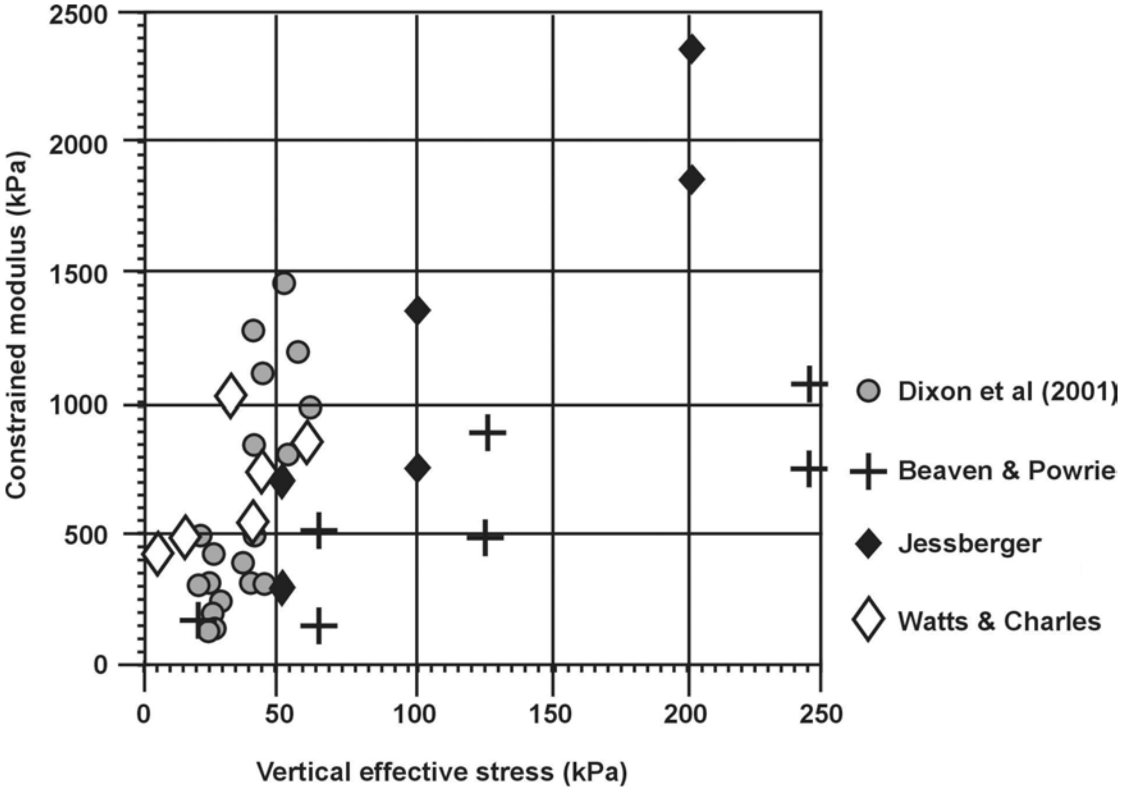


Figure 8.4 Constrained modulus vs. stress level (after McDougal & Pyrah 2001 and Dixon *et al.* 2001)

8.3.4 Secondary settlement

As discussed above the long-term settlement is mainly due to biodegradation and mechanical creep compression. It is common practice to model secondary compression using the following linear relationship on a settlement vs. log-time graph:

$$\delta_s = C_\alpha H \log \frac{t}{t_p} \qquad \text{Equation 8.6}$$

where:

t is the time at which settlement due to secondary settlement is required ($t > t_p$); t_p is the time for completion of primary settlement and C_α is the secondary compression ratio given by:

$$C_\alpha = \frac{\Delta\varepsilon}{\log t_2 - \log t_1} \qquad \text{Equation 8.7}$$

There is some field data obtained from long-term settlement monitoring studies to support this approximation. Oweis & Khera (1998) published values of C_α for a range of waste materials obtained from the literature. Table 8.3 shows selected values from their summary and demonstrates the problem of trying to use one C_α value for the entire period of secondary compression. As the rate of degradation is unlikely to be constant with time, it is not surprising that C_α is not a constant. Gourc *et al.* (1998) provide a comprehensive review of available calculation methods. Fassett *et al.* (1994) and Van Impe & Bouazza (1996) both give useful summaries of secondary compression data. Settlement prediction techniques based on modelling the biodegradation process are under development and appear promising (McDougall & Pyrah, 2001), but they are not practical tools at the present time.

Table 8.3 Secondary compression parameters for MSW material (after Oweis *et al.* 1998)

Material	C_α
Ten year old landfill	0.02
Fifteen year old landfill	0.24
Fifteen to twenty year old landfill	0.02
Old landfill	0.04
Old landfill with high soil content	0.001 to 0.005

8.3.5 Total settlement

Actual computations of settlement can be complex. For example, to estimate the total settlement for a recently closed landfill, the following considerations will be necessary for each of the layers in the landfill:

- settlement of waste from self-weight (primary settlement);
- settlement from the weight of each subsequent layer (including final cover) that overlies the given layer (primary settlement);
- settlement due to secondary compression, taking into account that C_α is likely to decrease with age (secondary settlement);
- settlement of the mineral basal liner (if present) due to primary and secondary settlement; and
- settlement of compressible subgrade.

Due to the heterogeneous nature of landfill constituents and their varied rates of decomposition, differential settlements occur. The problem is further complicated by the fact that adjacent cells are completed at different times and filling often takes place on top of older waste deposits. Differential settlements are difficult to predict but are important as they can jeopardise the stability and integrity of the final cap.

8.3.6 Discussion and summary

Principles developed for compressible materials that are extensively used in soil mechanics can be applied to MSW landfills to estimate both primary and secondary settlement. Although the assumptions on which the theories are based are not always fully satisfied, reasonable estimates of settlements are possible with predictions of primary settlement being the more reliable. MSW is usually partly saturated and as a result primary compression due to increased stress occurs in the short-term. Creep and degradation (secondary compression) are the dominant factors controlling time-dependant settlements over the medium to long-term.

As reliable methods of predicting long-term settlements from biological and chemical actions are not presently available, it is recommended that monitoring by geotechnical instrumentation over both short-term and long-term periods is the most reliable method of obtaining settlement data for use in subsequent designs. Information is required on variation of settlement with depth in addition to surface settlement data. Typically for MSW landfills, primary settlements are difficult to quantify, however secondary settlements in the range 15% to 25% of the initial waste thickness have been quoted (DoE 1995). It is thought that total settlements will be in the range 20% to 30%, and these values can inform the initial design process. These values are believed to relate to settlements that occur following completion of waste placement. Larger values may be appropriate if settlement during waste filling are included.

8.4 Shear Strength

At present, there is a dearth of information on MSW shear behaviour. There have been no detailed studies conducted in the UK and only limited studies in other countries. Shear strength of MSW is presently defined using the Coulomb failure criterion. This is commonly used in soil mechanics and in studies of other particulate materials. The approach is based on defining a shear strength failure envelope relating shear stress of a plane within the material mass to the normal stress on that plane (see section 7.1). It is common to approximate the failure envelope to a straight line over applied ranges of normal stress. Any combination of shear stress and normal stress that plots below the envelope indicates stability, and points on the envelope denote failure. The shear strength parameters that define the failure envelope are the slope of the line (ϕ) and intercept on the y axis (c).

The slope of this failure line indicates increasing shear strength with normal stress and describes the frictional strength of the material. The intercept, c, can denote real cohesion between particles, but is often a function of the curvature of the failure envelope and/or variation between samples and measurement errors. Therefore, it is common to define it at as the ‘apparent cohesion’ or ‘cohesion intercept’. Care should be exercised when applying experience of shearing in soils to the study of MSW. Waste contains particles that are compressible, can sustain large tensile strains (e.g. plastic), change with time (e.g. through degradation) and a significant proportion of which reinforce the waste mass. An outcome of using the Coulomb criterion is that it gives an increase in shear strength with increasing stress level and hence with depth of burial. This is consistent with waste being considered as a frictional material.

8.4.1 Measurement of MSW shear strength parameters

Field techniques

For the reasons outlined in Section 8.1.3 it is both difficult and costly to obtain representative and hence reliable strength parameters for MSW (i.e. large particle size, heterogeneity, control of structure etc.). It is preferable to obtain values from field studies and these can be divided into three approaches:

- back-analysis of landfill slope failures;
- in situ measurements; and
- back-analysis of controlled slope failure experiments.

Back-analysis of landfill slope failures can provide information on the shear strength of a large mass of waste. A number of key failures from the literature are highlighted in Chapter 11. However, without detailed knowledge of the pore water pressure conditions, the shape of the shear surface, the unit weight of the waste, the shear strength (both magnitude and stress/strain behaviour) of all other materials the shear plane passes through (i.e. lining system interfaces) and a number of other contributing factors, it is not possible to obtain reliable waste shear strength parameters from the back-analysis process. It is rare for this level of information to be available.

In situ techniques for measuring shear strength are presently inadequate and unreliable. The results from studies using standard penetration tests (SPT) and cone penetration tests (CPT) have been mixed. There are no established relationships between waste penetration resistance and shear strength. Two different investigations using CPT, one by Hinkle (1990) and another by Siegel *et al.* (1990), gave values of tip resistance that were two orders of magnitude different. This shows that, unlike soils, there is no correlation for cone penetration resistance for waste that can be used to determine the nature of the material penetrated. In waste the cone can meet particles with ranges of sizes, compressibility and deformability and therefore it would have been surprising if there was a correlation. The application of conventional shear vane testing is also not recommended for MSW because homogeneous materials with small particle sizes are required for useful results to be obtained (Jessberger & Kockel, 1993). However, vanes may provide useful information in degraded and hence potentially more homogeneous wastes. An in situ technique for measuring the shear strength of MSW at a range of depths and for material with varying degrees of degradation is urgently required.

Controlled cut slope failure experiments have been attempted by a number of researchers to enable back-analyse of shear strength parameters following failure (e.g. Singh & Murphy 1990, Cowland *et al.* 1993, Blower *et al.* 1996). In many cases these have proved unsuccessful as the slopes have deformed significantly but shear failure has not occurred. This is due to the reinforced nature of the waste and the small destabilising forces applied (i.e. low self-weight in relatively small cut slopes and the difficulty of applying large surcharge loads). The high stability of steep cut slopes in MSW is also supported by site experience.

Laboratory techniques

Results from laboratory tests should be viewed with scepticism. The waste will have been disturbed, and hence the structure will have been lost, large particles will have been removed or processed (e.g. some of the results in the literature are for shredded waste) and the in situ density and stress conditions may not have been reproduced. Many of the studies in the literature have used triaxial compression tests, often with the sample unconfined (i.e. load applied vertically and no lateral support provided). Studies include those by Jessburger (1994) and Grisolia *et al.* (1995b). The tests do not usually produce shear failure despite subjecting the samples to large vertical strains (20 to 40 %). This is due to the increases in sample density that occur during the test. As the sample is compressed it becomes denser and hence stronger. Therefore, even though significant shear stresses are applied to the material, the increases in shear strength mean that the sample does not fail.

Kavazanjian (2001) provides a more detailed explanation of the mechanism and concludes that triaxial compression testing is not an appropriate technique for measuring the shear strength of MSW. Inability to cause failure in these tests has led to shear strength test results being related to levels of strain (i.e. different shear strength parameters are given for each strain level). While this approach has some merit if used in design to try and control strains in the waste body, it can lead to confusion and great care should be taken in applying such values.

Table 8.4 Examples of measured shear strength parameters from the literature (Jones *et al.* 1997)

Reference	Shear Strength Parameters		Method	Comments
	c′ (kPa)	ϕ′ (°)		
Jessberger (1994)	7	38	Not stated	Reporting Gay *et al.* (1978) (MSW)
Jessberger (1994)	10	15	Back analysis	Reporting Spillman (1980)
Jessberger (1994)	10	17	Back analysis	Reporting Spillman (1980)
Jessberger (1994)	0	30	Estimate	Reporting Cassina (1979). From field observations
Jessberger (1994)	0	40	Estimate	Reporting Cassina (1979). From field observations
Jessberger (1994)	7	42	Simple shear	Reporting Gay *et al.* (1981). 9 month old MSW
Jessberger (1994)	28	26.5	Simple shear	Reporting Gay *et al.* (1981). Fresh MSW
Fassett et al. (1994)	10	32	Suggested values	Reporting Jessberger & Kockel (1991)
Fassett et al. (1994)	10	23	Suggested values	Suggested by authors
Kolsch (1995)	15	15	Suggested values	Suggested by author
Kolsch (1995)	18	22	Suggested values	Suggested by author
Cowland et al. (1993)	10	25	Back analysis	Deep trench cut in waste. Suggested values by authors
Del Greco & Oggeri (1993)	15.7	21	Direct shear	Tests on baled waste. Lower density bales
Del Greco & Oggeri (1993)	23.5	22	Direct shear	Tests on baled waste. Higher density bales
Landva & Clark (1986)	19	42	Direct shear	Old refuse
Landva & Clark (1986)	16	38	Direct shear	Old refuse
Landva & Clark (1986)	16	33	Direct shear	Old refuse + 1 year
Landva & Clark (1986)	23	24	Direct shear	Fresh, shredded refuse
Landva & Clark (1986)	10	33.6	Direct shear	Wood waste / refuse mixture
Golder Associates (1993)	0	41	Direct shear	Project specific testing

The most appropriate laboratory technique is the direct shear box, although the general concerns regarding the applicability of laboratory tests discussed above still apply. A large device is required if representative samples are to be tested. For example, Kolsch (1995) used a shear box 3m x 1m x 1.5m and a number of other studies have used devices in the order of 1m x 1m x 1m (e.g. Kavazanjian 1999, Gotteland *et al.*2001). Large shear displacements are required to reach failure and volume changes should be recorded in order to enable the measured shear strength to be related to the sample density. Unfortunately, this information is seldom provided in the literature, thus making interpretation of results difficult.

8.4.2 Measured shear strength values

This section summarises measured shear strengths obtained from the literature. The majority of the studies included here obtained strengths from direct shear box tests and back-analyses of failures. Those requiring more detailed information should read Van Impe & Bouazza (1996), Jones *et al.* (1997), Eid *et al.* (2000) and Kavazanjian (2001) all of which provide summaries of shear strength parameters from the literature. Table 8.4 gives waste shear strength parameters from the literature (Jones *et al.* 1997) although it is by no means a comprehensive summary. It is included to demonstrate the wide variation in values that can be obtained. Given the large range of possible wastes and the difficulties involved in measuring shear strength, the large scatter is not surprising.

Van Impe and Bouazza (1996) suggested that the failure envelope shown in Figure 8.5 could be used as a starting point in design if no site specific is available. Design values of c and ϕ are defined according to three distinct zones:

Zone A: corresponding to very low stress (0 kPa $\leq \sigma_V < 20$ kPa) where the MSW behaviour can be described as being only cohesive. In this case, c = 20 kPa.

Zone B: corresponding to low to moderate stresses (20 kPa $\leq \sigma_V < 60$ kPa). In this case, c = 0 kPa and $\phi \approx 38°$.

Zone C: corresponding to higher stresses ($\sigma_V \geq 60$ kPa). In this case, c ≥ 20 kPa and $\phi \approx 30°$.

In a similar approach, and based on data from North American studies, Kavazanjian (2001) suggested c = 24 kPa and $\phi = 0$ for normal stress below 30 kPa and c = 0 and $\phi = 33°$ for higher normal stresses. This envelope is shown on Figure 8.5. It is believed that some of the Kavazanjian (2001) data was considered by Van Impe & Bouazza (1996) and therefore contributed to the development of Figure 8.5. Based on the data in Table 8.4, Jones *et al.* (1997) suggested a design line defined by c = 5 kPa and $\phi = 25°$. The Jones et al. (1997) design line and the envelope of the data it is based on are also shown in Figure 8.5. It can be seen that the three 'suggested' design conditions differ significantly. It can be concluded that caution should be exercised when using the literature to obtain values for use in assessment of specific site and waste conditions. It would be considered nonsensical to suggest that a single failure envelope could be used for all soil types and suggesting the same for waste is equally ridiculous.

Kolsch (1995) has investigated the tensile strength of MSW using a modified version of the large shear box. This research was aimed at assessing the contribution to shear strength from the reinforcement provided by waste fibres (e.g. plastic). The results obtained help explain the stability of steep cut faces in waste and the stability of deep tension cracks that have been observed to form in waste masses under certain circumstances, however, it is unlikely that this tensile strength can be safely used for design purposes.

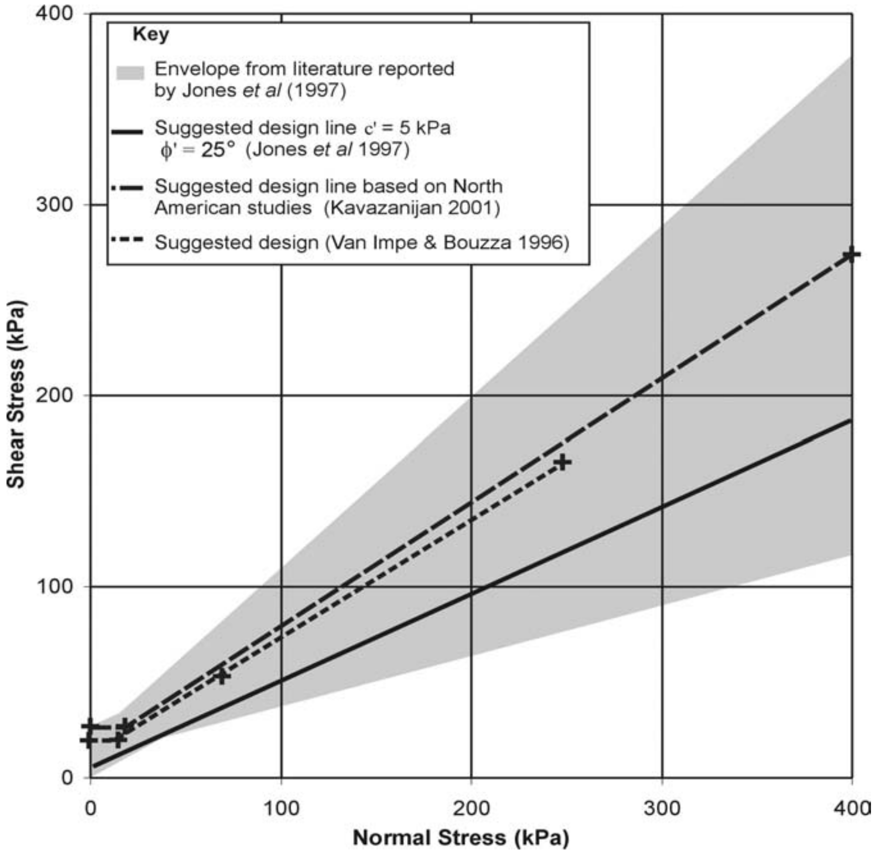


Figure 8.5 Suggested MSW shear strength envelopes for design (after Van Impe & Bouazza 1996, Jones *et al.* 1997 and Kavazanjian 2001)

8.4.3 Discussion and summary

Knowledge of shear strength is required in order to assess waste slope stability. Cases of landfill slope failure tend to be controlled by shear surfaces forming along interfaces within the liner system or within weak underlying soils (see Case Study 5 (section 4.3.6) and Chapter 11). However, failures do occur entirely within the waste mass, and those that are controlled by weak zones and interfaces still often have a section of the shear surface forming in the waste. Therefore, while it is important to evaluate weak interfaces and/or poor foundation materials it is also necessary to estimate the strength properties of waste when conducting stability analyses. Strain compatibility between waste and lining materials/interfaces should also be considered (see Chapter 11).

In situ measurements of waste shear strength is at present not possible. Back-analysis of failures provides the most reliable way of obtaining data, although this method is not without difficulties due to problems obtaining adequate detailed field information. Laboratory methods have been used widely but are not recommended due to their reliance on using disturbed samples. Of the methods available, the direct shear box produces the more reliable information. Waste slope design and assessment is presently based on experience (i.e. x° angle slopes have been stable for y years therefore this angle can be used for new slopes). Summaries of results from international research are also presently used in design as outlined above. This data is a compilation of results from a wide range of waste types, of different age and obtained using different test methods. Using it for the design of UK landfills is questionable.

An approach based on past experience is flawed for two reasons: i) MSW slope failures do occur. In the UK there are no published case studies, but Environment Agency records provide evidence that failures are not uncommon (see Chapter 4). There have been a significant number of waste slope failures in other countries. ii) The constituents, and hence mechanical properties, of new MSW are constantly changing. This is caused by changes in life style and legislation. In addition, mechanical properties of a MSW mass change with time due to the degradation process. A slope could become unstable tens of years after its formation.

The design of safe waste slopes in both the short and long-term is critical to the management of sites and hence optimisation of the landfill construction processes. Temporary slopes are increasingly being excavated in old waste to enable lining systems to be tied into areas that were previously unlined.

8.5 Lateral Stiffness

Information on the lateral stiffness of MSW is required to assess the performance of steep side slope lining systems that rely in part on the waste for their stability and integrity, and landfill spreading that can affect leachate collection wells. To date, Dixon and his co-workers have published the only information on lateral (i.e. horizontal) waste stiffness (Dixon & Jones 1998, Dixon *et al.* 2000). This section provides a brief summary of the results obtained by carrying out pressuremeter tests at different depths in MSW of varying age. This ongoing research is part of a project to investigate the interaction between steep slope lining systems and adjacent waste.

8.5.1 Stiffness parameters of MSW based on elastic theory

Elastic parameters such as shear modulus (G), Young's Modulus (E) and Poisson's ratio (ν) can be used to quantify the response of a material to a change in stress (i.e. calculate strains). The parameters are related, as is the constrained modulus (D) introduced in Section 8.3.3, and an example of their interdependence is given by Equation 8.8:

$$G = \frac{E}{2(1+\nu)} \qquad \text{Equation 8.8}$$

In situ measurement of these parameters is required. Waste placement methods, waste type and depth of burial will have a fundamental influence on the measured values. Tests on disturbed samples will not provide representative results. The measured values are dependent on, and therefore can be related to, other physical properties such as density, depth of burial, stress level etc.

8.5.2 Lateral stiffness parameters obtained from pressuremeter tests

Pressuremeter testing is a standard technique used in soil and rock mechanics to measure stiffness parameters (e.g. in situ lateral shear stiffness) and other ground properties such as in situ horizontal stress and, in certain materials, shear strength parameters. Dixon & Jones (1998) described a novel method of obtaining in situ stresses and shear stiffness values using the pressuremeter test in MSW. This initial research used a Self Boring Pressuremeter, and testing techniques have evolved with later studies using a High Pressure Dilatometer type pressuremeter (Dixon *et al.* 2000). The test takes the form of inflating a membrane to expand a preformed cylindrical test pocket. The pressure required to expand the pocket (and hence deform the surrounding material) is related to the magnitude of the radial expansion. To date, tests have been carried out in both young (1 to 5 years) and old (>15 years) MSW at depths of 1.7 to 17.0 m below ground level. In excess of 30 individual tests have been conducted.

8.5.3 Shear stiffness moduli obtained from pressuremeter tests

Stiffness values are obtained by calculating the slope bisecting small cycles of unloading and reloading. Figure 8.6 shows a typical pressuremeter test result for MSW (Dixon *et al.* 2000). The use of unload/reload loops is standard practice for obtaining consistent and repeatable values for shear modulus. Figure 8.7 (after Dixon *et al.* 2001) shows the measured relationship between shear modulus and average horizontal stress for wastes of different ages. The average horizontal stress can be related to depth below surface of waste (see Section 8.6). General trends of increasing stiffness with stress level are observed, and the older waste (partly degraded) is shown to be stiffer than fresh waste (little degradation). This trend of increasing stiffness with mean stress, and hence depth, is as expected for a drained particulate material. Although there are no other studies in the literature to corroborate these results, the systematic and consistent behaviour observed provides confidence in the validity of the measured trends.

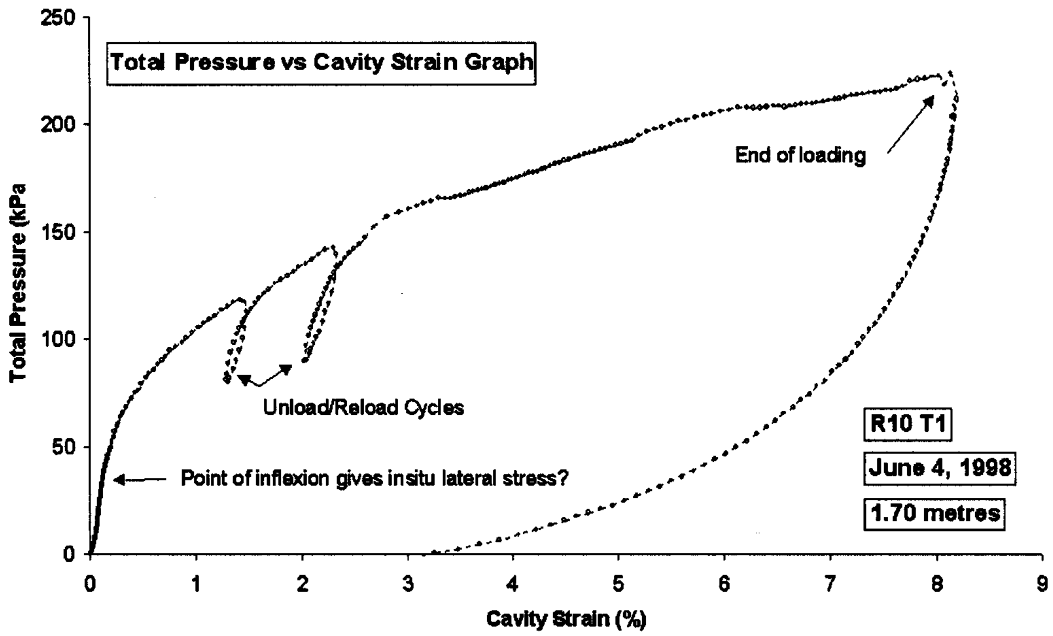


Figure 8.6 Example result of a pressuremeter test in MSW (after Dixon *et al.* 2000)

The data and trend shown in Figure 8.7 for the fresh waste can be used in an assessment of waste barrier interaction as part of the design of steep side slope lining systems (see Chapter 12). The information on elastic properties included in this report is limited and simplistic in nature. There are a number of issues related to the measurement technique and calculation of elastic parameters that requires detailed consideration if they are to be used to assess waste/barrier interaction (e.g. elastic parameters are strain dependent and hence strongly non-linear). However, even a low level assessment of the values in Figure 8.7 shows that the shear stiffness of MSW is significantly less than soil. This has important implications for waste/barrier interaction, and specifically for the distribution and magnitude of strains in steep slope barrier systems (see Chapter 12).

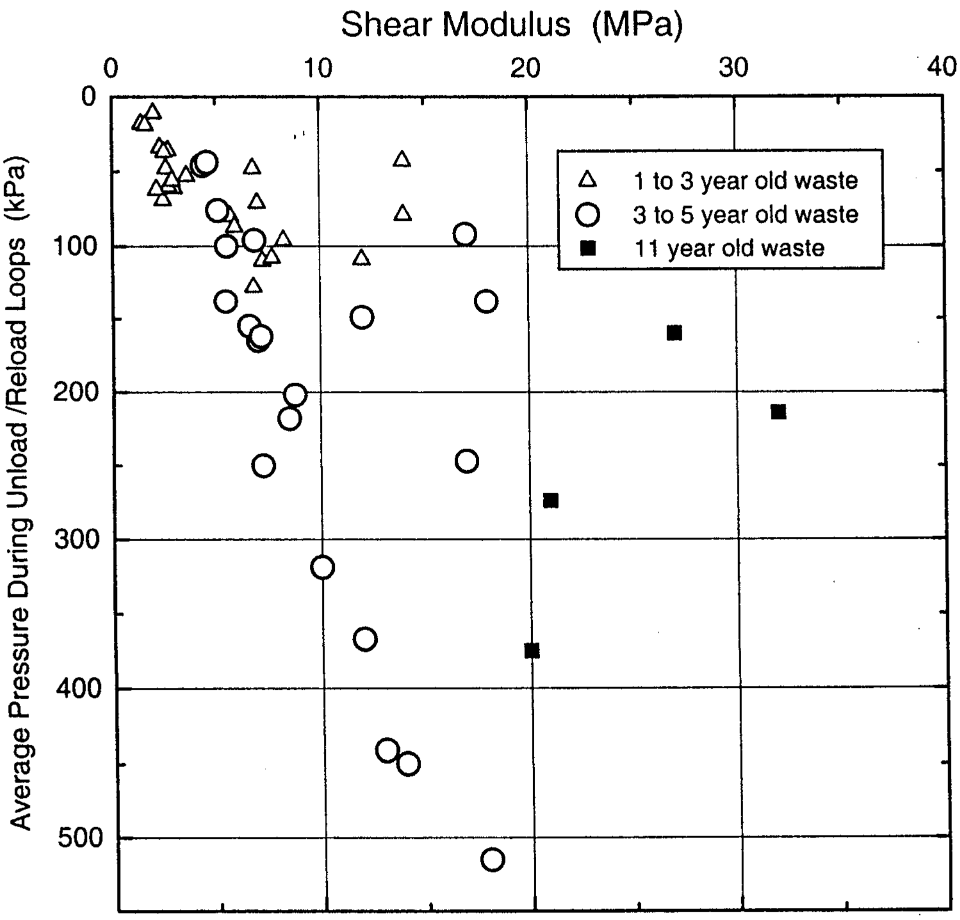


Figure 8.7 Shear modulus vs. average stress for fresh and partially degraded MSW (Dixon *et al.* 2001)

8.6 Horizontal In Situ Stress

Knowledge of horizontal in situ stress is required in order to aid assessment of both shallow and steep side slope barrier systems (see Chapters 11 and 12 respectively) and the performance of structures buried in the waste body such as leachate and gas wells. Measurement of horizontal stress in a particulate material such as waste is difficult because the act of introducing a measuring instrument will alter the stress being measured. For a body at rest, horizontal stresses (σ_h) can be related to vertical stresses (σ_v) by the coefficient of earth pressure at rest (K_0) where:

$$K_0 = \frac{\sigma_h}{\sigma_v} \qquad \text{Equation 8.9}$$

8.6.1 Laboratory study

Laboratory measurement of horizontal stress in a waste body can only provide an indication of possible field behaviour. Clearly the laboratory sample cannot replicate the field conditions, especially particle size and method of placement, and hence the structure of the sample cannot be modelled. These factors play important roles in the generation of horizontal stresses, and hence results from laboratory studies must be of questionable use. Landva *et al.*

(2000) have produced the only results from a laboratory study of MSW. They conclude that K_0 values in the order of 0.35 to 0.4 would be typical for fresh MSW and that K_0 would be expected to increase towards a value of 0.5 if less reinforcing material was present. If the degradation process destroys reinforcing material, these results indicate that the K_0 value, and hence horizontal stresses, will increase with time. This has not been substantiated by field measurements.

8.6.2 Field measurements

An estimation of K_0 values has been made by Dixon & Jones (1998) using results from pressuremeter tests. The preliminary results from the study are shown in Figure 8.8 (Dixon & Jones 1998). It can be seen that there is no clear relationship between K_0 and depth. This is due to disturbance caused by insertion of the pressuremeter (i.e. changing the values of horizontal stress being measured) and the heterogeneous nature of the waste tested. However, the results do appear to be suggesting that higher values than those obtained by Landva *et al.* (2000) might be applicable for in situ material.

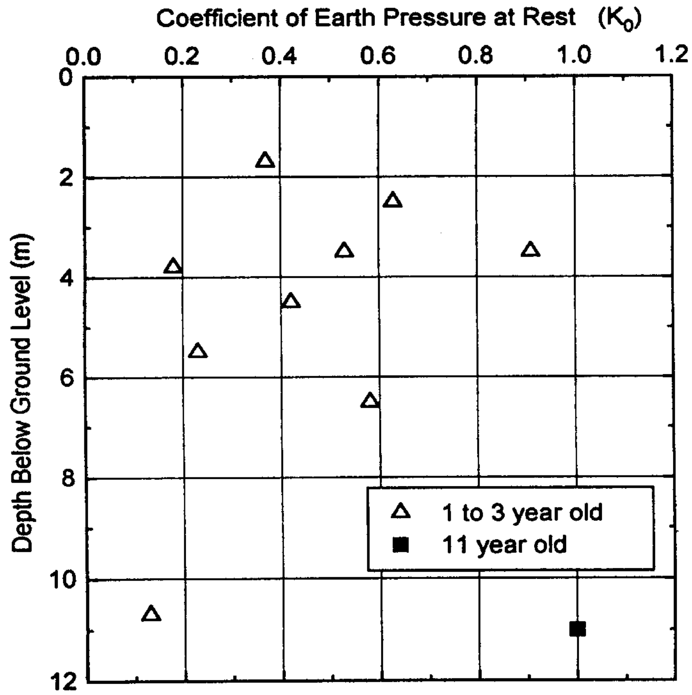


Figure 8.8 Coefficient of earth pressure at rest vs. depth (Dixon & Jones 1998)

Ng'ambi *et al.* (2001) report direct measurements of horizontal stresses in MSW. Pairs of pressure cells were buried in waste at a range of depths to measure vertical and horizontal stresses as part of a study of steep slope lining system performance. The preliminary measurements have been used to calculate K_0 values and these are shown in Figure 8.9. As with the pressuremeter test results, values higher than those proposed by Landva *et al.* (2000) are indicated.

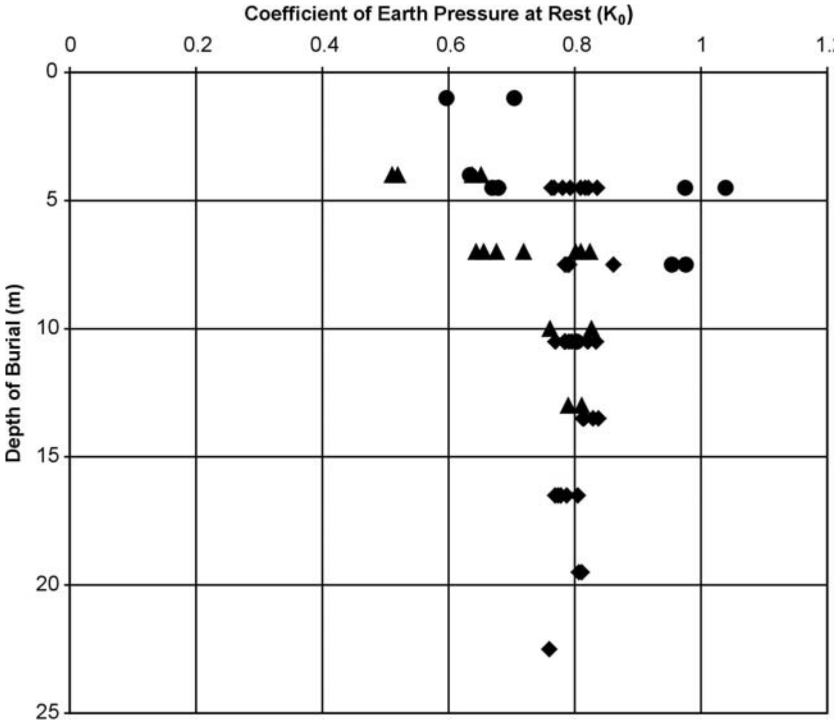


Figure 8.9 Coefficient of earth pressure at rest measured using pressure cells vs. depth of burial (Dixon *et al.* 2001)

8.6.3 Summary

Knowledge of in situ horizontal stresses is required to assess barrier performance post waste placement (see Chapters 11 and 12). Obtaining representative values is very difficult and this accounts for the small amount of information in the literature. The one laboratory study indicates that the coefficient of earth pressure at rest is around 0.4 for MSW (i.e. the horizontal stress at any depth is 40% of the vertical stress at that depth). The measurements made in situ using both a pressuremeter and pressure cells indicate that higher values may be more applicable. However, given the difficulties involved in carrying out both laboratory studies and field measurements, and the small number of studies conducted to date, it is not possible to draw any firm conclusions on a typical range of K_0 values for MSW. Although, the studies indicate that using a value of K_0 = 0.4 would probably be a reasonable (i.e. probably conservative approach in design.

8.7 Summary of Key Issues

The current understanding of waste behaviour is far from being complete. Evaluating the engineering properties and hence behaviour of MSW is very difficult due to the variety of materials present. Knowledge of unit weight of MSW is required for all aspects of design. It is generally believed that initially the unit weight of waste is very much dependant on waste composition, the daily cover and the degree of compaction during placement. As the waste becomes older, the unit weight becomes more dependent on the depth of burial, the degree of decomposition and climatic conditions. Values of unit weight typically range from 3 kN/m^3 to 10 kN/m^3, however values up to 20 kN/m^3 have been reported for older more soil like waste in the US.

Mechanisms resulting in settlement of waste include physical compression and creep due to mechanical distortion, bending, crushing and reorientation, ravelling due to migration of small particles into voids between large particles, collapse of containers and bridging components due to physical/chemical changes such as corrosion oxidation and decomposition due to biodegradation of organic components. For simplicity, the total settlement of a MSW landfill can be taken as the combination of primary settlement and secondary settlement. Primary settlement includes the physical compression (distortion, bending, crushing and particle orientation) and consolidation. Secondary compression includes all creep effects (i.e. mechanical compression under constant stress) and those relating to degradation (both chemical and biological).

Knowledge of shear strength is required in order to assess waste slope stability. In situ measurement of waste shear strength is at present not possible. Back-analysis of failures provides the most reliable way of obtaining data, although this method is not without difficulties due to problems obtaining adequate detailed field information. Laboratory methods have been used widely but are not recommended due to their reliance on using disturbed samples. Of the methods available, the direct shear box produces the more reliable information. Although various envelopes have been suggested for design, a conservative approach should be taken due to the heterogeneity of the waste.

Information on the lateral stiffness of MSW is required to assess the performance of steep side slope lining systems that rely in part on the waste for their stability and integrity. To date, there is limited information; the most comprehensive study has carried out pressuremeter testing in both young (1 to 5 years) and old (>15 years) MSW at depths of 1.7 to 17.0 m below ground level. In excess of 30 individual tests have been conducted. General trends of increasing stiffness with stress level are observed, and the older waste (partly degraded) is shown to be stiffer than fresh waste (little degradation).

Knowledge of in situ horizontal stresses is required to assess barrier performance post waste placement. Obtaining representative values is very difficult and this accounts for the small amount of information in the literature. The one laboratory study indicates that the coefficient of earth pressure at rest is around 0.4 for MSW, however measurements made in situ using both a pressuremeter and pressure cells indicate that higher values may be more applicable.

Measuring and interpreting MSW engineering properties are extremely difficult tasks. However, a knowledge of unit weight, vertical compressibility, shear strength, lateral stiffness and in situ stresses is fundamental to the assessment of landfill stability and integrity.

9. SUB-GRADE STABILITY

9.1 Introduction

Structural performance of sub-grade will control the stability and long-term integrity of a landfill lining system. Therefore, it is of utmost importance that the stability and compressibility of the sub-grade be assessed as part of the design process. Assessment of stability should be carried out both when an existing slope is to be used (i.e. natural slope or existing cut slope) and when a re-modelled slope is required as part of the landfill design. In addition, stability should be assessed in the short-term (i.e. before lining construction and waste placement), in the medium-term (i.e. following construction of the lining system) and in the long-term (i.e. post waste placement). Unstable sub-grade will result in either instability of the lining system or differential straining of the lining system leading to loss of integrity. Compressible sub-grade can also lead to differential straining of the lining system leading to loss of integrity.

The aim of this chapter is to provide an introduction to the key issues involved in sub-grade stability and compressibility. It is not intended to be a sole reference for assessment of slope stability. There are many standard reference books and journal papers that cover the topic of slope stability assessment techniques and analysis methods in great detail. If the reader requires more information on rock slope stability issues they should consult reference texts such as *A Geology for Engineers* by Blyth and de Freitas (1988), and *Foundations of Engineering Geology* by Waltham (1994). For information on soil slopes the reader is directed to consult *BS6031:1981 Code of Practice for Earthworks* and *The Stability of Slopes* by Bromhead (1992). All works to assess sub-grade stability including site inspections, field measurements and testing, laboratory testing and stability analyses must be carried out by a competent geotechnical engineer. An example of a definition of competence is given in the Quarry Regulations (1999), which states that '*The geotechnical specialist must have sufficient expertise and practical experience of similar conditions to adequately assess the safety of the excavation or tip and the precautions required to make and keep it safe*'. This chapter is divided into sections on cut slopes, natural slopes, fill slopes, basal conditions and a short section on analysis methods.

9.2 Cut Slope Stability Issues

9.2.1 Introduction of general issues

Many landfills are located in abandoned mineral workings. Whether these have shallow side slopes (e.g. sand and gravel quarries and brick pits) or steep side slopes (e.g. hard rock quarries) the boundary of the void will be lined as part of landfill construction. Therefore, the stability of these side slopes must be demonstrated as part of the landfill design. It is also necessary to prove that construction of the landfill will not result in future sub-grade instability. Slopes formed through excavation are defined here as cut slopes.

All mineral workings are governed by the current *Quarry Regulations (1999)*. Regulation 33 states that the operator should ensure that '*all excavations and tips are designed, constructed, operated and maintained so as to ensure that either instability or movement which is likely to give rise to a risk to the health and safety of any person is avoided*'. The regulations also state that '*the operator shall ensure that in the event of the abandonment of or ceasing of operations at the quarry, the quarry is left so far as is reasonably practicable in a safe* *condition*' (Regulation 6(4)). These regulations should mean that any cut slopes resulting from mineral extraction activity are safe. However, it does not ensure that there will be no movement of the slopes, only that such movements should not represent a risk to health and safety. This is an important point. The factor of safety required to operate a quarry safely is likely to be lower than usually required for geotechnical structures, even in temporary conditions. A factor of safety against slope failure that is greater than, but close to, 1.0 means the slope is stable but it also implies a degree of strain in the system (i.e. required to mobile the strength of the slope materials). The selection and use of factors of safety are discussed in detail in Report No.2.

An additional consideration is that an assessment of stability is only relevant for a specific period of time. Slopes are dynamic systems that respond to climatic events and human modification. Even if a quarry slope is left in a stable condition at the completion of mineral extraction, natural weathering processes and changes in groundwater pressures can result in degradation of the sub-grade slope surface (e.g. surface instability and softening) and could lead to large scale instability (e.g. rotational failure and wedge failure). While some weathering processes cause relatively slow changes, climatic events (e.g. periods of heavy rain) can result in very rapid changes in stability.

9.2.2 Hard rock slopes

Competent geological materials are likely to form steep slopes. The stability of such slopes will be controlled by the strength of the mass and not of the intact rock. Discontinuities (e.g. joints, bedding planes and faults) are the most important factor in rock slope stability. Assessment of stability requires a detailed knowledge of the orientation and frequency of discontinuities and of their shear strength. Shear strength is controlled by planarity (e.g. whether curved or planar), surface roughness, presence of any infill (e.g. clay), and aperture (e.g. whether closed or open). The stress relief that accompanies the excavation of any slope will cause a weakening of the rock mass through the opening of discontinuities and also causing deformations along discontinuities. This may result in a reduction in shear strength (i.e. mobilisation of post-peak shear strengths). Open fractures form flow paths for groundwater and this can further reduce shear strength and hence stability.

In addition, methods used to form the slope can have an important influence on the state of discontinuities. For example, blasting can result in an increased number fractures and also in the fractures becoming open. This will decrease stability and will have a particular influence on the stability of surface material. Other important factors are the material unit weight, slope angle and orientation (i.e. in relation to orientation of discontinuities) and pore water pressures. A number of types of failure of varying severity can occur. They can range from instability of individual pieces of rock on the slope surface, through rock falls, toppling failures and wedge failures to rotational and block slides. Figure 9.1 shows a range of common failure modes (Waltham 1994). Note that the single rotational and mud flow mechanisms are more relevant for soil slopes.

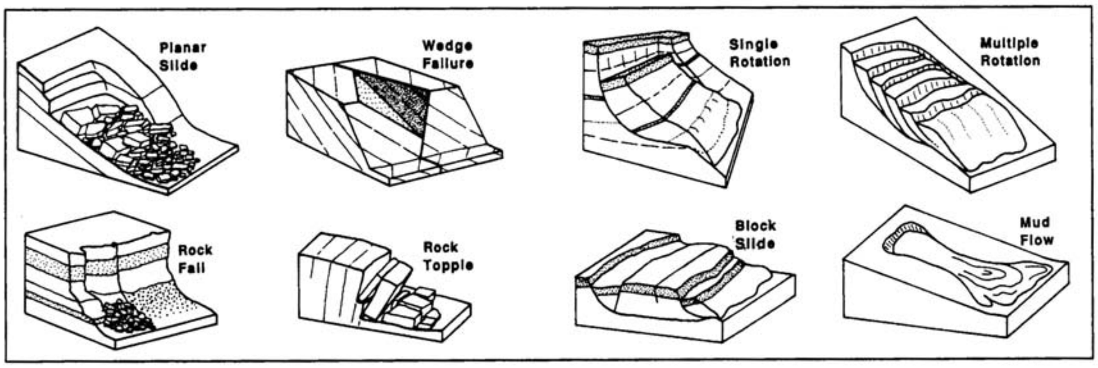


Figure 9.1 Common modes of slope failure (Waltham 1994)

Another important control on slope stability is layering of materials. It is possible for the majority of a slope to be formed of a competent rock but for stability to be controlled by a thin layer of weaker material (e.g. a band of clay, mudstone or shale). Knowledge of the orientation of such a layer in relation to the slope will be critical in any assessment of stability. A detailed walk over survey and rock mass study is the minimum requirement for an investigation carried out as part of the landfill design. Many sites will also need monitoring of groundwater conditions and laboratory tests conducted to measure discontinuity and/or material strengths. As in all assessments of stability, knowledge is required on the groundwater conditions in the slope. Measurements or estimates of pore pressures acting in controlling discontinuities and weak layers should be made to provide input data for the stability assessment. Worst-case conditions should be assessed (i.e. stability should be checked for the highest possible pore pressures during the life of the slope).

The impact of recent or ongoing dewatering of the void, and/or adjacent voids, should be considered when assessing measured pore water pressures and current stability conditions. Consideration should also be given to the likely influence of lining construction on sub-grade stability. The placing of a low permeability barrier against the sub-grade slope could result in a build up of pore water pressures and this could lead to sub-grade instability (see Figure 9.2). In summary, there are many complex and interrelated factors that control the stability of hard rock slopes. A suitably qualified geotechnical engineer should conduct a comprehensive and systematic investigation, leading to a slope design that ensures sub-grade stability.

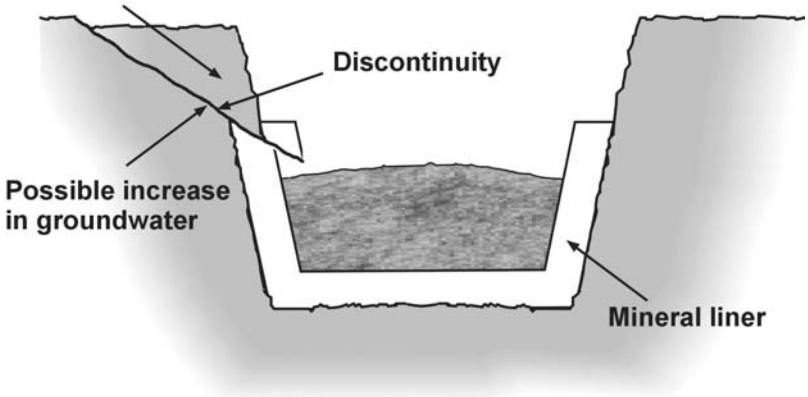


Figure 9.2 Possible influence of liner construction on sub-grade stability

9.2.3 Cohesive soils

Controlling factors

Many landfills are constructed in clay pits formed obtaining material for use in brick making. The sub-grade at these sites often provides a natural barrier and material for use in construction of compacted clay liners. Important clay strata include: London Clay, Gault Clay, Oxford Clay, Upper and Lower Lias Clays, Weald Clay and Mercia Mudstone (weathered to clay near the ground surface). Assessment of the long-term stability of slopes excavated in clay requires consideration of soil mechanics principles. It is possible for clay slopes to fail after remaining stable for many years. Design of clay sub-grade slopes must consider stability in both the short and long-terms. The key design issue is that clay slopes are often excavated at an angle that although stable in the short-term, will become unstable in the long-term. If instability occurs, it will compromise stability of the lining system and could lead to a loss of integrity.

A number of the factors that control stability of hard rock slopes are also relevant for cohesive soil slopes. Discontinuities (e.g. bedding planes) and weak layers often control the stability of clay slopes. Stress relief induced deformations can cause preferential straining along such features and this mechanism has been shown to lead to reductions in shear strength towards residual values (Burland *et al.* 1977). A detailed knowledge of the orientation and shear strength of such features is required in order to undertake a stability analysis. Of particular importance is an understanding of the pore water pressure distribution in the slope. The low permeability of cohesive soils means that the measurement of pore water pressures can take a significant time. It should also be appreciated that pore water pressures can vary significantly over short distances (i.e. laterally and with depth) and therefore it is not acceptable to use open standpipes to measure 'groundwater levels'. Measurement of the distribution of pore water pressures may require the use of a number of piezometers installed at different plan and depth locations in the slope. When interpreting measurements it needs to be appreciated that the values may be transient pressures and not long-term values in equilibrium with the hydraulic boundary conditions. Assessment of measured values requires information on the age of the slope (i.e. when it was excavated) and the consolidation properties of the material (i.e. permeability and coefficient of compressibility). Prediction of the long-term pore water pressures is needed, as these values are use in analysis of long-term sub-grade slope stability. As with the hard rock slopes, the likely impacts of current dewatering activities and construction of a low permeability liner against the slope must be considered.

Theory of undrained unloading during excavation

The low permeability of cohesive soils means that pore water flow is very slow and this leads to the following time dependent conditions. It is assumed that the soil is fully saturated. In the UK, this assumption is valid for all but the top couple of metres below ground level. Figure 9.3 shows typical changes in total stress, pore water pressure and factor of safety with time following excavation of a slope in cohesive material (after Bishop & Bjerrum 1960). Note that: Effective stress = Total stress – Pore water pressure, and that shear strength of soils is controlled by effective stresses (i.e. the stress taken by the soil particles). The processes involved in controlling short and long-term stability are described below, and demonstrated in Figure 9.3.

Short-term conditions

- Excavation of a slope results in unloading of the clay and this is shown by the decrease in total stress.
- The soil particles try to move apart in response to the decrease in total stress (i.e. like a spring increasing in length when the load on it is reduced). However, in the short-term, the low permeability of the soil means that there is insufficient time for water to flow into, and thus expand, the void spaces between the particles. The soil is said to be in an undrained state.
- Therefore, the pore water pressures decrease (i.e. negative excess pore water pressures are generated) in response to the decrease in total stress. As the particles cannot move apart, the stress taken by the soil skeleton, i.e. the effective stress, cannot change (i.e. the load taken by a spring cannot change if the spring is not allowed to change in length).
- As the effective stress has not changed then the strength of the soil must have remained unchanged. The factor of safety of the slope decreases during formation because the steeper the slope the lower the stability (i.e. destabilising moments increase).
- Following excavation, a zone of depressed (possibly even negative) pore water pressures will exist in the material beneath the slope (i.e. in the zone of material subjected to a decrease in total stress).

Long-term conditions

- Water will flow towards the zone of depressed pore water pressure from the areas adjacent to the slope with higher pore pressures unaffected by slope formation. The rate of flow of water is dependent upon the permeability of the soil.
- As water flows into the soil under the slope the void spaces will increase in size and the particles will move further apart (i.e. the soil swells). The pore water pressures will increase and the effective stresses will decrease (i.e. the load on a spring will decrease if it is allowed to increase in length). The total stresses remain essentially the same because no more material is excavated.
- As the effective stresses decrease so does the shear strength of the soil. Hence the factor of safety against slope failure also decreases.
- The soil will have minimum shear strength in the long-term condition when steady seepage pore water pressures have been established (i.e. swelling of the soil stops and the pore water pressures remain constant). The factor of safety is at a minimum in this condition. The soil is said to be in a *drained* state.
- therefore, the long-term condition is the critical case for stability. Stability analyses should be carried out using effective stress shear strength parameters (c' and ϕ') and the long-term steady seepage pore water pressure distribution.

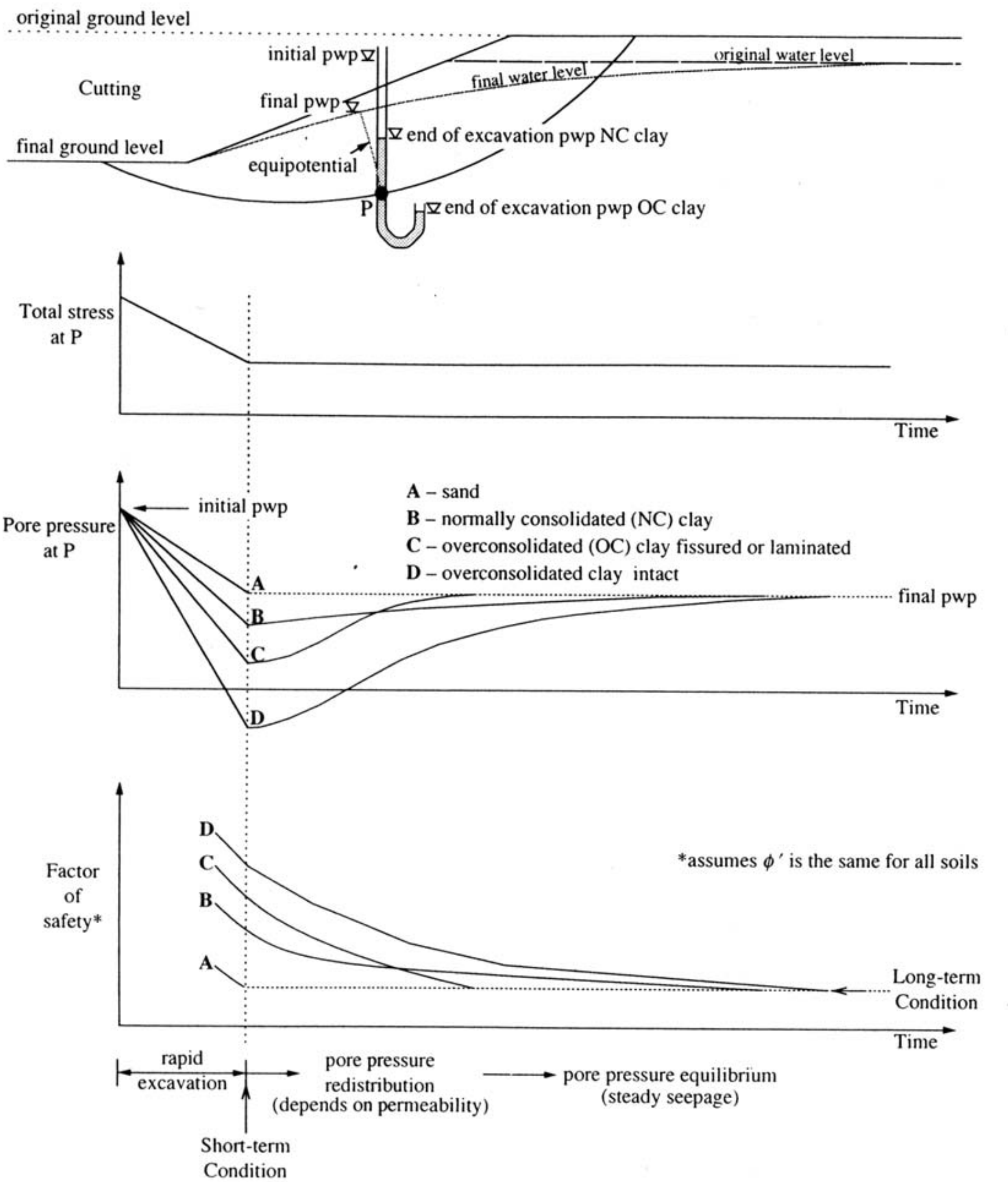


Figure 9.3 Changes in total stress, pore water pressure and factor of safety with time following excavation of a slope (after Bishop & Bjerrum 1960)

The key question is how long will it take to establish the long-term critical slope stability conditions? Many designs of temporary slopes in cohesive material rely on the slow equilibration of excess pore pressures to ensure stability in the short-term (e.g. steep slope mineral lining systems prior to waste placement). The rate of dissipation of depressed pore water pressures is dependent upon the mass permeability of the soil (i.e. including the influence of any open fissures and more permeable sand/silt layers) and drainage path lengths. Further explanation of this process can be obtained from standard soil mechanics text books (e.g. Barnes 2000, Craig 1999).

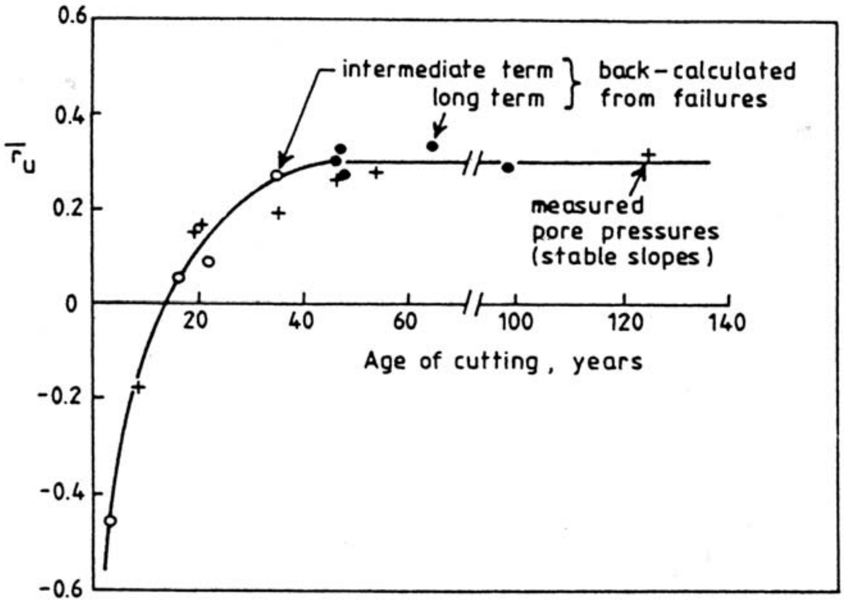


Figure 9.4 Relationship between pore pressure and time to failure of cutting slopes in Brown (weathered) London Clay (after Chandler 1984)

It should be noted that dissipation rates in slopes formed of soils such as London Clay could take many tens of years. This means that failure might not occur until tens of years after excavation of the slope. Figure 9.4 (after Chandler 1984) shows data obtained from a number of case studies of failures in London Clay cutting slopes, along with estimated pore water pressures at the time of the failure. All the data points are for approximately 10 metre high slopes formed in Brown (weathered) London Clay. The pore water pressure distribution in a slope is defined using r_u values (where r_u = pore water pressure/total stress). The time at which failure of each slope occurred is given on the graph. The Figure demonstrates that pore water pressures were still increasing up to 50 years after slope formation (i.e. the r_u values increase during this period). The increase is due to the dissipation of the depressed pore pressures formed during excavation. The conclusion is that for a 10 metre high slope in Brown London Clay it takes about 50 years to establish the long-term conditions. If constructed too steep, it is possible for the slope to fail at any time during this period.

9.2.4 Granular soils

The design of cut slopes in granular soils is relatively straight forward. In a dry state they will be stable at slope angles up to the internal angle of friction of the material (i.e. slope angle (β) $\leq$ friction angle (ϕ')). If the soil has some cementation between particles then steeper slope angles can be formed. If layers of cohesive soil are present it is possible that these will influence stability of the slope. The main factor controlling stability is usually groundwater flow. Seepage will result in reduced effective stresses and hence reduced shear strength (i.e. the higher the pore water pressures the lower the effective stresses). Seepage can also cause fines to be washed out of the slope (piping) and this can lead to instability. Particular attention should be given to concentrations of groundwater flow caused by the presence of low permeability layers (e.g. clay). These can concentrate seepage, and hence piping, and can also result in reduced shear strength in the clay layer.

9.3 Fill Slope Stability Issues

Fill slopes such as embankments and bunds are generally constructed by compacting soil in layers to achieve a required density, and hence strength and stiffness. Standard compaction guidelines should be used for all structural fill (i.e. that is required to withstand load). Comprehensive guidance on the compaction process is given in the Highways Agency Specification for Highway Works 1998. An assessment of the sub-grade must be carried out to ensure that it is capable of supporting the fill without bearing capacity failure or excessive settlements.

9.3.1 Cohesive soil

Slopes formed of cohesive fill are dependent for their stability on the density, and hence shear strength, of the as placed material. The moisture content of the fill controls the engineering properties and time dependent behaviour. The relationship between moisture content and density is discussed in Chapter 6. If the fill is compacted dry of the plastic limit, the compaction process can generate significant pore suctions (i.e. negative pore pressures). This means that in the short-term there will be high effective stresses within the fill and hence it will have a high strength and relatively high factor of safety against slope failure. The fill can be described as being in an undrained state. These large suctions mean that the fill will have an affinity for water and will readily swell. This will dissipate the suctions, reduce the effective stresses and reduce the shear strength and stability of the slope.

Shallow failures in softened surface material are common in fill slopes. Care should therefore be taken to design slopes in cohesive fill to be stable at the anticipated long-term moisture content. Control of surface drainage is a key consideration in protecting the fill material from softening. Cohesive fill excavated from a borrow pit immediately before use can result in very high suctions being present following compaction. The stress relief experienced by the fill results in high suctions (see Section 9.2.3), which are further increased during the compaction process. Stability of cohesive fill slopes in the short-term may therefore be an ephemeral condition.

9.3.2 Granular soils

Shear strength of granular fill is dependent upon the grading of the material, particle shape and density. In a dry state the fill will be stable at slope angles up to the internal angle of friction of the material (i.e. slope angle (β) $\leq$ friction angle (ϕ')). If the soil has some cementation between particles then steeper slope angles can be formed. The main factor influencing stability is groundwater flow. Seepage conditions will result in reduced effective stresses and hence reduced shear strength (i.e. the higher the pore water pressures the lower the effective stresses). Seepage can also cause fines to be washed out of the slope (piping) and this can lead to failure.

9.4 Natural Slope Stability Issues

Issues affecting stability of natural slopes are the same as many of those influencing the stability of cut slopes. Natural slopes often contain surfaces with shear strengths at or close to residual (i.e. pre-existing shear planes). These result from the slope formation processes. Even prior to any landfill construction activity such slopes can have marginal stability. Translational type failures are common due to the presence of weak weathered veneers of material on the slope. Failure occurs on a shear plane parallel to the slope, the depth of which is controlled by the influence weathering has on the shear strength profile. Investigation of the natural slope must be conducted as part of the landfill design process. It should aim to identify any weak layers and/or pre-existing shear surfaces, obtain information on shear strength and establish the groundwater regime.

9.5 Slope Stability Analysis Methods

The majority of slopes formed in rock and soil can be analysed using limit equilibrium methods. These entail defining a failure mechanism involving a specific shear surface and comparing the restoring moments to the disturbing moments in order to define a factor of safety. If the restoring moments are larger than the disturbing moments then the factor of safety is larger than 1.0 and the slope is considered to be stable. All potential shear surfaces must be analysed in order to find the critical condition (i.e. the surface that gives the lowest factor of safety). Failure modes that can be assessed using limit equilibrium methods include: translational slides, wedge failures, toppling failures and rotational slides.

It is not appropriate to cover the common methods here. Standard analysis techniques for rock slopes are described in detail in a number of standard text books (e.g. Hoek & Bray 1994). An introduction to methods of analysis for translational and rotational slides is provided in Chapter 11. A discussion of the key input parameters and some issues related to the use of stability computer programs are also included. Further information on analysis methods for rotational slides can be found in Bromhead (1992).

9.6 Sub-Grade Base Stability

9.6.1 Excessive settlements

Settlement of sub-grade can have a direct affect on the integrity of the lining system. Of particular concern are differential settlements. These can result in tensile stresses in lining components. These can lead to the formation of cracks and shear zones in mineral liners and tears and large tensile strains in geomembranes. Possible causes of sub-grade settlement must be assessed as part of the design process, and the magnitude of likely differential settlements calculated. The design of the lining system and/or the method of construction should be revised if the predicted values of differential settlement are large enough to result in loss of liner integrity. Two mechanisms can result in excessive settlement: compressible sub-grade and the presence of cavities.

Compressible sub-grade

If a highly compressible sub-grade is present, such as soft cohesive or loose granular soils (either in situ or fill materials), the magnitude of total and differential settlements, their spatial distribution and rates of settlement should be calculated. Differential settlements can be caused by a varying thickness of a sub-grade deposit and/or variations in load (i.e. height of waste). Granular materials will settle immediately on application of load and therefore rate effects are not usually a significant factor. Cohesive soils will consolidate over a period of time, with the rate of settlement depending upon the permeability of the deposit and drainage path lengths. Settlement may not be completed until many months or even years after application of load. Phasing of waste filling must be considered. As filling progresses, a settlement wave could form under the advancing front of waste. This can cause tensile stresses in the lining components and could lead to loss of integrity (see Figure 9.5). If the sub-grade has low shear strength, extrusion of the soil and local bearing failure can add to the strains in the liner. These mechanisms can be minimised by careful phasing of the filling sequence.

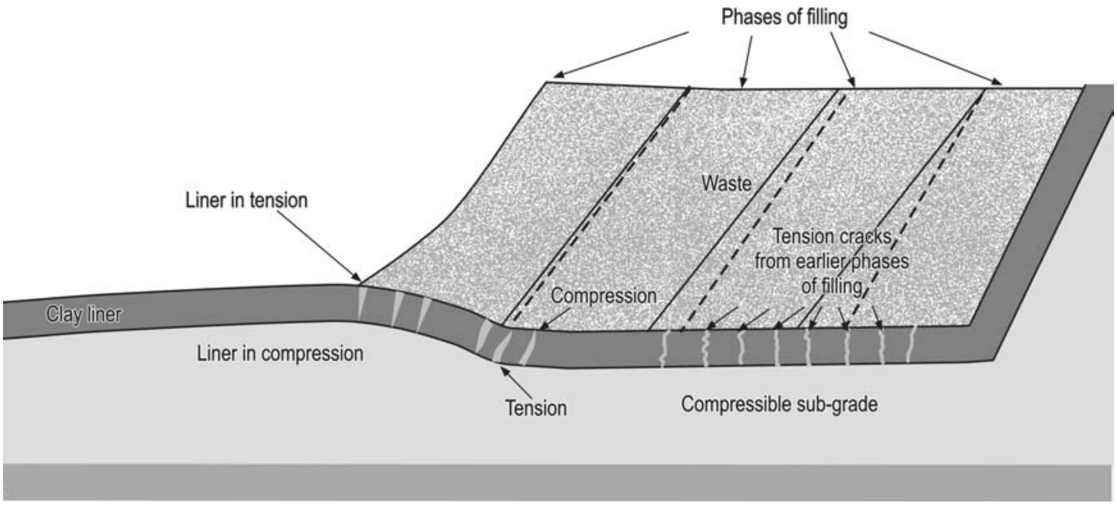


Figure 9.5 Influence of construction on stresses in liner

Cavities in the sub-grade

The possible presence of natural and artificial cavities must be assessed as part of design. The significance of a cavity is a function of its size, in relation to its depth below the lining system, and the strength and stability of the material between the cavity and the lining. Sometimes, cavities have been in-filled, but with compressible material which can also cause settlement problems. Natural cavities are usually associated with particular geological strata (e.g. chalk, limestone and gypsum). Waltham (1994) provides an introduction to the processes involved in forming natural cavities and basic stability issues.

The British Geological Survey's geological memoir for the region of the landfill site can provide background information on the geology and hence the likelihood of natural cavities being present. If a possibility exists, then a thorough desk study and ground investigation must be conducted to assess the risk posed to the lining system. The potential for groundwater flow to initiate collapse should be considered. The worst-case scenario is if the cavity migrates to the underside of the lining system. This is discussed in Chapter 10 in relation to the strains that can be mobilised in the lining if it has to bridge a void of a given size. If it is shown that the integrity of the lining will be compromised, then remedial works should be carried out to fill the cavity. The cavity can be grouted or a raft provided to support the lining (e.g. reinforced soil or a layer of geosynthetic reinforcement).

Artificial cavities will usually be the result of mining activity. The likelihood of the site being located in an area of previous or current mining can be assessed as part of the desk study by consulting the relevant Geological Memoir and sources of mining records. If there is evidence of previous mining activity in the general area, then a thorough desk study and ground investigation must be conducted at the location of the landfill. A large percentage of early activity is unrecorded, and the majority of this was at relatively shallow depths. As for natural cavities, the stability of the cavity must be assessed, including the possible influence of groundwater, and the consequence of it migrating to the underside of the lining system must be considered. The remedial techniques given for natural cavities are also appropriate for those formed by mining.

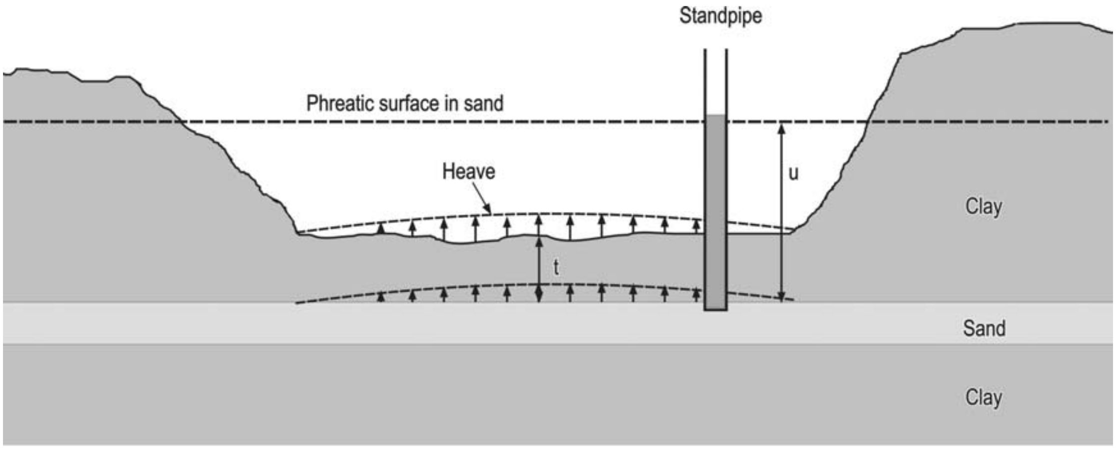
Basal heave

Basal heave will occur if the pore water pressure at a given depth in the sub-grade is greater than the total stress from the overlying strata. A factor of safety against base heave is calculated as the ratio of total stress/pore water pressure. Therefore, the knowledge required to assess the possibility of base heave is the bulk unit weights of the sub-grade strata (i.e. used to calculate total stress), the pore water pressure distribution within the sub-grade, levels of specific strata and depth of excavation. Structure of the sub-grade has a controlling influence. Calculations are carried out for specific layers of material.

A structure of low permeability layers inter-bedded with high permeability layers is particularly prone to causing instability. The low permeability layers (e.g. cohesive soils, mudstone and shale) confine groundwater in underlying permeable layers (e.g. sand, silt, gravel and sandstone) and this can lead to artesian conditions. The high permeability strata can generate the artesian pore water pressures through hydraulic connection with zones of high pore water pressure outside the excavation. The depth from excavation level to the bottom of the low permeability layer is used to calculate the total stress. The pore water pressure in the underlying high permeability stratum is assumed to act on the base of this low permeability layer. Figure 9.6 shows typical ground conditions that can lead to basal heave.

Once the factor of safety drops below 1.0, the high permeability strata can provide a relatively large volume of water to uplift the overlying strata and hence cause heave of the sub-grade. If hydraulic fracturing of the low permeability layer occurs, then large volumes of water can be transmitted to the upper surface of the sub-grade. This will cause softening of the sub-grade and compacted clay liners and local inflation and hence straining of geomembranes. A site investigation must identify the strata below proposed formation level and provide information on their structure, including thickness and spacing of inter-bedded high and low permeability layers and the presence and spacing of discontinuities such as fissures and joints. The pore water pressure depth distribution must be obtained with specific attention given to measuring pore pressures in individual stratum (i.e. an open borehole measuring general groundwater levels is of limited use). Consideration must be given to the likely worst-case pore water pressure conditions that could be present during the construction period.

Excavation will cause instantaneous stress relief, and therefore calculations of base heave should use the maximum depth of excavation as this gives the minimum total stress (i.e. the top of the lining should not be used). As the sub-grade strata can dip across the site, the worst-case combinations of excavation depth, base level of confining layer (i.e. low permeability layer) and pore water pressures must be assessed to obtain the minimum factor of safety.



Factor of safety for basal heave = $\frac{\gamma_{soil} \times t}{\gamma_{water} \times u}$

(where $\gamma_{soil} \cong 20\ kN/m^3$ and $\gamma_{water} \cong 10\ kN/m^3$)

Figure 9.6 Ground conditions that can lead to basal heave

9.6.2 Filling on waste

It is becoming common practice to construct landfill cells on top of existing waste bodies. This has the advantage of extending the use of a site (with all the financial benefits) but introduces significant technical difficulties for the designer, especially if a fully contained cell is required. The challenge is to construct a liner that will retain its integrity during settlement of the underlying waste. The majority of the issues outlined in Section 9.6.1 on excessive settlements of sub-grade are relevant for constructing new landfills on existing waste bodies.

The magnitude and distribution of total and differential settlements will depend upon the type of waste being built on, its thickness and how this varies across the site, initial placement conditions (i.e. density) and the age of the waste. Issues discussed in Chapter 8 on types and prediction of waste settlement should be considered. Primary compression will occur under the applied load of the new fill and this will take place in the short-term (i.e. within weeks and months). Secondary compression due to creep and degradation affects will take place in the longer term (i.e. over years to tens of years). Construction of liners on thick deposits (i.e. tens of metres) of recently deposited MSW material will over a period of time experience metres of settlement. The heterogeneous nature of waste will mean that differential settlements will also be large. A standard lining system will not be able to withstand such movements without suffering significant strains, and hence losing its integrity. Filling over older MSW (i.e. after completion of degradation) and wastes with a higher soil content will result in smaller more manageable settlements, although differential settlements capable of influencing liner integrity would still be experienced.

In addition, cavities can form in waste due to degradation of large components (e.g. white goods, drums etc). The design approach is the same as for cavities within the natural sub-grade (see Section 9.6.1 above). The liner has to be designed to span an assumed size of cavity if it were to migrate immediately beneath it (see Chapter 10). If a liner system has to be used above a waste body it should be constructed on a support layer to protect the liner from localised large differential settlements and cavities. Solutions could include the use of cellular mattresses and reinforced soil rafts.

9.7 Summary of Key Points

Assessment of sub-grade stability is an essential part of the landfill design process. Standard geotechnical investigation techniques should be used and results assessed by qualified geotechnical engineers. As with the study of any slope, the key issues are the structure of the ground, the strength of the materials and surfaces controlling stability and a thorough understanding of the magnitude, distribution and temporal variation of groundwater pressures. Integrity of lining systems can be compromised by differential settlement of compressible sub-grade and resulting from the presence of cavities. In addition, basal heave should always be considered in below ground landfills. Constructing liners on existing waste bodies requires the use of foundation layers that can minimise differential settlements of the liner.

10. BASAL LINING SYSTEMS

10.1 Introduction

Basal lining systems can suffer from instability due to movements in the sub-grade. A description of the failure mechanisms is given in Chapter 9, and details of the factors controlling failure and analysis methods are given in this chapter in respect to settlement and heave and in chapters 11 and 12 for slope instability involving the lining system and waste body respectively. Basal lining system performance can be significantly impaired by damage caused by excessive settlement and basal heave.

10.2 Factors Controlling Failure

Excessive settlement, both total and differential, can be caused by the construction of basal lining systems over compressible sub-grades such as soft clays, silt lagoons, peat or even previously deposited waste materials. It is important that adequate site investigation is carried out during the early stages of landfill development to allow an engineering appraisal of the likely ground conditions on which the landfill will be constructed. This is particularly important when old mineral workings are used for landfill development due to the presence of silt lagoons from the mineral washing processes. It is important that the likely settlement of the subgrade is calculated for the lifetime of the landfill, long term settlement can be the key part of any design. Details of methods for the calculation of sub-base settlement are given in Section 10.3.1 below.

Cavities or voids can develop below a basal liner from two main causes. Firstly, natural cavities can be formed normally due to groundwater flow (chalk and limestone) or chemical reaction (gypsum). Man-made cavities usually develop from mining activities or from the degradation and sudden collapse of large items (e.g. drums, fridges etc.) deposited in waste beneath the liner. Sufficient site investigation should be carried out to ensure that such cavities are not present directly beneath the basal liner, however cavities can migrate to the surface during the lifetime of the landfill.

Basal heave will occur if the pore water pressure at a given depth in the sub-grade below the basal liner is greater than the total stress from the overlying strata. This may manifest itself initially as a series of discrete wet patches on the base of the site, however excessive over-excavation can sometimes lead to significant groundwater inflow and it is not unheard of for two aquifers to become hydraulically connected in some mineral working due to basal heave.

Excessive settlement and basal heave can cause significant damage to both mineral and geosynthetic basal lining systems. Tensile and bending stresses can be induced in lining components and this can lead to the formation of cracks and shear zones in mineral liners and tears and large tensile strains in geomembranes.

10.3 Analysis Methods

10.3.1 Excessive settlement

The settlement that the basal liner will undergo is dependent both on the nature and compressibility of the sub-grade together with the vertical stress (loading) applied to it. The applied load is the weight of materials above the basal liner, i.e. protection/drainage layers, waste, daily cover layers and capping systems. A methodology for the calculation of applied vertical stress is given in Chapter 8 and is not repeated here. The mechanism for settlement in the sub-grade will depend on the nature of the material; granular materials will undergo elastic compression, saturated cohesive material will undergo consolidation and peat will undergo long-term creep settlement.

Elastic compression

Elastic compression of granular sub-grades can be calculated using the constrained modulus D, as described for waste in Section 8.3.3.

Consolidation settlement

Consolidation is the gradual reduction in volume of a fully saturated soil of low permeability due to drainage of pore water. The process continues until the excess pore water pressure set up by an increase in total stress has completely dissipated. An assessment of the likely settlement due to consolidation can be carried out by the One-Dimensional Method or the Skempton-Bjerrum Method; both methods are described in standard soil mechanics text books such as Craig (1999) or Barnes (2000).

Settlement of peat

Organic deposits such as peat, because of their fibrous structure, have non-linear consolidation characteristics. The first phase (primary consolidation) is controlled by the network of plant fibres, but as time progresses, water within the fibres begins to be expelled, giving the material a marked phase of secondary consolidation. Primary consolidation can be estimated as described above, whilst secondary consolidation (or creep) is estimated using a linear relationship on a settlement vs. log-time graph as described for secondary settlement of waste in Section 8.3.4.

10.3.2 Deformation in mineral liner

The behaviour of a mineral liner when subjected to excessive settlement is difficult to assess. Research work has been carried out in Europe to investigate the performance of mineral liners under conditions in which they lose support due to movement in the sub-grade beneath the liner. Edelmann *et al.* (1999) describe laboratory tests carried out to assess the performance of a 600 mm thick clay liner in bending. Bending was induced in the clay liner by lowering the foundation of the liner to simulate conditions for local settlement. A subsidence velocity of 4 mm per day was used. Failure was defined as the point at which the water content in a drainage layer underneath the liner increased due to flow through the liner.

Two materials were used; a plastic clay and a silt, and a summary of the Atterberg limits are given in Table 10.1.

Table 10.1 Description of the liner material used by Edelmann *et al.* (1999)

Parameters	Clay	Silt
Moisture content	17.5%	17.7%
Liquid limit	42.8%	31.4%
Plastic limit	20.6%	20.1%
Plasticity index	22.2%	11.3%

Three tests were carried out: one on the clay and two on the silt. The silt failed with a maximum settlement of 31.5 mm, with the barrier deformation corresponding to an arc of a circle, with a calculated radius of curvature of R = 70 m. On dismantling the apparatus, no cracks were observed visually. However in the clay test, a maximum settlement of 380 mm was achieved with no increase in water content in the drainage layer. The corresponding radius of curvature is 6 m.

Jessberger & Stone (1991) report on centrifuge tests carried out to investigate the effect of subsidence on clay barriers and in this investigation the two materials described in Table 10.2 below were used.

Table 10.2 Description of the liner material used by Jessberger & Stone (1991)

Parameters	Kaolin clay	Sand/Silica/Bentonite
Moisture content	32.5%	35%
Liquid limit	44.4%	-
Plastic limit	28.1%	-
Plasticity index	16.3%	-

Liner thicknesses relating to 1.75 m to 2.25 m at prototype scale were subjected to a loss of support over a section of the liner which resulted in a forced settlement. The hydraulic performance of the barriers was monitored during the testing.

Two distinct modes of failure were observed. Firstly, tension cracking was found at the top of the liner in the region of maximum liner deformation. Secondly, multiple shear ruptures were observed within the body of the liner. It should be noted that the tests which resulted in tension cracking were carried out with no overburden soils above the liner. The authors report that when overburden was present, the tension cracking was not observed and this was attributed to the overburden generating increased initial lateral stresses within the liner and thus allowing greater distortion of the liner before tensile stresses were generated.

Jessberger & Stone also note that the presence of shear ruptures in the liner did not affect the hydraulic performance of the liner. The authors state that work carried out by Henne (1989) on the bending response of compacted clay specimens has shown that the deformations necessary to induce tensile cracking increase with the plasticity of the clay.

Although the work described here does not lead to a design approach, it does indicate the importance of clay plasticity. The higher the plasticity the larger the deformations the barrier can undergo without integrity being compromised.

10.3.3 Deformation in a geomembrane liner

Giroud *et al.* (1990) explain the load carrying mechanism for the case of a soil layer overlaying a geosynthetic. Initially, the soil and geosynthetic are resting on a firm foundation, and at some point in time a void develops below the geosynthetic which deflects under the weight of the soil layer and any applied load. This has two effects; firstly the geosynthetic stretches and secondly the soil bends.

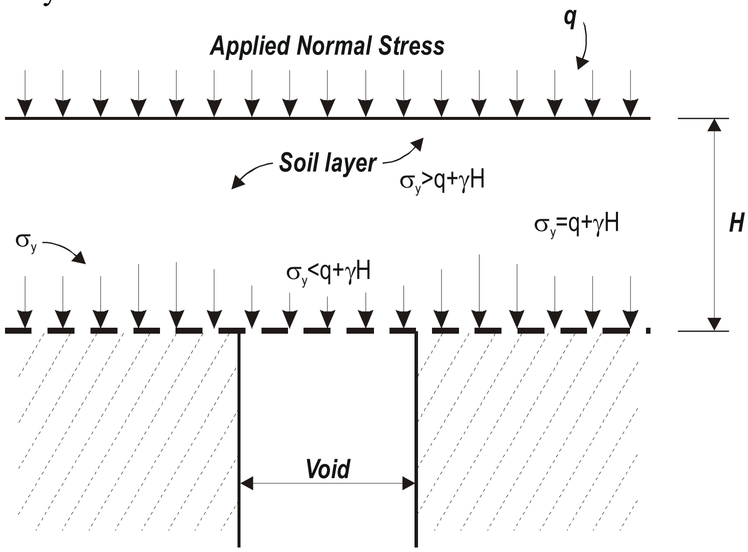


Figure 10.1 Development of arching in soil

The bending of the soil layer generates arching inside the soil, which transfers part of the applied load away from the void area, see Figure 10.1. As a result, the vertical stress over the void area is smaller then the average vertical stress. The stretching of the geosynthetic mobilises some of its tensile strength. Consequentially, the geosynthetic acts as a "tensioned membrane" and can carry a load applied normally to its surface. The soil-geosynthetic system deflects and the geosynthetic stretches until either it fails or until an equilibrium condition is reached.

When a geosynthetic deflects, arching develops in the soil layer and as a result a portion of the applied stress is transmitted laterally. Therefore the normal stress transmitted to the portion of the geosynthetic located above the void is smaller than the average vertical stress due to the self weight and applied surcharge. The approach for calculating the reduced stress transmitted to the geosynthetic was presented by Giroud *et al.* (1990). The load on the geosynthetic, ω (kN/m^2), is given by:

$$\omega = 2\gamma a\left(1 - e^{-0.5\,h/a}\right) + qe^{-0.5\,h/a} \qquad \text{Equation 10.1}$$

where:

γ is the unit weight of the soil layer, a is the width of void, h is the thickness of soil layer and q is the applied surcharge above the soil layer.

Equations, tables and charts are given by Giroud *et al.* (1990) to design soil/geosynthetic systems to span voids on horizontal surfaces. A more generic approach was presented by Jones & Pine (2001) which includes the design of inclined systems such as lining systems for vertical landfill expansion. A theory for the spanning of voids on inclined surfaces was presented by Netlon (1997) and the approach given by Jones & Pine (2001) is based on this Netlon approach.

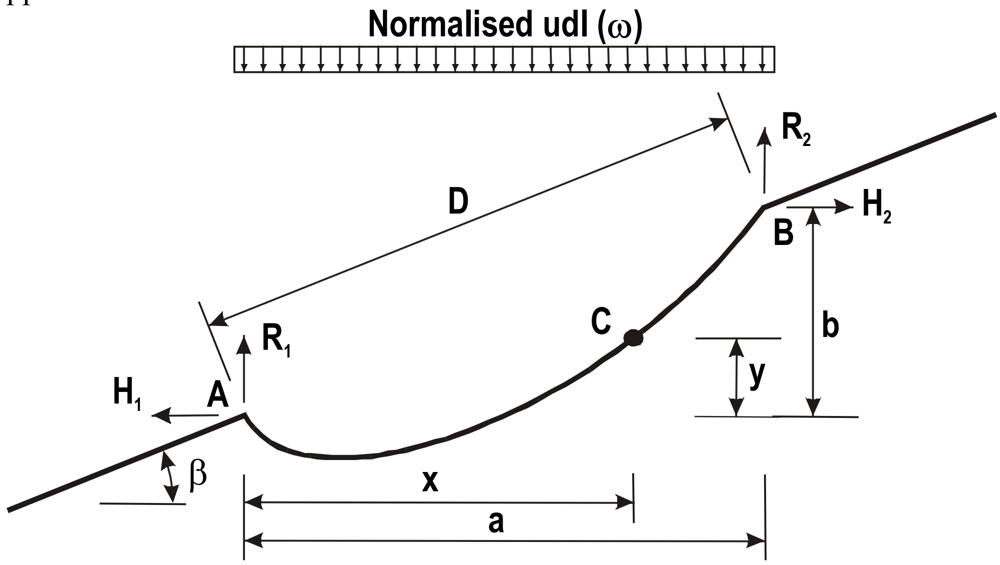


Figure 10.2 Geometry of catenary

Consider a geosynthetic placed on a surface inclined at an angle β to the horizontal, with a uniformly distributed load ω acting on it (Figure 10.2). If an infinitely long void of width a appears beneath it then the geosynthetic will deform into the void. Jones & Pine (2001) develop equations for the horizontal forces at the edge of the void and for the deformed length of the catenary. The strain in the catenary, ε, can be calculated as follows:

$$\varepsilon = \frac{L - D}{D} \qquad \text{Equation 10.2}$$

where:

L is the deformed length and D is the original length before the void developed.

$$\text{since } D = \frac{a}{\cos\beta}, \quad \varepsilon = \frac{L - a/\cos\beta}{a/\cos\beta}$$

The strain in the catenary is then calculated in terms of the original length, the void width and the gradient of the slope.

The design procedure suggested by Jones & Pine is therefore as follows:

- calculate the reduced load due to arching;
- calculate the horizontal force at the edges of the void (H) and taking an allowable tensile force in the geosynthetic;
- calculate the deformed length of the catenary and then the strain in the catenary;
- compare the calculated strain with the assumed tensile strength for the selected geosynthetic for compatibility;
- adjust as necessary (e.g. select stronger or weaker geosynthetic) and recalculate.

The critical part of the design is the choice of the void size. An assessment is therefore required of the likely size of void developed directly beneath the liner. This is difficult to accurately predict and therefore a range of values are normally used in the design.

10.3.4 Basal heave

An assessment of basal heave is carried out by calculating the factor of safety as follows:

$$\text{F of S} = \frac{\sigma_v}{u} \qquad \text{Equation 10.3}$$

where:

σ_v is the total vertical stress and u is the pore water pressure. The calculation should be carried out at the location of the groundwater (see Figure 9.6), and if there are several water bearing bands beneath the basal liner, then the calculation should be carried out for each layer. Calculations should be carried out for the temporary case where the landfill has been excavated to formation level (i.e. before the placement of the basal liner) as well as for the as-built case.

It should be noted that the factor of safety against basal heave will generally increase with time as the placement of waste in the landfill will increase the total stress. However, in many instances groundwater is lowered by the mineral extraction process and there is a great deal of pressure to turn off the dewatering systems once waste is placed in the site. Basal heave calculations often control the rate of which groundwater recharge can safely be made.

10.4 Summary of Key Issues

Basal lining systems can fail due to excessive movements in the ground beneath the liner caused by sub-grade materials that undergo large compression when subjected to loading, through the development of cavities or through basal heave. This chapter has presented the factors that control this damage and analysis methods for the various causes. Each element needs to be assessed to ensure the long-term performance of the basal lining system. Assessment of basal heave is required during construction and for any phased groundwater recharge.

11. SHALLOW SIDE SLOPE LINING SYSTEMS

11.1 Introduction

The stability of a geosynthetic landfill lining system is often controlled by the shear strength between the various interfaces, i.e. geosynthetic/soil and geosynthetic/geosynthetic interface shear strengths. The importance of interface shear strength was illustrated by the slope failure in Phase IA of Landfill B-19 at Kettleman Hills in the USA, which instigated a major investigation carried out by the University of California at Berkeley (Seed *et al.*, 1988). It has also played an important role in a number of UK failures (e.g. Case Histories No. 1 and No. 2, see Section 4.3). In the context of this report, shallow side slopes are considered to be those less than 30°.

11.2 Mobilised Strength and Strain Compatibility

Materials mobilise their shear strength at different strains. Therefore, it is possible for adjacent materials and areas of a shear surface (i.e. an interface) to have different mobilised strengths at a given instance in time (i.e. sections could have post peak values of strength close to residual while other sections are still pre-peak). This process of progressive failure is well known in soil mechanics and should be considered in the design of landfills. In limit equilibrium analyses it is necessary to allocate shear strength parameters to the shear surface and this means that a decision must be made on whether to use peak or residual values (see Report No. 2). This is a difficult decision, as it needs an assessment and understanding of strains in the lining system and waste. These can't be assessed in limit equilibrium analyses and therefore numerical modelling techniques are required (see Section 11.3.2).

A clear example of how strain incompatibility between materials can contribute to failure is given by the Cincinnati landfill failure. Figure 11.1 shows typical shear stress vs. displacement graphs for the waste and the underlying cohesive in situ soil. It can be seen that the soil mobilises peak strength at a shear displacement of 2mm while the waste requires shear displacements in the order of 40mm to mobilise peak strength. Stark *et al.* (2000) proposed that this resulted in post peak shear strength being mobilised in the soil under the waste and that this contributed to the failure.

An example of large displacements developing at geosynthetic interfaces is given by Gourc *et al*. (1997). A field experiment was conducted to measure the displacements at geosynthetic interfaces due to the placement of gravel on a 1 in 2 side slope. At the interface between a protection geotextile and a geomembrane liner it was found that:

- 70 mm displacement occurred in response to 6 m of gravel placement;
- this increased to 170 mm in response to the removal of gravel from the toe region; and
- a further increase to 650 mm of displacement in response to placing gravel to 8 m up the slope.

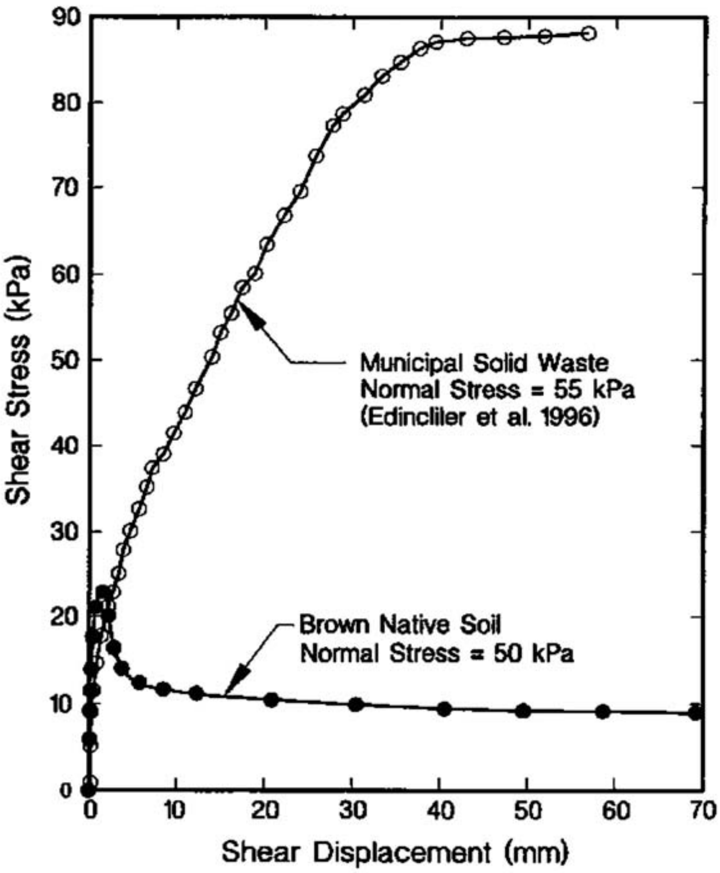


Figure 11.1 Shear stress vs. displacement graphs for waste and underlying cohesive in situ soil (Stark *at al.* 2000)

11.3 Unconfined Slopes Including Capping Systems

In terms of the stability of landfill lining systems, slopes can be considered to be unconfined or confined. Unconfined slopes are landfill side slopes prior to waste placement, i.e. before the waste is placed and provides support to the slope. Additionally, landfill capping systems are also considered to be unconfined slopes.

11.3.1 Stability: factors controlling failure

Shear strength

The measurement and selection of shear strength parameters for interfaces between soils and geosynthetics and between geosynthetics and geosynthetics are discussed in detail in Chapter 7. Shear strength parameters for the mineral layers in the lining system and the sub-grade materials should be obtained using standard soil mechanics sampling and testing procedures. Effective stress parameters should be measured unless the use of total stress parameters can be justified. A key issue in the selection of strength parameters is the mechanism of progressive failure resulting from the strain incompatibility of materials (e.g. between the waste and cohesive soils shown in Figure 11.1). A decision must be made whether to use peak or residual shear strength parameters (or values in between). This is a complex issue and is discussed in detail in Section 11.4.4 and in Report No. 2.

Groundwater

When considering the groundwater conditions that control stability it is important to measure and/or calculate the pore water pressures acting in the lining system and sub-grade. The pore water pressures modify the effective stresses within the materials and at interfaces between soils and geosynthetics, and hence they influence the stability of the lining system. The groundwater conditions in the sub-grade must be known and an assessment should be made of likely changes in groundwater due to cessation of dewatering operations.

The installation of a barrier layer can have a significant impact on the local hydrogeological conditions, i.e. water pressures can build-up behind barrier layers and there have been many instances of this causing failures (e.g. Case History No. 4, No. 6 and No. 7, see Chapter 4).

Surface water

Surface water, from both uncontrolled discharges up-slope and direct precipitation, can collect above and below a liner systems. This water can influence the stability of lining systems in three ways. Firstly, the weight of water can add to the disturbing force thus decreasing the stability. Secondly, the water will reduce the effective stress acting on the upper and lower surfaces of the lining system and reduce the strength (e.g. softening of cohesive soils), and thirdly it can apply an additional destabilising force due to seepage flow. Design calculations should include for these effects.

Location and shape of potential shear surface

The shape and position of a shear surface (also called slip surface or failure surface) is controlled by the weakest layers and interfaces (i.e. with the lowest shear strength). Lining systems comprise a number of layers and are constructed on planar surfaces (base and slope) and this means that shear surfaces readily follow the lining system. In most cases it will be the interfaces between the lining system components that will control stability. However there are instances when failure can occur through formation of shear surfaces within mineral layers (e.g. a compacted clay liner) particularly if constructed on steep slopes. High undrained shear strength will often ensure stability until after waste has been placed, however the time taken for the clay to soften and the waste to be placed are often outside the control of the designer. Section 11.4 discusses stability post waste placement.

Gas pressure

The build up of gas pressure from the landfill is relevant to the stability of capping systems and the lining of existing waste slopes. Gas generated during the degradation process acts on the underside of the low permeability barrier. Gas is often extracted from the landfill by a series of boreholes installed port-waste placement, however before such wells are installed (or indeed if such systems fail) significant gas pressures can build up. A methodology for the inclusion of gas pressure in stability analysis is presented by Thiel (1999).

Loading

The self weight of materials is very important and is used to calculate the destabilising forces in stability calculations. Increased self weight due to increased moisture content (e.g. from precipitation) can reduce the overall stability. Equipment loading during construction can be the most onerous loading conditions encountered by the lining systems. The assessment of stability should include both the dead load (self weight) of the construction plant and also the live loading due to breaking and acceleration forces. Details of the assessment of equipment loading on the stability of lining systems are given by Kerkes (1999) and Jones *et al.* (2000). Additional loading can also come from the stockpiling of materials on site. All combination of loading should be considered in the design and this could mean that a partially constructed lining system may be more critical than the same system built for its full height.

11.3.2 Integrity: factors controlling over-stressing of geosynthetic components

Geosynthetic components of lining systems are vulnerable to being overstressed. The self weight (and any imposed loading) of cover soils placed above a geosynthetic on a slope is transferred through the various geosynthetic components in shear. Shear forces mobilised at the upper surface of a geosynthetic is transferred to its lower surface by shear until the maximum shear strength of the lower surface is reached; the remaining force is then taken in tension in the geosynthetic. The amount of tension developed in the geosynthetic is dependent on the interface shear strength between the various materials. The integrity of the lining system is assessed by comparing the stress transferred into the geosynthetic with its tensile strength. It should be noted that the stresses developed should be calculated for all geosynthetics in the system.

11.3.3 Analysis methods

Limit equilibrium approach

Slope stability is typically assessed using limit equilibrium methods. It is considered that failure is on the point of occurring along an assumed, or a known, failure surface. The shear strength required to maintain a condition of limiting equilibrium (Factor of Safety (F. of S.) = 1.0) is compared with the available strength of the soil/interface. This gives the average factor of safety along the failure surface.

F. of S. = $\sum$ Restoring moments / $\sum$ Disturbing moments

The slope is considered in two-dimensions (i.e. a uniform slope of infinite length) and the forces on a 1m length of slope (i.e. along the slope) are considered. In most cases this has been shown to give conservative results (i.e. lower factor of safety values), although there are slip geometries (e.g. at the junction of two slopes) for which 3-dimensional analyses give lower factors and are considered to be a more appropriate approach.

For the reasons discussed in Section 11.4, failure surfaces in landfills are usually non-circular. There may be instances when the slip surface can be approximated to part of a circle but these will be rare and should be justified. As discussed in Section 11.2, limit equilibrium analyses cannot be used to obtain information on strains in barrier components, and hence they can't be used to assess barrier integrity.

Slopes can fail during construction due to the disturbing forces resulting from the slope geometry exceeding the resisting forces from the material strengths. Factors that can cause existing 'stable' slope to fail include:

- change in pore water pressures (e.g. due to increased leachate levels, increases in groundwater pressures, changes in drainage conditions, rainfall events);
- change in loading (e.g. placement of materials on the slope, stockpiling of material at top of slope, removal of materials from toe of slope, or equipment forces); and
- vibrations (e.g. from blasting, equipment and seismic events).

A stable slope can be easily transformed into an active slide mass by the above changes. The role of the waste degradation process on long-term shear strength of waste, and hence stability of waste slopes, is as yet unknown. Due to the time related changes in pore water pressure that can occur in cohesive sub-grade (slopes and base) it is often necessary to assess both the short-term (i.e. during and at end of construction) and the long-term stability conditions, see Chapter 9.

Infinite slope method

The stability of a cover soil (i.e. including veneer soil layers placed during lining construction) above the geosynthetics was discussed by Martin & Koerner (1985), who, using an infinite slope approach, presented the factor of safety against the failure of a uniform cover soil as:

$$F = \frac{\tan\delta}{\tan\beta} \qquad \text{Equation 11.1}$$

where:

δ is the friction angle between the geomembrane and cover soil and β is the slope angle.
The above equation applies when the cover soil is dry, however such conditions are uncommon as there is usually some form of active seepage in the cover soil. For full depth seepage, Martin & Koerner (1985) suggest an approach based on a reduction in effective normal stress on the liner, i.e.

$$F = \frac{\gamma_b \tan\delta}{\gamma_s \tan\beta} \qquad \text{Equation 11.2}$$

where:

γ_b is the buoyant unit weight of cover soil, and γ_s is the saturated unit weight of cover soil.

Note that $\gamma_b = \gamma_s - \gamma_w$, where γ_w is the unit weight of water. Further, Equation 11.2 is only valid if the hydraulic gradient is numerically equal to the tangent of the slope angle. This however, only accounts for buoyancy effects and does not consider seepage forces.

Finite slope method

Giroud & Beech (1989) give two reasons why a finite slope is more stable than an infinite slope assumed in the analysis method described above; the presence of a geosynthetic anchorage at the crest, and the buttressing effect of the soil at the base of the slope. As slippage along the critical geosynthetic interface occurs, tensile forces are generated in the geosynthetics above the critical interface, and these tensile forces contribute to the stability of the potential sliding block. The authors summarise the three factors contributing to the stability as:

- geosynthetic tension resulting from the crest anchorage;
- shear resistance developed along the interface; and
- toe buttressing effect.

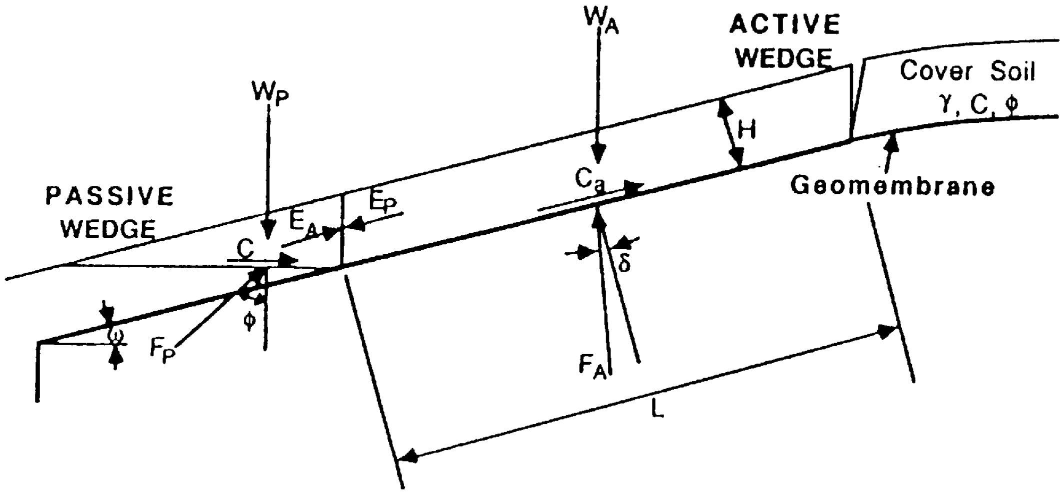


Figure 11.2 Wedge analysis method for finite slopes (after Koerner & Hwu, 1991)

In the limiting equilibrium method proposed, Giroud & Beech (1989) divide the system into two wedges and forces are balanced in the vertical and horizontal directions. This method provides two equilibrium equations and three unknowns, and an iterative process is required to provide a solution. A major drawback with this method is that the distribution of tensile stresses within the geosynthetic layers cannot be determined.

Koerner & Hwu (1991) proposed a limiting equilibrium method also based on the two part wedge method, and considered sliding of the active wedge to be resisted by only the shear strength along the geosynthetic/cover soil interface and the passive soil wedge buttress at the toe of the slope (Figure 11.2).

The factor of safety with respect to sliding of the system is a solution of the following quadratic equation:

$$aF^2 + bF + c = 0 \qquad \text{Equation 11.3}$$

where:

$$a = \frac{\gamma HL}{2}\sin^2(2\beta)$$

$$b = -[\gamma HL\cos^2\beta\tan\delta_u\sin(2\beta) + \alpha_u L\cos\beta\sin(2\beta) + \gamma HL\sin^2\beta\tan\phi\sin(2\beta) + 2cH\cos\beta + H^2\tan\phi]$$

$c = (\gamma HL\cos\beta\tan\delta_u + \alpha_u)(\tan\phi\sin\beta\sin(2\beta))$

and,

γ is the unit weight, H is the thickness of cover soil, L is the slope length, β is the slope angle, ϕ is the angle of internal friction of cover soil, c is the cohesion of cover soil, δ_u is the interface friction angle at the upper interface, α_u is the apparent cohesion at upper interface

This approach assumes that the factor of safety is the same value at every point along the sliding surface defined by the two wedge mechanism. By default this means that the factor of safety is the same with respect to the shearing resistance at the active wedge/geosynthetic interface as that with respect to the shearing resistance of the cover soil beneath the passive wedge. Further, this analysis does not consider the effects of pore water pressures at the interface.

Koerner & Hwu (1991) further proposed a model to assess the tension in a geosynthetic due to unbalanced interface shear forces, see Figure 11.3. By assuming uniform mobilisation of the interface shear strengths along the geomembrane, the authors developed an expression for the tensile force per unit width of slope induced as follows:

$$T = [(\alpha_u - \alpha_l) + \gamma H\cos\beta(\tan\delta_u - \tan\delta_l)]L \qquad \text{Equation 11.4}$$

where,

δ_l is the interface friction angle at the lower interface, α_l is the apparent cohesion at lower interface

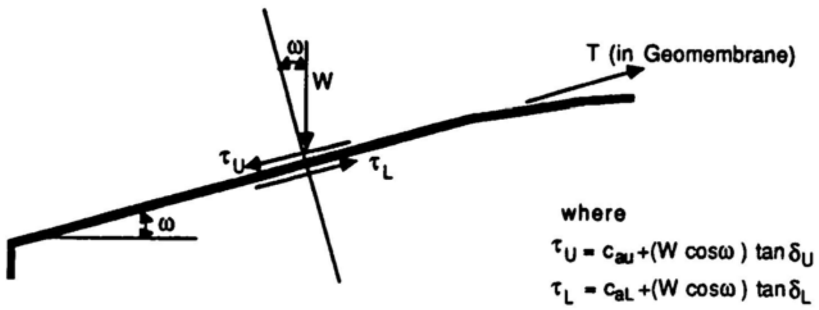


Figure 11.3 Model for tension in a geosynthetic (after Koerner & Hwu, 1991)

This equation expresses the imbalance between the maximum shear force that can act at the geomembrane upper interface and the maximum shear force at the lower interface. When the upper shear force is smaller than the force at the lower surface the geomembrane is in equilibrium and is not stressed. However, when the upper shear force is greater than the lower, a tensile force T is required in the geomembrane to ensure equilibrium.

This approach to calculate the tensile force in the geosynthetic was adopted by Quinn & Chandler (1991) in the analysis of a multi-layered geosynthetic lining system. They demonstrated the use of geogrid and geotextile veneer reinforcement to dissipate shear stresses in the upper layers of a multi-layered system to minimise tensile stresses induced in the geomembrane. The use of geogrid veneer reinforcement is also discussed by Heerten & Scheu (1990) and Hall & Gilchrist (1995).

A major shortcoming with this method is that the tensile force computed in Equation 11.4 is independent of the level of shear stress effectively mobilised at the upper interface. The shear force at the upper interface in this equation should be the mobilised shear force. Bourdeau *et al.* (1993) proposed a coupling between equations 11.3 and 11.4 by replacing the ultimate upper shear strength with a mobilised value calculated by dividing the ultimate value by the factor of safety calculated in Equation 11.3, i.e.

Replacing $\alpha_u + \gamma H \cos\beta \tan\delta_u$

By $\dfrac{\alpha_u + \gamma H \cos\beta \tan\delta_u}{F}$

which gives a new expression for the tensile force in the geosynthetic:

$$T = \left[\left(\frac{\alpha_u}{F} - \alpha_l\right) + \gamma H \cos\beta\left(\frac{\tan\delta_u}{F} - \tan\delta_l\right)\right] L \qquad \text{Equation 11.5}$$

For a multi-layered system, the limit method proposed by Koerner (1998) can be used to determine the tensile forces in subsequent lower layers. This is a force equilibrium procedure that balances forces in the direction parallel to the slope. The shear force mobilised in the upper surface of a geosynthetic is transferred to its lower surface by shear until the maximum shear strength of that interface has been reached, and the remaining force will be taken in tension in the geosynthetic.

A two part wedge analysis has also been proposed by Druschel & Underwood (1993), with additions to take into account of loadings due to seepage forces and equipment working on the slope. The authors treat seepage forces as a negative static load on the basis of the buoyancy applied to the cover soil, i.e. the frictional shear stresses are reduced because of the reduction in effective normal stress, however they do not consider seepage forces. Equipment operating on the side slope have a static weight that acts in the same manner as the active wedge cover soil. However, additional forces are generated by acceleration and braking. Druschel & Underwood (1993) use the suggestion proposed by Richardson & Koerner (1987) that these forces should be treated as a separate breaking force equal to 30% of the equipment's weight and acting downslope, parallel to the interface.

A parametric evaluation is presented by Druschel & Underwood (1993) which demonstrates the impact of each variable on stability. The factor of safety is taken as the ratio of actual shear strength to the mobilised shear stress, and example calculations are presented which demonstrate the importance of anchorage forces in increasing the factor of safety. Druschel & Underwood (1993) state that each material used in a geosynthetic lined slope have different shear stress-displacement behaviour, and thus there is a strain compatibility issue. They suggest that residual shear strengths are used in design instead of peak values to ensure conservative design.

Soong & Koerner (1996) consider a granular cover soil with an internal friction angle of ϕ, and in the consideration of seepage forces this is satisfactory. In addition, the interface shear strength between the upper geosynthetic and the cover soil is only represented by a friction angle (δ). In an attempt to make this approach more generic, the effect of a cover soil with cohesion (c) and an interface with a cohesion intercept of α, the equations were re-written by Jones & Dixon (1998b) to include these terms. The inclusion of these parameters changes the b and c terms in the quadratic equation as follows:

$$b = -\left[W_A \sin^2 \beta \tan\phi\right] + \left[U_h \sin\beta \cos\beta \tan\phi\right]$$

$$-\left[\cos\beta\left((\alpha L) + N_A \tan\delta\right)\right] - \left[(W_P - U_v)\tan\phi\right] - \left[\frac{ch}{\sin\beta}\right] \qquad \text{Equation 11.6}$$

$$c = \sin\beta\tan\phi[\alpha L + N_A\tan\delta] \qquad \text{Equation 11.7}$$

Further, the stress normal to the interface used in the calculation of the geosynthetic tensile force (Equation 11.5) should take account of the piezometric surface. This equation now becomes:

$$T = \left[\left(\frac{\alpha_u}{F} - \alpha_l\right) + \left(\gamma_{sat} h_w + \gamma_d (h - h_w)\right)\cos\beta\left(\frac{\tan\delta_u}{F} - \tan\delta_l\right)\right].L \qquad \text{Equation 11.8}$$

It is proposed that the stability of a cover soil over several layers of geosynthetics together with the tension developed in the geosynthetics can be established as follows:

1. Calculate the factor of safety against cover soil sliding using the approach of Soong & Koerner (1996), modified to allow for c and α.

2. Calculate the mobilised tension in the upper geosynthetic using Bordeau *et al.* (1993) with the modification for γ_{sat} and γ_d.

3. Calculate the mobilised tension in the remaining geosynthetics.

Prior to the development of textured geomembranes, the interface shear strength between geosynthetics and cover soils was generally low and thus it was common practice to taper the cover soil with a thicker zone at the base. Since the above infinite slope methods cannot be used for such situations, Martin & Koerner (1985) suggest the use of a wedge analysis. In this method, the geometry is divided into the active wedge which is tending to cause failure and the neutral block which is tending to resist failure and an example of an iterative graphical solution method is presented by the authors. A similar example of the analysis of a tapered cover soil is presented by Giroud & Ah-Line (1984), in which the stability of earth and concrete covers for reservoirs is analysed. The classical wedge analysis is again used to determine the overall stability, however the authors also consider the stresses induced in the geosynthetics, when geomembranes are used for remedial works and placed over cracked concrete canal linings. A limit method of transferring shear stresses through the geosynthetics was introduced.

Assessment of gas pressure

Thiel (1999) has proposed a method for assessing the build up of gas pressure beneath a geomembrane barrier. Pore pressures generated by landfill gas can be shown to significantly reduce the effective normal stress on the lower geomembrane interface. This can lead to instability (e.g. of a cover veneer). The method relies on an estimation of gas flux from the landfill surface to design a gas relief layer beneath the geomembrane. This can take the form of a sand layer or drainage geocomposite.

Standard soil mechanics methods can be used to assess stability when excess gas pore pressures are present. Thiel (1999) recommends the following steps to incorporate gas pressures in a landfill design:

Estimate the maximum gas flux that may need to be removed from below the geomembrane;
perform slope stability analyses to estimate the maximum allowable gas pressure;
design a vent system below the geomembrane that will evacuate the assumed gas flux under the estimated maximum allowable driving pressure.

Estimating gas flux

This will be site specific and will vary spatially and temporally at a given landfill. The amount of gas will depend on the waste type, age, temperature, moisture content, gas extraction systems etc. Experts in landfill gas engineering should be consulted to obtain estimates of gas flux for a particular site.

Slope stability including gas pressures

Gas pressures can be incorporated into any standard stability analysis method. In this application the gas will act on the underside of the geomembrane and therefore the failure mechanism to be assessed is sliding of the geomembrane on the underlying material. Sliding above the geomembrane should be assessed separately (i.e. including such issues as seepage forces and plant loading).

As an example, gas pressures have been incorporated into the equation for stability of a translational slide in Equation 11.9 (after Thiel, 1999). Note that gas pressures is incorporated in the same way as pore water pressure.

$$\text{F of S} = \frac{\alpha' + \left(h.\gamma.\cos\beta - u_g\right)\tan\delta'}{h.\gamma.\sin\beta} \qquad \text{Equation 11.9}$$

where:

h is the cover soil thickness above the geomembrane and measured perpendicular to the slope, γ is the unit weight of soil above the geomembrane, β is the slope angle, u_g is the gas pressure on the underside of the geomembrane, α' is the effective apparent adhesion parameter for the geomembrane/underlying soil interface and δ' is the effective friction angle for the geomembrane/underlying soil interface.

Once an allowable factor of safety has been selected the maximum allowable gas pressure can be calculated.

Gas pressure relief system

The gas pressure relief system is designed to ensure that the maximum allowable gas pressure is never exceeded. Thiel (1999) proposed that a system should comprise:

A blanket gas relief layer;
a series of parallel high permeability trenches or strip drains that collect the gas from the gas relief layer; and
outlet points for the strip drains.

The design of the blanket and strip drains is covered in detail by Thiel (1999).

Gas pressure has caused failures. The above method can be used to assess the impact of gas pressure on stability and to design a relief system. However, estimation of gas pressures is a difficult process and involves many assumptions and experience. Maximum estimated gas pressures should be used in design (i.e. not average values) due to these inherent uncertainties.

Assessment of plant loading

One of the main construction effects on the development of interface shear strength (and therefore on the overall stability) is the loading of the lining system by the earth moving equipment. The equipment may take the form of excavators, dozers, graders etc. and each will have different loading characteristic.

Several authors have developed methodologies for the inclusion of equipment loading on the overall veneer stability of a geosynthetic lined slope. Druschel & Underwood (1993) consider a sliding block limit equilibrium analysis which takes into account the forces due to equipment loading by the addition of the equipment's self weight, together with a force due to breaking/accelerating. The breaking load is taken to be 30% of the equipment's self weight as suggested by Richardson & Koerner (1987). A more rigorous approach to equipment loading is given by Koerner and Soong (1998a) where the actual acceleration of the equipment is used in the analysis. This method also uses the sliding block approach.

Kerkes (1999) criticises the conventional sliding block approach since the passive block at the toe of the slope provides support to the active block irrespective of where the equipment may be located on the slope. Hence, only one global failure scenario is considered and a localised failure in the vicinity of the equipment is not assessed. Kerkes (1999) proposes a sliding block analysis that considers three (active, central and passive) blocks above the potential failure surface.

In all the above methodologies, no account is taken of the strain incompatibility between the various soils and geosynthetics that make up the lining and cover soil system. The use of construction equipment on a geosynthetic lined slope will certainly have a detrimental effect on the shear strengths mobilised along the interfaces (e.g. Case History No. 2, Section 4.3). The consequence of using common types of tracked dozers on mobilised shear strength is considered by Jones *et al.* (2000). They conclude that strain softening behaviour of lining interfaces is an important consideration when cover soils are traversed by construction plant.

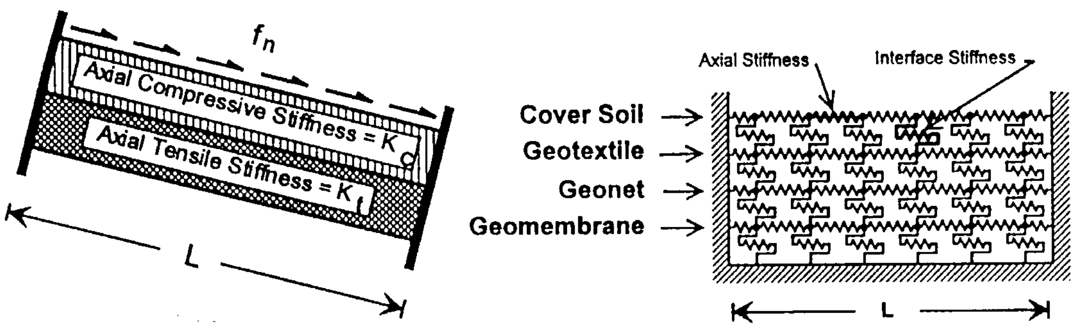
Post peak shear stresses can be mobilised, possibly reducing to residual directly under heavy plant. This could lead to local failure of the protection and hence damage to the geomembrane.

Assessment of strain softening interfaces requires the use of numerical modelling techniques which are unlikely to be used in most design cases. The approach proposed by Kerkes (1999) is recommended, but consideration should be given to the possibility of strain softening interfaces controlling behaviour. Keeping equipment loads to a minimum, particularly on thin veneers of soil, will minimise the risk of developing post-peak interface strengths.

Assessment of geosynthetic stress

The method suggested by Jones & Dixon (1998) and described above can be used to calculated the stresses induced in geosynthetic elements on a slope. The model is based on the transfer of shear stresses through the various layers of geosynthetic and does not take account of the stiffness of the components.

Two methods for assessing the loads on geosynthetics in landfill slopes have been proposed by Long *et al.* (1994). The first method approximates a composite layered system as an axially loaded composite column which exhibits an axial stiffness in both compression (modelling the soil layer) and tension (modelling the geosynthetic), see Figure 11.4a. Since no slippage is assumed at the interface between the two structural layers, the two columns must strain equally. A drawback of this approach is that the interface shear strength is assumed to be independent of displacement and it cannot therefore be recommended.



(a) Simple composite column model (b) Finite difference model

Figure 11.4 Models used for assessing loads on geosynthetics (after Long *et al.*, 1994)

The second method presented by Long *et al.* (1994) is a more rigorous method which takes into account changes of the interface shear strength with displacement. This approach uses a finite difference model that includes non-linear mechanisms to model the shear stress-displacement behaviour at each interface, and to model the axial load-displacement behaviour within each component (Figure 11.4b).

11.4 Confined Slopes (Post Waste Placement)

Failures of confined landfill slopes usually incorporate underlying weak layers; interfaces between elements of lining systems, temporary cover soil layers and the waste body. There are no published case histories of major waste slope failures that have occurred in the UK, although they do occasionally occur (e.g. Case history No. 5, see section 4.3). There are however a number of case histories in the literature relating to major failures in landfills located in both developed (e.g. USA, Italy) and developing (e.g. South Africa, Columbia) countries. Koerner & Soong (1998b) summarised the key factors that resulted in ten significant waste slope failures. Table 11.1 (after Koerner & Soong 1998b) gives information on the location, year, mode of failure and volume of waste involved. The failures described as non-circular in the table were defined as translational by Koerner & Soong (1998b). This is a difference of definition, not of interpretation.

Table 11.1 Summary of major waste failures (after Koerner & Soong 1998b)

Case History	Year	Location	Type of failure	Quantity of waste involved
Unlined sites				
U-1	1984	N. America	Single rotational	110,000 m^3
U-2	1989	N. America	Multiple rotational	500,000 m^3
U-3	1993	Europe	Non-circular	470,000 m^3
U-4	1996	N. America	Non-circular	1,100,000 m^3
U-5	1997	N. America	Single rotational	100,000 m^3
Lined sites				
L-1	1988	N. America	Non-circular	490,000 m^3
L-2	1994	Europe	Non-circular	60,000 m^3
L-3	1997	N. America	Non-circular	100,000 m^3
L-4	1997	Africa	Non-circular	300,000 m^3
L-5	1997	S. America	Non-circular	1,200,000 m^3

A significant number of these failures have occurred in landfills designed, constructed and operated using methods comparable to present UK practice. Clearly there are lessons to be learnt from these failures. There is a preconception that waste slope failures only occur in valley infill and land raise type landfills and that as there are a relatively small number of such landfills in the UK, the risk of waste failure is small. Experience does not support this view. Slope instability involving temporary waste slopes can occur during filling of below ground cell based landfills. Examples of this are provided by the case histories 1 & 5 (see section 4.3) and the Kettleman Hills failure the USA (Byrne *et al.* 1992).

11.4.1 Stability: factors controlling failure

Shear strength

Shear strength of waste is discussed in Section 8.4. Effective stress shear strength parameters c' and ϕ' are required. Given the present uncertainty on the measurement and interpretation of shear strength data, and the constantly changing composition of waste, it is recommended that a range of values be selected and a thorough sensitivity analysis conducted. The measurement and selection of shear strength parameters for interfaces between soils and geosynthetics and between geosynthetics and geosynthetics are discussed in detail in Chapter 7. Shear strength parameters for the mineral layers in the lining system and the sub-grade materials should be obtained using standard soil mechanics sampling and testing procedures. Effective stress parameters should be measured unless the use of total stress parameters can be justified. A key issue in the selection of strength parameters is the mechanism of progressive failure resulting from the strain incompatibility of materials (e.g. between the waste and cohesive soils). A decision must be made whether to use peak or residual shear strength parameters (or values in between).

Groundwater

When considering the groundwater conditions that control stability it is important to measure and/or calculate the pore water pressures acting in the lining system and sub-grade. The pore water pressures modify the effective stresses within the materials and at interfaces between soils and geosynthetics, and hence they influence the stability of the waste mass. An example is the pore water pressures in a basal mineral liner, or cohesive soil sub-grade. As cohesive soils have a low permeability, the rate of consolidation will be slow. If waste is place rapidly above the liner (i.e. in relation to the rate of consolidation) an undrained loading condition will occur. This means that there will be little increase in shear strength of the soil during waste placement and hence the as placed shear strength of the material will control stability.

If a layer of soft cohesive soil is present (this could be a thin layer only a few centimetres thick) the low strength of this material will control stability as it will form a preferential path for the potential shear surface. Only with time, as consolidation takes place, will the strength of this material increase and hence with it, the stability of the slope. An example of this type of behaviour is the Case history No. 1 (see Section 4.3). A layer of softened mineral barrier was left in place beneath the geomembrane and although several metres of waste were placed above the barrier, the strength of the softened material did not change in the period of filling. This low strength layer led to failure of the temporary waste slope. Proof that the soft layer still remained after filing and that it controlled failure were provided by the post failure investigation (i.e. visual assessment and strength testing).

Leachate

The quantity, location and pressures generated by leachate have a controlling influence on landfill stability. Leachate affects stability in two main ways: i) increased amounts of leachate result in higher waste bulk unit weights and hence an increase in gravitational destabilising forces (see Section 8.2); and ii) leachate pressures reduce effective stresses in the waste and barrier materials which reduces their shear strength and hence results in decreases in stabilising forces. Koerner & Soong (2000) have produced a useful discussion of factors influencing leachate in landfills and the affect on stability. A brief summary of the their findings is presented below.

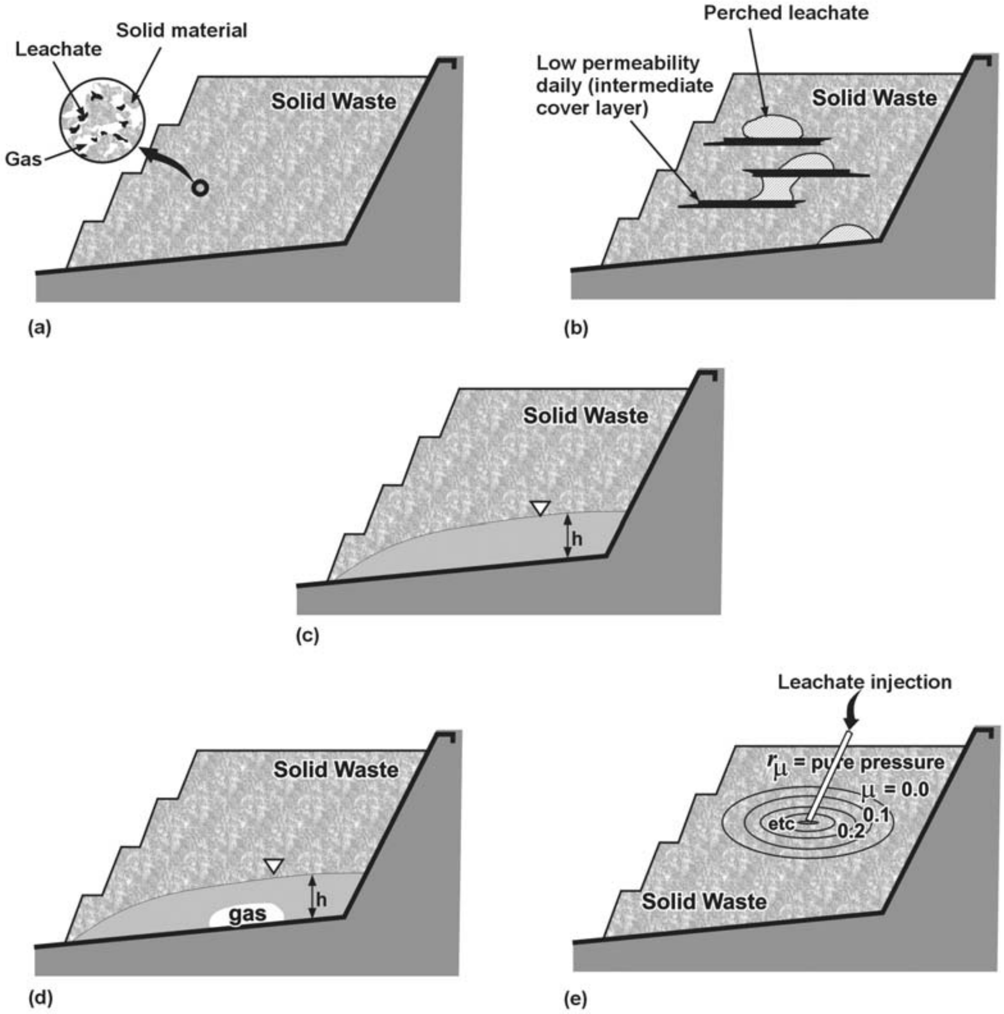


Figure 11.5 Possible leachate distribution scenarios in landfills a) discontinuous, b) perched or localised, c) head on liner, d) head above gas on liner, and e) excess pore pressure (Koerner & Soong 2000)

Koerner & Soong (2000) proposed that leachate distributions in landfills could be described by one of the following types. They are listed in order of lowest to highest total quantities of leachate per unit volume of waste:

- discontinuous leachate;
- perched (or localised) leachate;
- leachate head on liner;
- leachate head on liner with gas entrapment;
- leachate under excess pore pressure.

The scenarios are illustrated in Figure 11.5. It is possible for multiple combinations of the scenarios to exist in any given landfill. Unfortunately, there is a dearth of high quality field monitoring of leachate pressures and therefore little corroborative evidence exists, although the proposed scenarios are based on well-understood scientific principles supported by some field observations. The specific influence of the five leachate scenarios are given below.

Discontinuous leachate

This is believed to occur at degrees of saturation less than 50% (i.e. when less than 50% of the voids between and within the waste particles (inter and intra-particle voids) are filled with leachate). The leachate increases the bulk unit weight of the waste and this results in a decrease in the factor of safety against slope failure through an increase in the gravitational destabilising forces. The implication is that high (and hence conservative) unit weights should be used in stability analyses. Koerner & Soong (2000) provided an example of how the factor of safety can change with increasing unit weight by carrying out a parametric study of case history L-3. A summary of the results is shown in Figure 11.6.

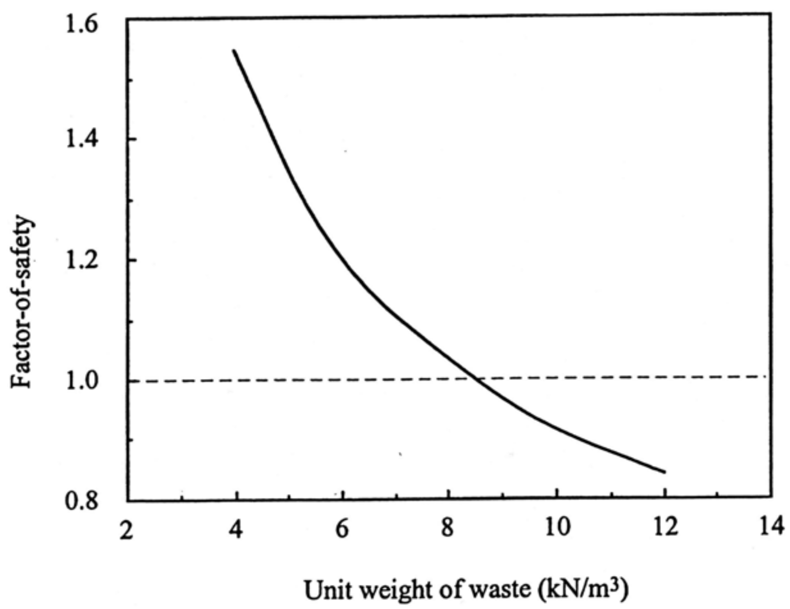


Figure 11.6 Effect of unit weight on factor of safety obtained from a parametric study of case history L-3 (Koerner & Soong 2000)

Perched leachate

When the degree of saturation approaches values of 90% and higher it is possible for the leachate to sustain hydrostatic pressures. Zones of increased degree of saturation will form above low permeability layers (e.g. cover soil layers). The high degree of saturation will increase the bulk unit weight of the waste and decrease stability as discussed above. The hydrostatic leachate pressures will reduce effective stresses in the waste and low permeability zones, resulting in a reduction in shear strength and decrease in the factor of safety.

Leachate head on liner

Although leachate collection systems are designed to restrict the head on the lining system (e.g. to a maximum of 1 m), it is common for higher heads to be experienced during the design life of the landfill due to damage and changes in the drainage system performance. Higher leachate heads produce lower effective stresses and hence a decrease in factor of safety. An example of where high leachate heads on the basal liner resulted in failure is the Dona Juana landfill failure in Columbia (Hendron *et al.* 1999).

Leachate head, with gas entrapment, on liner

Koerner & Soong (2000) postulate that gas bubbles within saturated waste will result in higher leachate heads (i.e. the gas displaces leachate from the voids thus resulting in the higher head). The effect on stability is through the same mechanism as for just the leachate head but as the heads are greater, the factor of safety will be lower (i.e. the presence of gas gives the worst case). There is presently no field evidence to confirm that this mechanism occurs.

Leachate under excess pore pressures

Leachate pore pressures greater than hydrostatic can be generated by activities such as leachate re-circulation. The higher the pore pressures the lower the effective stresses and hence the lower the factor of safety against slope failure. Leachate re-circulation under pressure was a factor in the build up of leachate on the basal liner of the Dona Juana landfill in Columbia (Hendron *et al.* 1999), and hence was a contributing factor to the failure. It is also believed that leachate pumped into the waste mass close to the interface between two phases of filling was a contributing factor in the failure of the Kwazulu-Natal landfill in South Africa (Brink *et al.* 1999).

A factor additional to those considered by Koerner & Soong (2000) is the decrease in permeability coefficient (also called hydraulic conductivity) of waste with increasing confining stress. As the overburden (vertical) stress increases with depth in a landfill the unit weight increases (see Section 8.2) and voids become smaller resulting in a reduction in permeability coefficient. Tests conducted in a large-scale compression chamber by Powrie & Beaven (1999) show that the permeability of MSW decreases by several orders of magnitude when the vertical stress increases from 20 to 900 kPa (Figure 11.7).

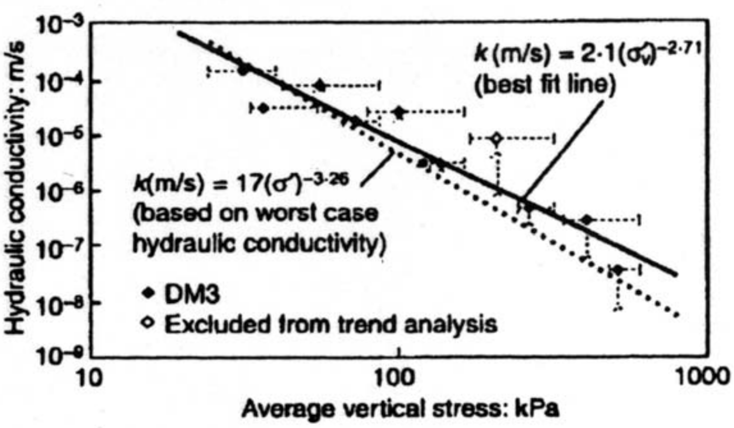


Figure 11.7 Variation of saturated permeability with vertical effective stress for MSW (Powrie & Beaven 1999)

The implication of the decreasing permeability with depth is that the effectiveness of the basal drainage system for controlling leachate pressures in the body of the waste is reduced. The lower permeability at depth effectively isolates the upper layers of waste from the under-drainage Pore pressure distributions close to hydrostatic are then possible in the upper part of the landfill even if the drain at the base is controlling the head to say 1 m. Figure 11.8 shows an idealised (i.e. ignoring cover soil layers) leachate pore pressure vs. depth distribution that can be obtained with decreasing permeability. The presence of low permeability cover soil layers within the waste body would complicate the pore pressure distribution but the general trend of higher pore pressures in the upper layers of waste would not be altered.

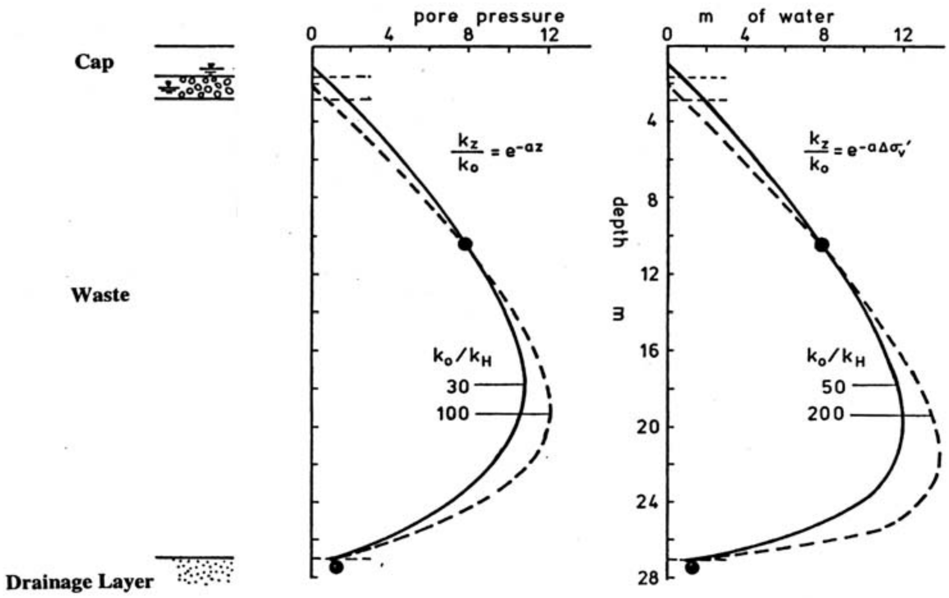


Figure 11.8 Influence of depth dependent permeability on pore pressures in an under-drained waste mass

Location and shape of potential shear surface

The shape and position of a shear surface (also called slip surface or failure surface) is controlled by the weakest layers and interfaces (i.e. with the lowest shear strength). Lining systems comprise a number of layers and are constructed on planar surfaces (base and slope) and this means that shear surfaces readily follow the lining system. Thus the use of lining systems in landfills introduces potential instability. However, they also represent the most obvious locations of surfaces to be checked as part of the design. Controls on the shear surface are summarised below and examples are given of failures where specific controls have been dominant.

Sub-grade

Weak layers of in situ or fill soils can underlie the lining system. Cohesive soils cause most problems. Material can be naturally weak (e.g. soft alluvial clay deposits) or can be softened by exposure to climatic events (e.g. stiff clay sub-grade left exposed to precipitation during the construction process and not removed prior to liner construction). In situ materials often contain planes with reduced shear strength (e.g. bedding planes and pre-existing shear surfaces from past tectonic or slope movements). An example of where the sub-grade has controlled the shape and position of a significant section of the shear surface is the Cincinnati landfill failure (Eid *et al.* 2000, Stark *et al.* 2000). Figure 11.9 shows both the pre and post-failure ground profiles. The basal section of the shear surface followed a layer of in situ cohesive soil on which the waste was placed. Other important factors contributed to the failure and these are highlighted in later sections.

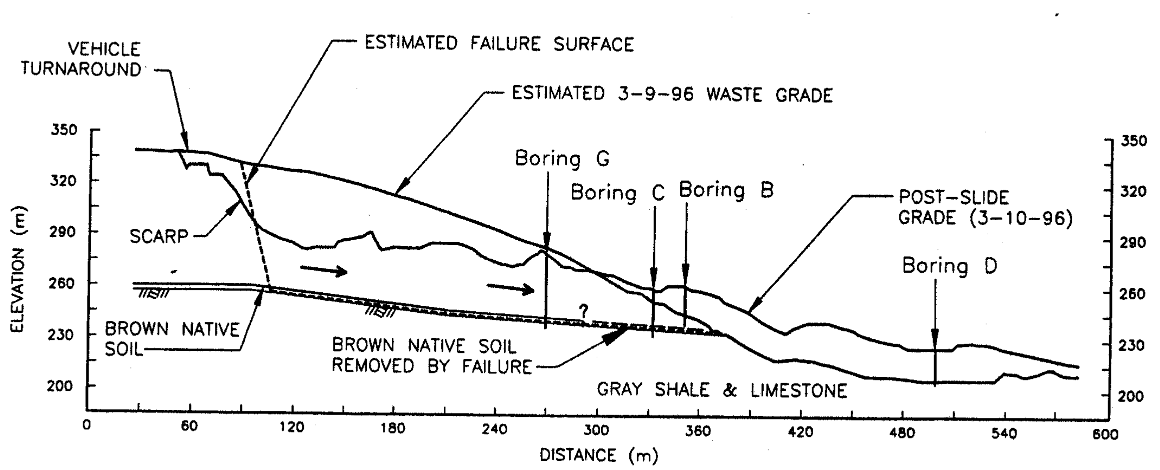


Figure 11.9 Cincinnati landfill failure in USA (Eid *et al.* 2000)

Lining system components and interfaces between them

It is possible for the critical failure surface to pass through the mineral liner and therefore this should be considered as part of design. Mineral liners are primarily designed to ensure low permeability. It is common practice to compact the soil wet of the optimum moisture content as this helps achieve the design permeability. However, this has the affect of reducing the shear strength (i.e. related to the reduction in dry density achieved at moisture contents wet of optimum). The shear strength of the mineral liner material should be known over the range of moisture contents that are allowed during placement, and stability checked for these shear strengths. Construction quality control must ensure that the field shear strength is in accordance with the design assumptions.

Interfaces between soils, soils and geosynthetics and between geosynthetics often form the weakest planes within the slope. The shear strength of these has been considered in detail in Chapter 7. An example of a failure where the interfaces between lining components controlled the slip is the Kettleman Hills failure (Mitchell *et al.* 1990). Figure 11.10 shows the pre-failure ground profile. The position of the failure surface followed two interfaces. Along the base and the lower part of the slope it was located at the interface between the geomembrane and geotextile. In the upper part of the slope it was located at the interface between the geomembrane and clay mineral liner. The location of the failure surface changed because at different stress levels (i.e. related to height of waste overburden) the interface with the critical (i.e. lowest) shear strength changed. The 1997 Bulbul Drive landfill failure in Kwazulu-Natal, South Africa, is also an example of where the lining system provided a path for the basal part of the failure surface (Brink *et al.* 1999). The 1989 failure in the Pescantina landfill, Verona, Italy (Mazzucato *et al.* 1999) is also an example of this.

Daily cover soil layers

The waste mass is often stratified with daily cover soil layers. Depending on the site practices for waste placement, and the depth of burial, these layers of soil can have a vertical spacing as small as 1m. They are usually sub-horizontal and extend over a significant area. Depending upon the thickness of the layer and the soil type (i.e. cohesive or granular) it is possible for these layers to form preferential paths for the critical failure surface. A contributing factor is that these layers often have permeability lower than the waste and therefore perched leachate levels form. This reduces the effective stresses in the soil and hence results in lower shear strengths.

Through waste body

Observations of failures have shown that the rear sections of failure surfaces often pass through the waste and are usually at steep angles (i.e. 60º to vertical). This is shown by the cross-sections through the Cincinnati (Figure 11.9) and Bulbul Drive (Figure 11.11) failures. The steepness of the failure plane is due to the reinforced nature of MSW (i.e. high shear strength) and the mobilisation of disturbing forces generated by the waste mass. Tensile strength of the waste plays as important a role in the failure as shear strength, particularly when slope movements occur on a sub-horizontal basal shear plane and hence are essentially translational.

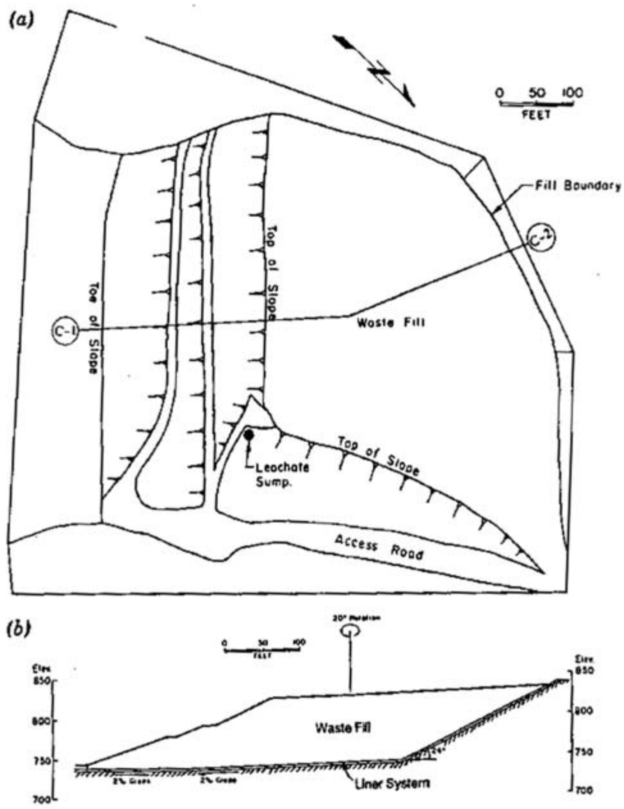


Figure 11.10 Kettleman Hills landfill failure in USA (Mitchell *et al.* 1990)

In cases where phases of waste filling result in lateral expansion of the slope (i.e. the slope angle remains the same but the position of the slope moves laterally) planes of weakness can be formed within the waste mass along the boundaries between phases of filling. This mechanism played an important role in the Bulbul Drive landfill failure, with the rear part of the failure surface forming within the waste at the boundary between two distinct phases of filling (Brink *et al.* 1999). The boundary between two phases of filling is weaker because there is no reinforcing effect of the waste across it. Figure 11.11 shows views of the landfill shortly before and following failure. A cross-section through the failure is shown in Figure 11.12. Following this failure, recommendations were made that waste slopes should be benched prior to new phases of filling to increase interlocking.

Figure 11.11 Bulbul Drive landfill shortly before and following failure (Brink *et al.* 1999)

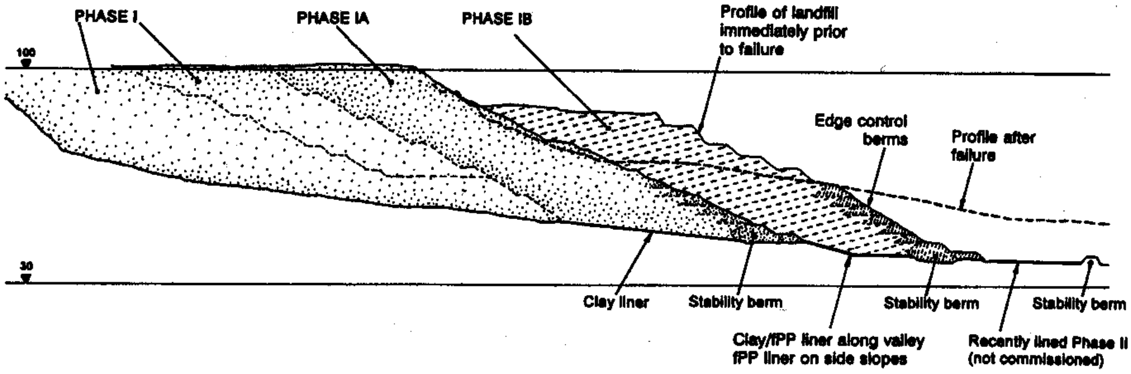


Figure 11.12 Cross-section through Bulbul Drive landfill showing location of shear surface (Brink *et al.* 1999)

Analysis of a waste slope as part of the design process must assess the likely locations and shapes of potential shear surfaces and all possibilities must be checked in order to find the critical surfaces that control the design. It should be noted that different shear surfaces could be critical during the construction and post construction cases.

11.4.2 Stability: analysis methods

Limit equilibrium approach

As described in Section 11.3.3 above, slope stability is typically assessed using limit equilibrium methods. Unconfined slopes are normally considered in two-dimensions, however for confined slopes there are slip geometries (e.g. at the junction of two slopes) for which 3-dimensional analyses give lower factors and are considered to be a more appropriate approach. The amount of input information required to carry out a 3-dimensional analysis is significantly more that for 2-dimensional. Three-dimensional analyses are seldom carried out in the design phase but have been used to investigate failures (e.g. the back-analysis of Kettleman Hills carried out by Mitchell *et al.*, 1990).

A summary of the 3-dimensional stability method and a discussion of situations when is should be used are given by Bromhead *et al.* (2002). Circular shaped slip surfaces (i.e. part of a circle) form in homogeneous materials and non-circular slips form in non-homogeneous materials. There may be instances when the slip surface can be approximated to part of a circle but these will be rare and should be justified. As discussed in Section 11.2, limit equilibrium analyses can't be used to obtain information on strains in barrier components, and hence they can't be used to assess barrier integrity.

Method of slices

Waste, soil, geosynthetic surfaces and the interfaces between the materials are frictional. This means that the major part of their shear strength is related to the frictional properties, defined by angles of friction φ for soil and δ for interfaces. Shear strength (τ) is calculated using the Coulomb failure criteria where for soil:

$$\tau = c' + \sigma' \tan \phi' \qquad \text{Equation 7.2}$$

and for an interface (see chapter 7)

$\tau = \alpha' + \sigma' \tan \delta'$ Equation 7.3

Note that σ′ is the effective stress normal to the failure plane under consideration. The equation is written in terms of effective stress as it is the inter-particle stress that controls shear strength, not the total stress. Effective stress (σ′) = total stress (σ) – pore pressure (u), hence if the leachate pore pressure (u) increases the effective stress (σ′) will decrease, and therefore so will the shear strength (τ), assuming the total stress remains constant (i.e. bulk unit weight of waste does not change). The relationship between shear strength and effective stress means that the shear strength will vary along any failure surface because the effective stress will vary with depth of overburden.

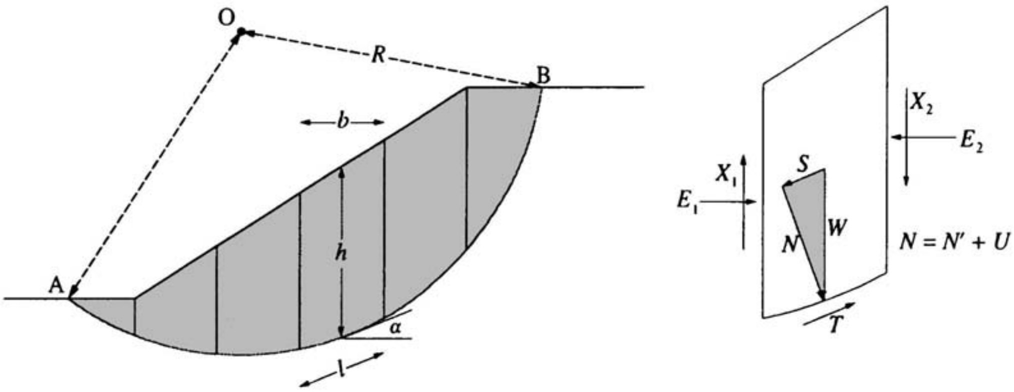


Figure 11.13 Stability analysis: method of slices (Barnes 2000)

Figure 11.13 shows a cross-section through a slope including a potential 'circular' failure plane. Also shown is a vertical slice through the slide body and the forces acting on the slice from the surrounding material for equilibrium. Most methods divide the slide mass into slices to aid solution of the problem. The direct solution of the forces acting on a slice is not possible because the problem is statically indeterminate. This means that assumptions about the relationship between forces acting on the slice have to be made in order to solve for the factor of safety. This has resulted in the development of a large number of analysis methods (i.e. based on different assumptions). However, they all use essentially the same approach (i.e. they use the method of slices).

Analysis of circular shaped slip surfaces is significantly easier and quicker than non-circular surfaces due to the greater number of unknown forces in the latter (resulting from internal shearing of the slide mass). It is not appropriate to cover in detail the methods of analysis in this document. However, the simplest equation for assessing the stability of a circular slip surface in a homogeneous material (Fellenius' method) is shown below. The method is simple due to the assumptions made regarding the inter-slice forces (i.e. they are assumed to be equal and opposite), but this also causes the calculated factors of safety to be conservative (i.e. low).

$$FofS = \frac{\sum c' l + \tan\phi' \sum(\gamma hb \cos\alpha - ul)}{\gamma hb \sin\alpha}$$ Equation 11.10

where:

W is the weight of the slice and is calculated as (γ_{bulk} x h x b), N′ is the effective normal component (reaction) of W, T is the shearing force at the base of the slice (i.e. tangential component of W), E_1 and E_2 are the normal inter-slice forces, X_1 and X_2 are tangential inter-slice forces, L is the length of the slice base and α is the average slope of the base of the slice to the horizontal (see Figure 11.13).

The basic methods are covered in standard soil mechanics text books (e.g. Barnes 2000, Craig 1999). A more detailed and rigorous review of the theory, and all aspects of slope stability analysis, is provided by Bromhead (1992). Stability analyses should be in terms of effective stress. The use of a total stress analysis (i.e. using undrained shear strength parameters) requires justification on a case specific basis.

Circular stability methods include those by Fellenius, Bishop (the most commonly used) and Janbu. Common non-circular methods are those by Morgenstern and Price, Spencer, Janbu and Sarma. While it is possible, although time consuming, to carry our analysis of circular slip surfaces by hand, analysis of non-circular surfaces requires a computer. All geotechnical design teams will have access to a computer based slope stability program. Users of such programs are often experienced in the analysis of circular slips but less experienced in analysing non-circular failure surfaces. As discussed in Section 11.3.1 above, circular surfaces are seldom appropriate in the study of landfills. There is a danger that with the large number of programs available and their ease of use (in terms of the ability to input data and generate results) that incorrect and inappropriate analyses can be carried out. It is important that those experienced in stability analyses, not those who are just able to operate the program, carry out the analyses.

A particular danger is the reliance on the use of automatic slip surface search routines. While these can reduce the number of analyses conducted to find the critical shear surface and therefore save time, there is a danger that they might miss the critical surface. There are many possible controls on the location and shape of the slip surface and the designer should assess these systematically. Information on stability calculations made available for checking should include: input parameters (design values and ranges, pore pressure conditions, details of the search for the critical shear surface and a summary of the calculated factors of safety referenced to the specific analysis details.

An initial assessment of the stability of a slope can be carried out using stability charts. These are based on circular slip surface and therefore should be used with care. A number of methods using stability coefficients (e.g. Barnes 2000) have been developed which give the critical (minimum) factor of safety for the long-term effective stress stability of a homogeneous slope. These methods are based on using standard tables or charts to obtain stability coefficients for a given slope and soil type, which when combined with the long-term pore pressure conditions enable the minimum factor of safety to be calculated. The methods provide a relatively quick and simple way of obtaining the approximate factor of safety for a slope. However, such simple analyses, even if appropriate, should not replace the rigorous analytical methods.

Variability of input parameters

The main input parameters are: waste slope profile, sub-grade ground profile, barrier location and type, sub-grade strata boundary levels, groundwater levels (leachate and sub-grade), unit weights of materials, additional loads (e.g. from equipment and vibrations), shear surface location and shape, and shear strengths of materials and interfaces. This section provides guidance on the selection of unit weight, water levels and shear strength values. The exact existing, or likely, value of a parameter is seldom known. In most cases a range of possible values will exist. Therefore, it is necessary to carry out analyses with relevant characteristic values and also to undertake a sensitivity analysis (sometimes called a parametric study) to assess the implication of the values being less conservative than the characteristic values. Combinations of the key parameters should also be considered in order to demonstrate stability under the worst possible conditions. The selection of characteristic values for interface shear strength is discussed in Chapter 7. The concept is the same for all parameters The characteristic value should be a conservative estimate over the volume or area controlling the mechanism being considered.

Information on unit weight of waste is provided in Section 8.2 and the importance of unit weight on landfill stability is highlighted above. Values used in stability analyses must be consistent with the case being assessed. Assessment of waste slope stability during construction is considering the stability of fresh waste with relatively low unit weights. The low unit weights are partly due to low degrees of saturation. However, an assessment of a slope in older waste will require the use of higher unit weights to reflect the denser wetter state of the material. The amount of soil in the landfill, either inert waste or cover soil material, also has an important influence on unit weight. Values should be selected on a site and waste specific basis. If there is uncertainty about the value to use then the higher estimated or measured values should be selected, as these will give lower factors of safety (i.e. high values are usually conservative).

Section 11.4.1 provides information showing the importance of using relevant leachate levels in stability analyses. However, on a site-specific basis there is rarely sufficient information on the actual (existing landfill) or likely (new landfill) leachate pressure distributions in the waste. Engineering judgement based on the possible scenarios discussed in Section 11.4.1 should be used to derive possible worst-case leachate levels (i.e. the highest) that might act during the design life of the slope. Consideration should be given to the proposed operation of the site (i.e. the possible use of leachate re-circulation) and if applicable, restrictions should be placed on certain activities in order to safeguard stability. The consequences of failure of the leachate control system should also be considered.

Shear strength of waste is discussed in Section 8.4. Effective stress shear strength parameters c' and ϕ' are required. Given the present uncertainty on the measurement and interpretation of shear strength data, and the changing composition of waste, it is recommended that a range of values be selected and a thorough sensitivity analysis conducted. The measurement and selection of shear strength parameters for interfaces between soils and geosynthetics and between geosynthetics and geosynthetics are discussed in detail in Chapter 7. Shear strength parameters for the mineral layers in the lining system and the sub-grade materials should be obtained using standard soil mechanics sampling and testing procedures. Effective stress parameters should be measured unless the use of total stress parameters can be justified. A key issue in the selection of strength parameters is the mechanism of progressive failure resulting from the strain incompatibility of materials (e.g. between the waste and cohesive soils). A decision must be made whether to use peak or residual shear strength parameters (or values in between). This is a complex issue and is discussed further in Section 11.4.5.

11.4.3 Integrity: factors controlling failure

All the issues described above in sections 11.4.1 and 11.4.2 are important. In addition, assessment of the long-term integrity of lining systems requires an understanding of the interaction between the lining system and waste and quantification of the strains in the lining components. Waste settlement can lead to a loss of protection to the geomembrane through tensile failure of geotextile layers and loss of protection layers.

11.4.4 Integrity: analysis methods

Introduction

Numerical analysis techniques can be used to assess the integrity of both mineral and geosynthetic lining systems. There are many computer software programs available for the analysis of geotechnical problems, however there are two main features necessary to enable landfill liners to be assessed. Firstly, the model needs to be able to accommodate large deformations due to waste settlements; finite difference codes are inherently more suited to this than finite element codes. Secondly, the computer code needs to be able to model interfaces and, in particular, strain softening interfaces. Although there are a number of computer codes available, an example of the use of FLAC (Fast Lagrangian Analysis of Continua) is given in this section.

An example of the assessment of the integrity of a shallow slope lining system post waste placement is given in Jones & Dixon (2002). In this example the finite difference code FLAC was used to model the effect of waste settlement on a geosynthetic lining system. Numerical analyses were carried out for the typical landfill cross section geometry shown in Figure 11.14 using the baseline material properties given in Table 11.2. The effect of variations in waste properties, side slope gradient and landfill height on the mobilised shear stresses were subsequently investigated. The interface was assumed to be in a drained condition with zero pore water pressures. There will be no excess pore water pressures at the geosynthetic interface since settlement rates are slow (i.e. taking typically 30 years plus), the waste body is unlikely to be saturated and there is usually a drainage layer (sand, gravel or a drainage geocomposite) above the lining system (i.e. drainage path lengths are short).

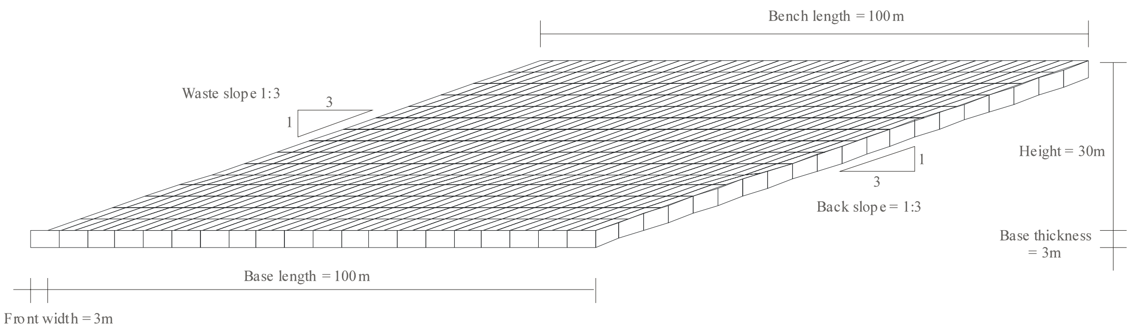


Figure 11.14 FLAC mesh layout and geometry for a cross-section through a typical liner and waste configuration

Table 11.2 Input variables for baseline conditions

Material	Property	Value
Geometry	Mesh size Waste slope Side slope Height	(40, 17) 1 in 3 1 in 3 30 m
Subgrade (material beneath interfaces)	Young's Modulus Poisson's Ratio Density	50 MPa 0.3 1900 kg/m^3
Waste	Young's Modulus Poisson's Ratio Density Friction angle Cohesion	500 kPa 0.3 1223 kg/m^3 25° 5 kPa
Base and side slope interfaces	Interface shear stiffness Interface normal stiffness Peak friction angle Peak cohesion intercept Residual friction angle Residual cohesion intercept	3 MPa 30 MPa 24.5° 3.2 kPa 12.8° 2.5 kPa

Results for baseline conditions

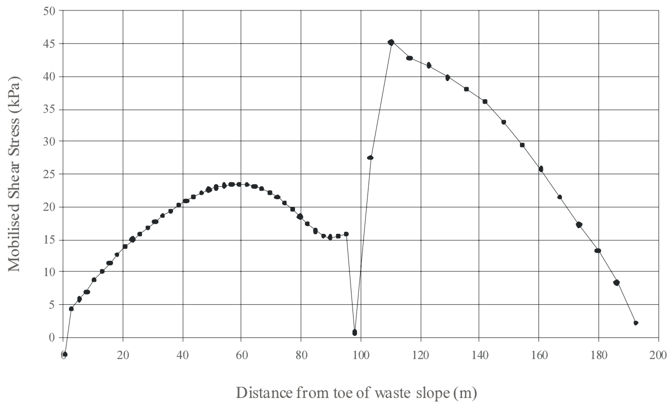
In the assessment of the results of this analysis, the mobilised shear stresses, interface displacement and mobilised friction angle were plotted against the distance from the toe of the waste slope. For the geometry shown in Figure 11.14 the first 100 m from the toe of the waste slope comprises the base of the landfill, with the remaining 95 m being the distance up the 1:3 landfill side slope.

The mobilised shear stress (Figure 11.15a) increased to an initial peak at a distance of 60 m from the toe of the waste slope which approximately corresponded to the mid point in the outer waste slope. The shear stress then reduces towards the toe of the landfill side slope with an upturn in shear stress at around 10 m from the toe of the side slope.

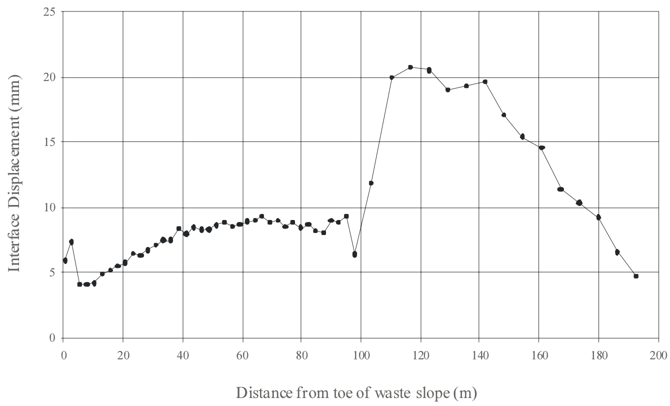
A second peak and maximum mobilised shear stress was generated at a distance of around 10 m up from the toe of the side slope; the shear stress then reduces up the remainder of the landfill side slope. The maximum shear stress was not mobilised at the toe of the side slope due to the geometry at the corner, i.e. the waste "zones" within the mesh must be allowed to compress during the stress redistribution. Since the corner is surrounded by sub-grade zones that were fixed, movement is limited near the corner and the largest stresses were therefore mobilised adjacent to the corner.

The displacements and friction angles mobilised along the interface are shown on Figure 11.15b and 11.15c respectively. The interface displacements follow a similar pattern to the mobilised shear stress, with the largest displacements mobilised on the side slope next to the corner. The friction angles mobilised are consistent with these displacements. The lower displacements mobilised along the base interface of between 4 mm and 9 mm result in friction angles between around 21° and 23°, while the higher interface displacements towards to the base of the side slope gives post peak friction angles of between 17° and 19°.

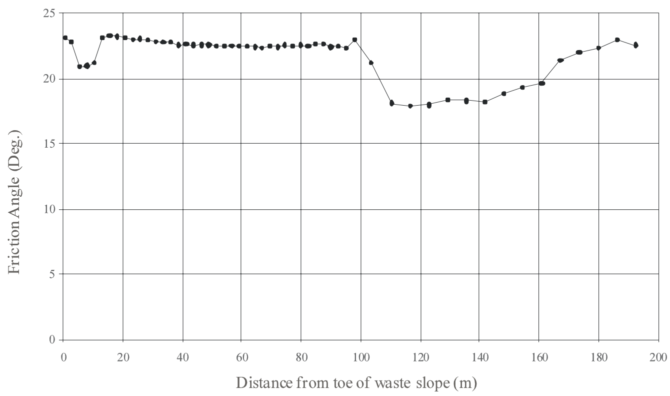
It should be noted that the friction angles (and adhesions) are used to calculate the envelope (i.e. peak allowable shear stress) for a particular interface displacement and that the actual mobilised interface shear stresses are generated using the overall stress redistribution. Consequently, the shear stresses plotted in Figure 11.15a are less than would be calculated using the friction angle from Figure 11.15c and the appropriate adhesion intercept, i.e. the mobilised shear stresses are in many instances less than the shear strength.



(a) Mobilised shear stress



(b) Mobilised interface displacement



(c) Mobilised interface friction angle

Figure 11.15 Results of numerical analysis of the conditions shown in Figure 11.14

Effect of waste stiffness and Poisson's ratio

The importance of Young's modulus and Poisson's ratio have been investigated with a series of runs in which Young's modulus was decreased from 500 kPa to 250 kPa and increased to 1000 kPa while keeping a Poisson's ratio of 0.3. The maximum shear stresses mobilised along the base and side slope interfaces are given in Figure 11.16. The shear stresses are proportional to the stiffness values.

Calculated increases of shear stress at the geomembrane/geotextile interface with increasing waste stiffness are due to the large interface displacements that are mobilised. The effect of changing the Poisson's ratio of the waste on the mobilised interface shear stress was also investigated, however the $\nu = 0.1$ and $\nu = 0.3$ runs gave very similar shear stress distributions. Higher values of ν (i.e. representing behaviour closer to undrained condition, $\nu = 0.5$) were not studied, as only long-term drained conditions were considered. The distributions of mobilised shear stress along the interface were not sensitive for the range of waste stiffness and Poisson's ratio considered appropriate.

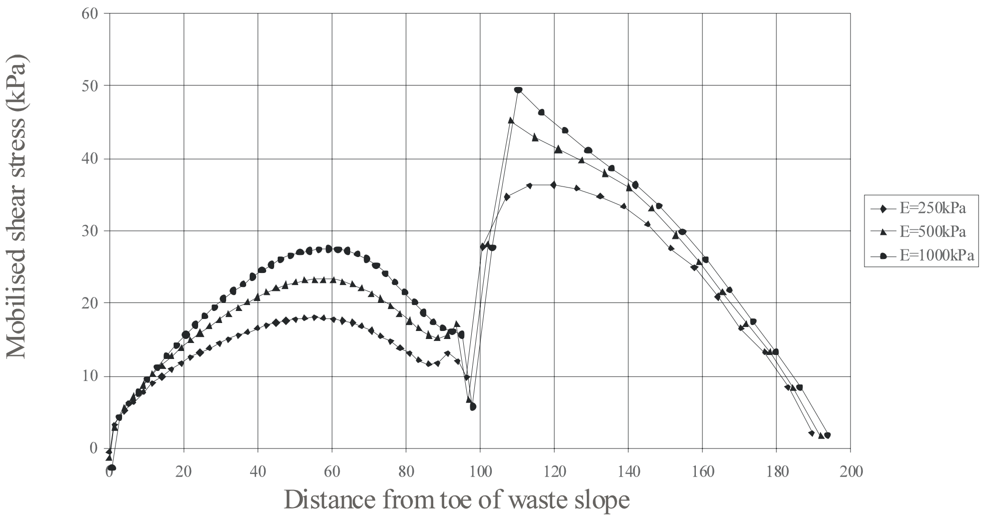


Figure 11.16 Mobilised shear stresses for various waste Young's Moduli

Effect of waste shear strength and unit weight

To investigate the importance of waste shear strength on stability of a geosynthetic lining system the shear strength parameters shown in Table 11.3 were used. The design, mean and maximum values mobilise similar shear stresses along both the side slope and base interfaces, with comparable distributions to those shown in Figure 11.15. The waste shear strength did not have a large effect on the mobilised shear stresses, provided that the strength is greater than the waste slope angle. If it is not (i.e. in the case of the minimum values specified), the waste slope will deform and there will be an increase in shear stresses mobilised along the base, and a decrease in side slope stresses.

Table 11.3 Waste shear strength values (after Jones *et al.* 1997)

Friction angle, ϕ' (°)	Cohesion intercept, c′ (kPa)	Comments
25	5	Suggested design values
42	19	Maximum envelope
15	10	Minimum envelope
31	10	Numerical mean values

The effect of waste unit weight on the stability of a geosynthetic lining system was investigated using unit weights of 10 kN/m^3, 12 kN/m^3 and 14 kN/m^3. Increasing the unit weight of the waste from 10 kN/m^3 to 14 kN/m^3 increased the shear stresses mobilised along both the side slope and base interfaces. The general distribution of mobilised shear stress was again similar to those shown in Figure 11.15a. It should be noted, however, that these analyses have been carried out using waste stiffness values that achieved a settlement of around 20%. It is likely that higher waste unit weights on site would be due to less biodegradable material being present and would probably lead to higher stiffness and less settlement.

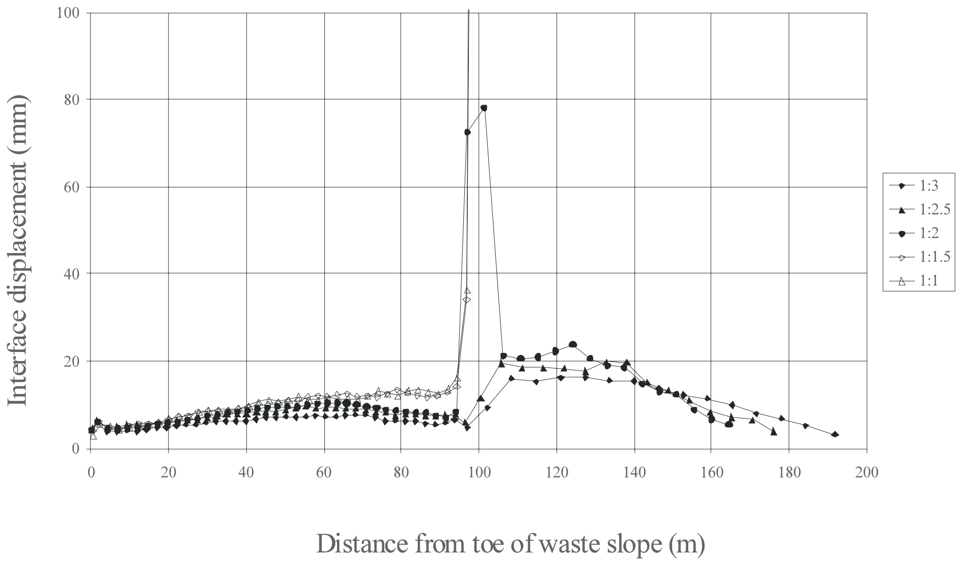
Effect of landfill side slope gradient

It is unusual for a landfill side slope to be shallower than 1 in 3 and there is often pressure to increase the gradient to give a larger landfill void space for waste placement. The importance of the side slope gradient on overall stability was investigated by considering side slopes of 1 in 3, 1 in 2.5, 1 in 2, 1 in 1.5 and 1 in 1. In all cases, the material beneath the geosynthetics (i.e. the mineral liner and sub-grade) was considered to be inherently stable.

The distribution of shear stress along the base of the landfill was very similar for all five runs for distances up to 50 m from the toe of the waste slope (i.e. the mid point of the base). The magnitude of the shear stresses mobilised along the side slope interface decreased as the side slope gradient increases. To explain this, the interface displacements need to be considered.

Figure 11.17 shows the interface displacement distribution for the four runs. All side slope gradients mobilised a similar interface displacements along the base of the landfill. However, the 1 in 1.5 and 1 in 1 slopes both resulted in maximum interface displacements greater than 2.5 m along the side slope, and with the 1 in 2 slope giving a peak of around 80 mm. A smaller scale plot of interface displacement against distance from the toe of the waste slope is presented in Figure 11.17b.

a. Interface displacement - up to 100 mm



b. Interface displacement - full scale

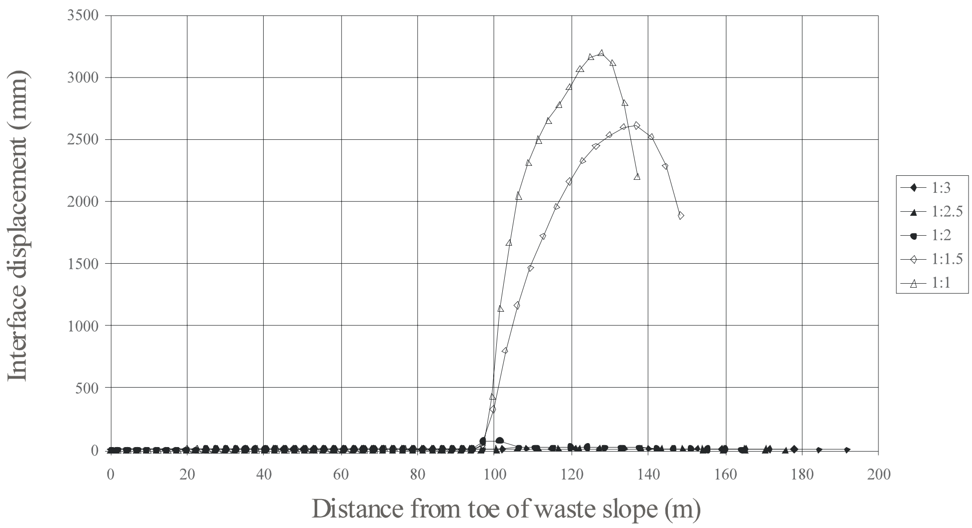


Figure 11.17 Mobilised interface displacements for various side slope gradients

These large displacements along the side slope result in the mobilisation of residual friction angles and adhesion intercepts, e.g. Figure 11.18. Both the 1 in 1.5 and 1 in 1 slopes have residual conditions mobilised along the side slope with the 1 in 2 slope mobilising residual conditions at the toe of the side slope with post peak values along the remainder of the side slope. Since the interface modelled in this exercise is a textured geomembrane/geotextile interface, it is likely that displacements of the order of metres would result in tensile failure of the geotextile above the geomembrane, thus causing a failure of the protection for the geomembrane. Displacements of this magnitude along similar interfaces have been measured in field trials by Gourc *et al.* (2000).

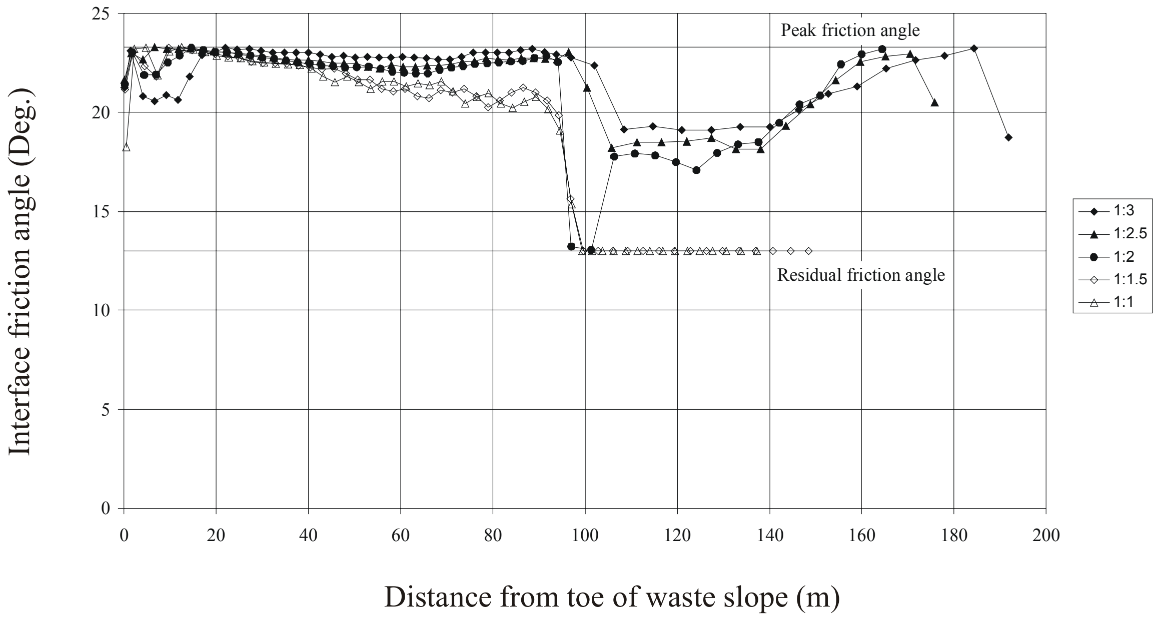
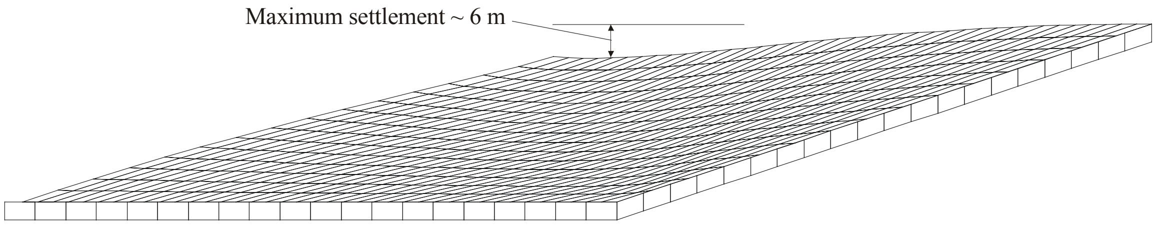


Figure 11.18 Mobilised interface friction angles for various side slope gradients

Large interface displacements of the deformed mesh are shown in Figure 11.19. The deformed mesh for a 1 in 3 side slope shows deformation within the waste body leading to a surface settlement in the region of 6 m (Figure 11.19a). This is achieved with no visible movement at the top of the side slope interface. Figure 11.19b however, shows the deformed mesh for the 1 in 1 side slope which gives surface settlements of the same order as the 1 in 3 side slope run, however in this instance the displacement at the interface is clearly visible.

a. Deformed mesh for 1 in 3 side slope



b. Deformed mesh for 1 in 1 side slope

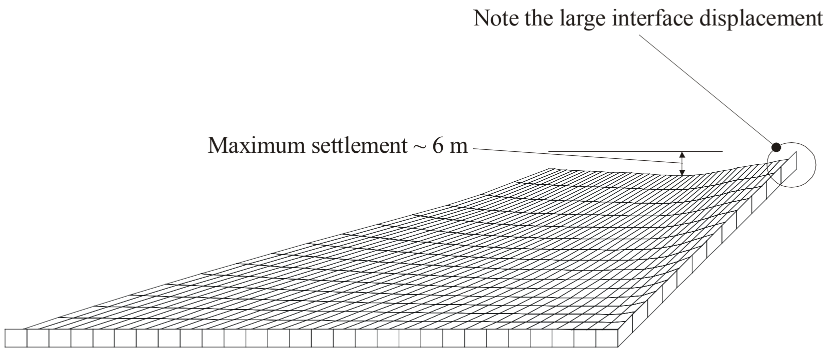


Figure 11.19 Deformed mesh

Effect of landfill height

The effect of waste height on the stability of a landfill lining system was investigated for waste heights of between 10 m and 60 m. The waste slope and side slope were both maintained at 1 in 3, and a base length of 100 m was used. The distributions of mobilised shear stresses for the various waste heights are given in Figure 11.20. As the slope height increases, the mobilised shear stress increases proportionately. It should be noted that the shear strength along this interface also increases proportionately. The distribution of mobilised shear stresses along the base is also dependent on landfill height, however the development of these stresses is controlled by the overall geometry.

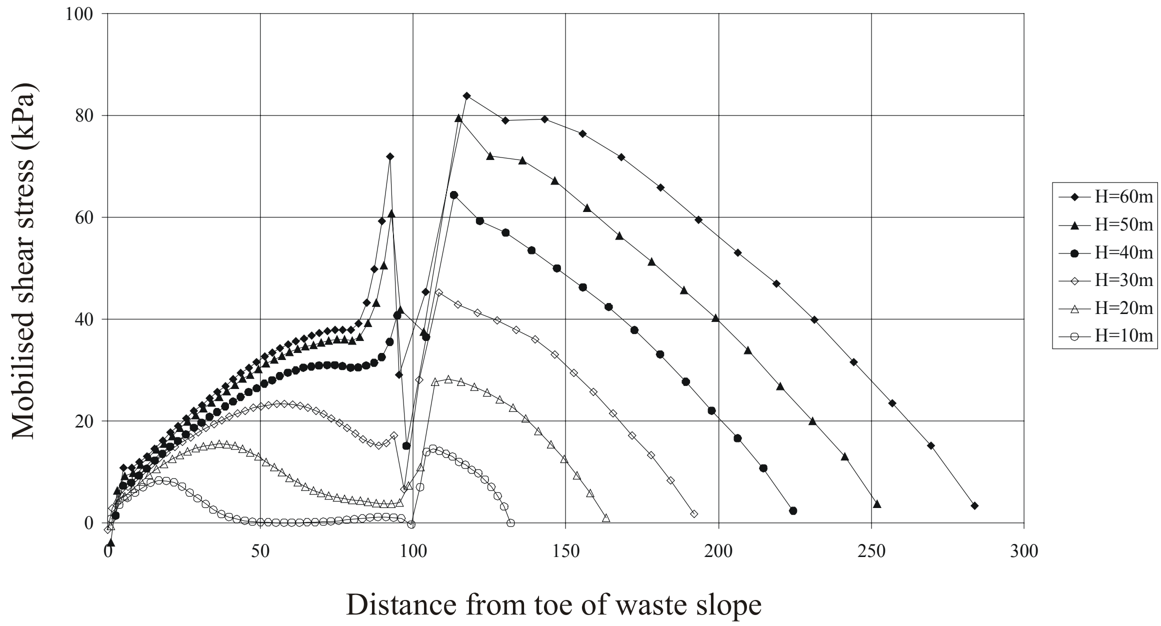


Figure 11.20 Influence of waste height on mobilised shear stress

11.4.5 Comparison of numerical techniques with limit equilibrium

In order to investigate the relative stability of the geometries analysed, partial factors can be applied to the textured geomembrane/geotextile interface shear strength parameters. Partial factors of between 1.2 and 1.8 were applied in the usual manner:

$c'_{field} = c'_{lab}/\gamma$

$\tan\phi'_{field} = \tan\phi'_{lab}/\gamma$

where γ is the partial factor.

As an example, the effect of applying partial factors on the analysis of a 30 m high 1 in 2 slope is shown in Figure 11.21. The mobilised interface shear stress along the base of the landfill increases as the partial factors increase, whilst the partial factors reduce the shear stress along the side slope interfaces. Along the base, the interface displacements are small (Figure 11.21b). However, on the side slope, displacements are significantly higher (3.0 metres plus) than for the case without a factor, and indicate that the ‘pseudo’ factor of safety against local failure is between 1.0 and 1.2. The results of the numerical analysis can be compared with those from limit equilibrium analysis by using these ‘pseudo’ factors of safety.

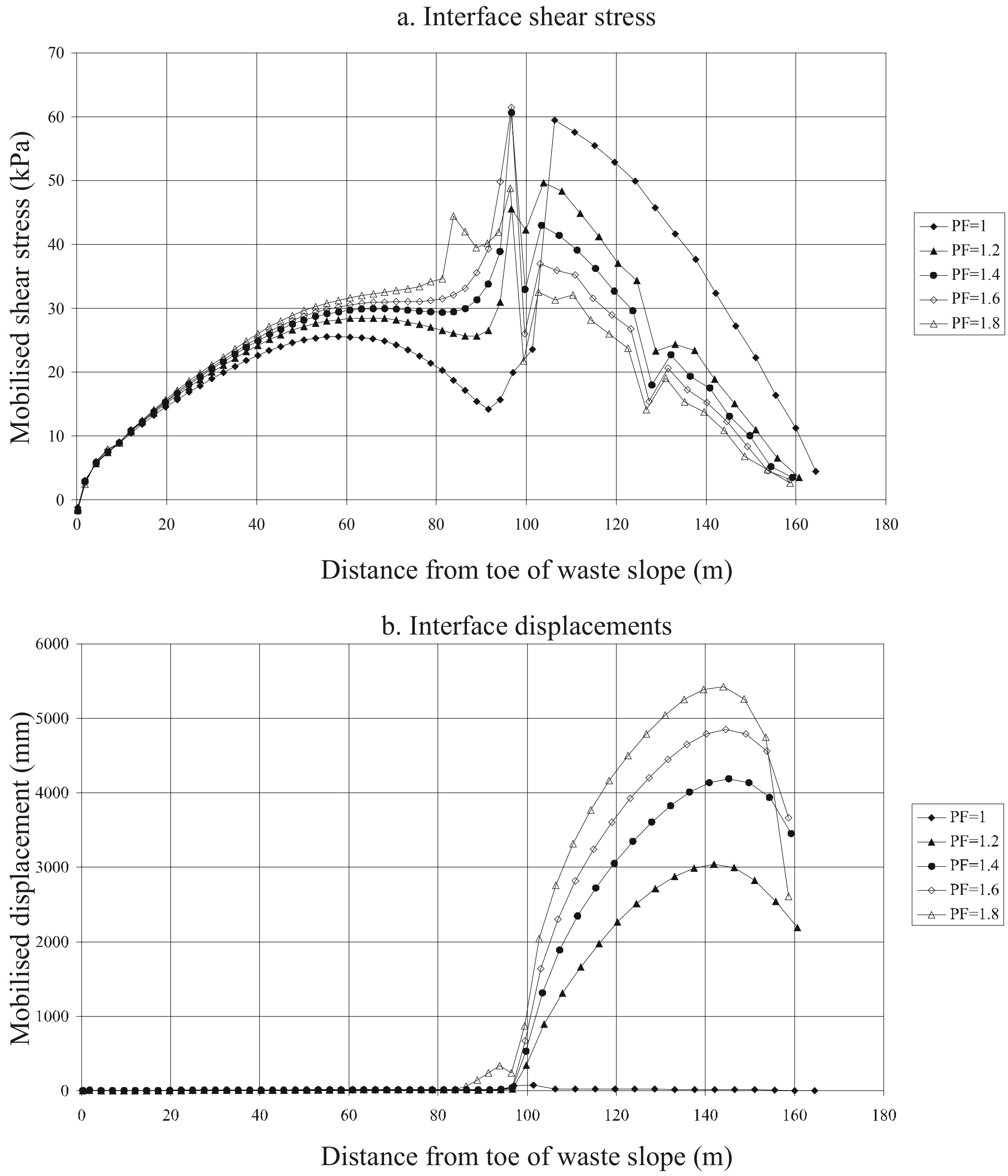


Figure 11.21 Influence of applying partial factors

Limit equilibrium analysis

Since numerical analysis may not be available to the landfill design engineer, the use of conventional limit equilibrium analysis needs to be considered. It has been reported by several authors, e.g. Byrne (1994), Gilbert *et al.* (1996) that limit equilibrium techniques cannot be used to determine the stability of strain softening material. The main issue identified by the authors is that it may be unsafe to assume that peak strength is available along the entire slip surface and conversely it maybe excessively conservative to assume that only residual strength is available.

The results of numerical analyses presented above enable an assessment of the shear strengths mobilised at the base and side slope interfaces. Limit equilibrium analysis can be used to assess the stability of landfill side slopes using peak, residual and mobilised interface shear strengths calculated from the numerical analysis. The textured geomembrane/geotextile interface is again considered.

Approach

The limit equilibrium slope stability program XSTABL (Sharma, 1991) has been used to assess the stability of various landfill geometries. Details of the material parameters and grid geometries used are given in Figure 11.22. The stability analysis was carried out by entering a single non-circular slip plane which corresponded to the centre of the interface on the side slope and falls at a gradient of 1:100 towards to the toe of the waste slope. The modified Janbu method (Janbu, 1973) was used to calculate the factor of safety against failure, using sets of interface shear strength parameters for the base and side slope interfaces (see Table 11.4). The range of geometries investigated are summarised in Table 11.5.

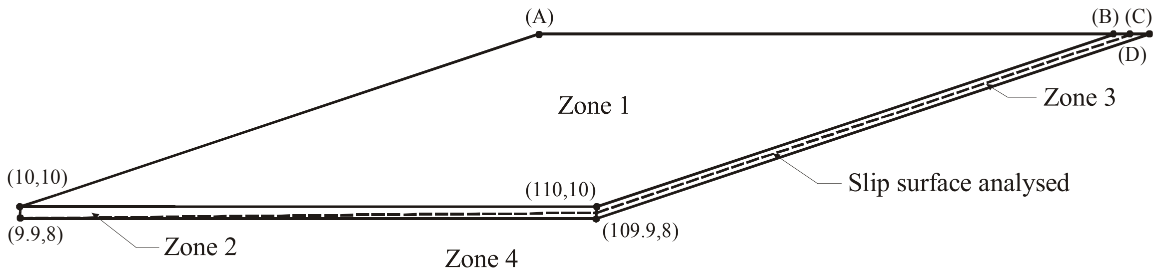


Figure 11.22 Schematic cross-section of the limiting equilibrium analysis

Table 11.4 Material properties used in limit equilibrium analysis

Zone	Parameters
Zone 1 (Waste)	Unit weight = 12 kN/m^3 Friction angle = 25° Cohesion = 5 kPa
Zone 2 (Base interface)	Unit weight = 18 kN/m^3 Friction angle, varies Adhesion, varies
Zone 3 (Slope interface)	Unit weight = 18 kN/m^3 Friction angle, varies Adhesion, varies
Zone 4 (Sub-stratum)	Unit weight = 18 kN/m^3 Friction angle = 0° Cohesion = 500 kPa

Table 11.5 Geometries used for limit equilibrium analysis

Case	Side slope gradient	Waste height (m)
Case 1	1 : 3	30
Case 2	1 : 3	60
Case 3	1 : 2	30
Case 4	1 : 1	30

Four combinations of interface shear strength parameters were used as follows:

- peak strength on both the base and side slope;
- residual strength on both the base and side slope;
- peak strength on base and residual strength on side slope; and
- values on the base and side slope based on mobilised shear strengths from the numerical analysis.

Mobilised shear stresses

From the results of the numerical analysis, the strength conditions along the textured geomembrane vs. geotextile interface can be investigated. The mobilisation of interface shear strength have been divided into the following four stages:

- pre-peak condition;
- at peak and post-peak up to 50% strength reduction;
- post-peak and greater than 50% strength reduction; and
- residual condition, i.e. no further strength reduction.

A summary of the effect of landfill geometry on the development of these four stages is given in Table 11.6 . The cases used in the limit equilibrium analyses are highlighted. Mean values of mobilised shear strengths can be calculated for both the base and side slope based on weighted averages from the four stages. This is achieved by calculating the peak, residual and 50% reduction friction angles and adhesion intercepts, and multiplying the values obtained by the percentage of the interface length with those conditions (i.e. using Table 11.6).

Table 11.6 Summary of effect of geometry on shear strength condition along geosynthetic interface

Side slope gradient	Landfill height (m)	Strength condition along geosynthetic interface							
		Base				Side Slope			
		Pre-peak	Post-peak	50% Reduction	Residual	Pre-peak	Post-peak	50% Reduction	Residual
1:3	10	100%	0	0	0	66.7%	33.3%	0	0
	20	79.5%	20.5%	0	0	19.7%	80.3%	0	0
	30	27.4%	72.6%	0	0	15.5%	84.5%	0	0
	40	14.2%	85.8%	0	0	8.6%	59.1%	32.3%	0
	50	0	97.4%	2.6%	0	8.4%	39.8%	51.8%	0
	60	0	94.7%	5.3%	0	8.3%	33.3%	58.4%	0
1:2.5	30	21.3%	78.7%	0	0	6.7%	93.3%	0	0
1:2	10	100%	0	0	0	34.4%	65.6%	0	0
	20	77.0%	23.0%	0	0	6.6%	93.4%	0	0
	30	21.3%	78.7%	0	0	6.6%	47.4%	46.0%	0
	40	5.5%	89.3%	5.2%	0	0	0	0	100%
	50	0	63.7%	36.3%	0	0	0	0	100%
	60	0	46.5%	53.5%	0	0	0	0	100%
1:1.5	30	19.0%	78.4%	2.6%	0	0	0	0	100%
1:1	30	19.2%	78.2%	2.6%	0	0	0	0	100%

Results of limit equilibrium analysis

The factors of safety against a failure plane developing along the full length of the interface for each of the four cases, calculated using limit equilibrium (LE) analysis, are given in Table 11.7.

Table 11.7 Calculated factors of safety using limit equilibrium analysis

	Calculated Factor of Safety			
Geometry	**Base Peak Side Peak**	**Base Residual Side Residual**	**Base Peak Side Residual**	**Mobilised values**
Case 1	3.3	1.7	2.5	2.9
Case 2	2.1	1.0	1.3	1.6
Case 3	2.9	1.5	2.4	2.4
Case 4	2.6	1.3	2.3	2.1

All factors of safety calculated using peak interface shear strengths resulted in values significantly in excess of unity. Factors of safety ranging from 1.0 to 1.7 are calculated using residual strengths on both base and side slope with the lowest value corresponding to the 60 m high geometry (Case 2). However, the basal interface is unlikely to have full residual conditions along its length and so it may be more appropriate to use peak values on the base and residual values on the side slope. This approach gives calculated factors of safety ranging from 1.3 to 2.5 which corresponds better with the values calculated from the mobilised shear strengths from the numerical analysis which range from 1.6 to 2.9.

At first the concept of using peak shear strengths along the base and residual strengths along the side slope seems to take account of waste settlement along the slope and should give a conservative estimate of stability. However, the calculated factors of safety are still up to 0.4 different from the results using the mobilised values. It would be dangerous to use this approach since while Cases 1 and 2 seem to underestimate the stability, Case 4 overestimates the stability.

From the numerical analysis Cases 1, 2 and 3 gave stable configurations with no excessive displacements. Case 4 however, gives maximum displacements along the side interface of around 3.2 m and is clearly the most unstable geometry. This instability does not manifest itself at the toe of the waste slope and is not therefore an overall slope failure. It can be considered, however, as a local failure of the side slope lining system since such excessive movement could cause tensile failure of the geosynthetics. The mechanism of this failure is related to large compression of the waste close to the base of the slope. This failure mechanism is not predicted by the LE analysis since there is no continuous failure plane throughout the whole geometry.

Discussion

The comparison between limit equilibrium stability analysis and numerical analysis, has established that limit equilibrium analysis cannot be used to establish the stability of the side slope lining system during the lifetime of the landfill (i.e. integrity). However, it is interesting to note that while the numerical analysis does not give a factor of safety, partial factors can be applied to the interface shear strength parameters that can give an indication of how close the system is to instability. This has been carried out for Cases 1 to 4 to compare the LE factors of safety with the 'pseudo' factors of safety that can be estimated from the numerical analysis (i.e. the factors that result in significant displacements along the interface). In effect, this is investigating the stability of the geosynthetics on the slope. A comparison between these factors of safety is presented in Table 11.8.

Table 11.8 Summary of factors of safety from limit equilibrium and numerical analysis

Geometry	Limit Equilibrium Factor of Safety	Numerical Analysis Condition	Pseudo Factor of Safety
Case 1	2.9	Stable	1.65
Case 2	1.6	Stable	1.15
Case 3	2.4	Stable	1.05
Case 4	2.1	Local instability	<1.0

For Case 1 the application of partial factors up to 1.6 have no effect on the mobilised shear stresses and interface displacements. However, increasing the partial factor to 1.7 results in displacements on the side slope interface of 3.5 m which can be taken as failure. This suggests that a pseudo factor of safety (with respect to the interface shear strength) for Case 1 is between 1.6 and 1.7, say 1.65. If a similar approach is used for Cases 2 and 3 then failure is achieved on the side slope for partial factors of 1.2 and 1.1 respectively. This suggests that Cases 2 and 3 are only marginally stable and not as stable as the limit equilibrium analysis would suggest.

This demonstrates that the limit equilibrium analysis severely underestimates the stability of a lining system that incorporates a textured geomembrane/geotextile interface, and it can not model local failure conditions. These can result in significant relative displacements (i.e. in excess of 3 metres) between geosynthetic elements.

However, Jones (1999) shows that for certain geosynthetic interfaces (e.g. smooth geomembrane/geotextile) and landfill geometries, a continuous failure plane can develop and in such instances both limiting equilibrium and numerical analysis can predict global failure.

11.5 Summary and Key Issues

Stability assessment of shallow side slopes can be divided into two sections. Veneer (and capping systems) and lining systems post waste placement. Design must ensure the stability of the lining system during construction (pre-waste placement) and following waste placement. The integrity of the lining components must also be safeguarded in both the short-term and in the long-term, post waste degradation. Assessment of veneer systems must consider the influence of water pressures (including seepage pressures), plant loads and in certain circumstances also gas pressures (e.g. capping systems and liners over waste). Analysis methods that allow stresses in the lining components to be calculated should be used.

Stability of landfills post waste placement is often controlled by weak layers and interfaces beneath and within the waste. Failure surfaces are usually non-circular in shape, and hence appropriate analysis techniques should be used. All potential failure surfaces must be assessed. The important role that leachate pressures play in stability must be considered, although pressure distributions can be complex. Given the heterogeneous nature of waste, a range of possible properties should be used in analyses. Failures do occur post waste placement. They are as likely to occur during construction as at full waste height. Stability assessment must be carried out for all stages of construction, in addition to the finished profile.

Integrity of lining systems post-waste placement cannot be assessed using limit equilibrium techniques. Numerical modelling can be used to assess local instability (i.e. localised large deformations on interfaces). An approach has been outlined using 'pseudo' factors of safety obtained from a numerical analysis to assess the likelihood of local instability. This indicates that limit equilibrium analysis can give unconservative results.

12. STEEP SLOPE LINING SYSTEMS

12.1 Introduction

The waste disposal industry in the UK has relied for many years on the infilling of worked mineral voids as the primary means of disposal. These sites also have to be designed on a containment basis to prevent the migration of leachate and uncontrolled escape of landfill gas. The barriers used are typically either low permeability clay, geomembrane or various combinations. The lining of shallow sided quarries such as clay, sand and gravel extraction pits for landfill can provide the designer with challenges, notably in side slope design (see Chapter 11). Such challenges, however, are magnified when considering steep sided quarries. In the context of this report, steep is considered to be slopes in excess of 30°. The use of hard rock quarries for waste disposal is becoming increasingly popular due to the huge potential void space. The engineering design of suitable lining systems for steep sided quarries needs to be carefully considered. This section provides an introduction to the lining systems presently used in the UK. Issues of stability and integrity are discussed and analysis methods summarised.

It is necessary to develop barriers that are effective, can be readily constructed and are affordable. The barrier system needs to be able to prevent landfill gas migration off site through fissures in the bedrock, as well as deflect any perched leachate to the leachate collection and removal system at the base. Further, there must be a physical link between the basal and side lining system to ensure continuity of the barrier. A steep slope lining system can be constructed in two main ways; in lifts ahead of waste placement or built to full height in one lift. A full height barrier would need to be protected from the environment since clay would be prone to desiccation and geosynthetics can suffer from ultra violet attack.

There are two main materials that are typically used as barriers for steep sided landfills: natural clays and geosynthetics. These are currently used in isolation in most designs (i.e. not together in the same side slope lining system). However, introduction of the Landfill Directive will lead to the requirement for composite lining systems even for these steep slopes. Whatever type of lining system is chosen, its long-term stability and integrity must be assured. There are two approaches to the design of a steep slope lining system:

- the lining system is assumed to be self supporting and therefore could be constructed to the full height of the side slope and would be stable in the long-term without waste being present; and
- the barrier is constructed in lifts and is designed to be self-supporting for one or more lift heights, but requires the placement of waste to ensure stability of the lining system for subsequent lifts (i.e. the lining system is not stable at full, or possible partial height, unless waste is placed against it for support).

Fully self-supporting designs are seldom used in the UK due to their high costs. This is caused either by the complexity of construction or the loss of void space resulting from forming shallower side slopes. Therefore, for the majority of landfills in the UK, the structural integrity of mineral, geosynthetic or composite lining systems for steep side slopes is controlled by the interaction between the waste and barrier system. Presently, novel barrier systems for steep side slopes (e.g. reinforced earth, polystyrene face supports and buttressed clay barriers) are being constructed without a full understanding of the factors controlling either the short-term construction related, or long-term waste degradation controlled, deformations. In addition, despite the present designs relying on waste in part for their stability (i.e. a heterogeneous material whose engineering properties change during degradation, see Chapter 8), there are no published records of barrier systems having been instrumented and monitored in order to demonstrate satisfactory performance. While it is unlikely that these barrier systems will fail catastrophically with the barrier suffering deformations of several metres (i.e. because the waste provides some support), the low stiffness of MSW material will result in movement of the barrier into the waste until equilibrium conditions are established, and hence integrity is a major concern.

Numerical modelling of barrier configurations that rely on the presence of waste to provide lateral support has demonstrated, not surprisingly, that the waste properties control performance (e.g. Reddy *et al.* 1996). The following are areas of concern.

- Deformation of the lining system is controlled by in situ stress conditions and the stiffness of the as-placed waste. Of particular concern is the strain incompatibility between traditional mineral/geosynthetic lining materials and the waste, and specifically the large strains that are likely to occur for the lining system/waste body to reach equilibrium. There must be uncertainty regarding the integrity of the lining system in the short-term under these conditions.
- Degradation of the waste with time will alter its mechanical properties, and thus influence the long-term stability and hence potentially the integrity of the lining system.
- There is a dearth of information regarding the stresses in the barrier components resulting from the barrier/waste interaction.

To date there is only one comprehensive study of steep side slope lining system performance in the international literature. Edelmann *et al*. (1999) describe a field trial that was undertaken in Germany to investigate the interaction between a specific design of a steep side slope barrier system and waste (the barrier investigated was a compacted clay liner supported by a gabion wall installed at 80^{o}). They compared the observed behaviour of a large-scale laboratory model of the barrier with the performance of the actual barrier system obtained by in situ monitoring. Findings from this detailed study included:

- the barrier experiences 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 are shown diagrammatically in Figure 12.1.

Edelmann *et al.* (1999) concluded that ultimate limit state and serviceability must be examined for each barrier design separately and must be checked by appropriate in situ measurement. A project to investigate barrier/waste interaction is in progress at Loughborough University funded by the Engineering and Physical Sciences Research Council (EPSRC). This includes field monitoring of a steep slope lining system to assess the interaction with the waste and to provide data to validate a numerical model. Ng'ambi *et al.*

(2001) have published preliminary results from the monitoring and more detailed findings are expected in due course.

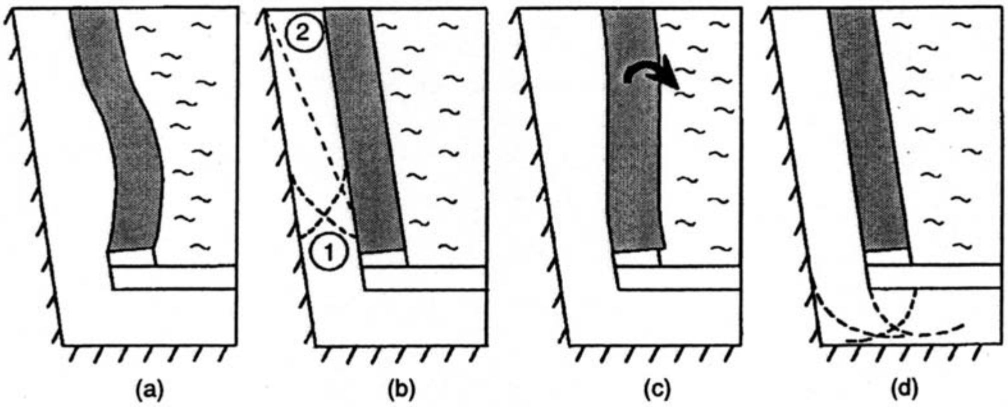


Figure 12.1 Possible failure modes in a steep slope mineral lining system: a) bulging, b) shear failure, c) toppling and d) bearing failure (after Edelmann *et al.* 1999)

12.2 General Design Issues

The economical lining of steep side slopes is a complex problem. There are many issues that need to be addressed in the design process including some that are still poorly understood. UK engineers have developed a range of lining systems as outlined in Section 12.3, and many have been built. However, there is little if any information on their long-term performance. This is in contrast to Germany where a comprehensive study was carried out by Edelmann *et al.* (1999). The following discussion aims to highlight the main issues regarding the lining of steep sided quarries.

12.2.1 Rock face

The quality of the quarry face will significantly affect the barrier design. Depending on the geology of the quarry, the rock face may be jagged and have overhangs that could damage geosynthetics and make clay placement difficult, or may have closely spaced discontinuities that provide an easy flow path for leachate and landfill gas. The face is likely to be irregular and, particularly in older quarries, may be dangerously unstable with considerable loose material. To overcome these problems, a combination of rock face stabilisation and geosynthetic protection would be required. The rock face should initially be cleaned of loose debris or even pre-split, and it may then be possible to stabilise the face using rock bolts, wire mesh and shotcrete. If the profile of the face is particularly difficult then thought must be given to creating an artificial surface. This may be achieved using metal or textile gabion baskets or reinforced earth to create a steep sided wall with a relatively smooth surface, however its internal stability would have to be ensured. The geometry of the face has important implications for the choice of barrier system. For the clay barrier supported by engineered fill, a steeper rock face will give a more stable overall configuration. However, for a reinforced soil solution to be more effective, a shallower rock face would be required in order to maximise support from the waste.

12.2.2 Engineering properties of the waste

The choice of whether any reliance can be made on the waste supporting the lining system can only be made with knowledge of the engineering properties of the waste. A detailed discussion of the key engineering properties is provided in Chapter 8. While the standard parameters of unit weight, compressibility and shear strength are important, it is the in situ stress state within the as-placed waste and the lateral stiffness that have the greatest control on waste/barrier interaction.

12.2.3 Transfer of load from the waste through settlement

The waste will undergo large settlements both during filling and in the long-term due to degradation (see Section 8.3), and this movement can cause significant stresses in a lining system. One of the approaches outlined below should be used to ensure the barrier is not overstressed.

- completely separate the waste from the barrier system, however, this is difficult to achieve;
- allow for movement in an intermediate zone between the waste and the barrier, such as the engineered fill; and
- introduce a very weak layer or interface between the waste and the lining system and allow preferential movements to occur in this layer (e.g. the interface between a smooth geomembrane and a geotextile).

12.2.4 Protection of the barrier

The integrity of the barrier must be maintained throughout the life of the landfill. A geomembrane must be protected from damage from the waste, while a clay barrier must be prevented from desiccation and cracking. A sand protection layer, in conjunction with a geotextile or a geonet, could be used to protect a geomembrane but problems could occur that are associated with long-term differential settlement of the waste. Differential movement of the waste could induce the sand to move away from the lining system allowing waste to fall behind the protection layer, or lead to tensile failure of the geotextile. Consideration should be given to introducing a buffer layer of selected waste between the protection layer and the main body of the waste.

It is more difficult to prevent a clay barrier from drying out and cracking. The internal moisture content of the landfill is generally very high and this can help to maintain the clay's integrity, although there is no information on moisture content changes in mineral liners post waste placement.

The outside face of the clay, in contact with either the rock face or a back drainage system, is more difficult to protect.

12.2.5 Drainage behind the barrier

The build up of water pressure behind the steep side slope lining system is a possibility since in some quarries there may have been drawdown of the water table during the quarrying operations. Also, perched water tables may exist in the surrounding rocks and ground water may infiltrate into the face. It is important that this is considered in the design and a back drainage system is normally required. The drainage layer could take the form of a gravel layer, a no-fines concrete wall or possibly a drainage geocomposite.

12.2.6 Instrumentation and monitoring

Since there is still significant uncertainty regarding the performance of steep side slope lining systems, there is a need for the systems that are built to be instrumented and monitored to assess structural performance. This instrumentation could take the form of inclinometers, settlement magnets and pins, pressure cells and tilt meters. The precise details of the instrumentation would need to be designed on a site-specific basis.

12.2.7 Construction Quality Assurance

Construction Quality Assurance (CQA) is a fundamental part of landfill design and construction, and has been shown to significantly improve the quality of the lining system in terms of reducing defects. The design for a steep side slope lining system must make provisions for rigorous CQA.

12.3 Current Designs Used in the UK

12.3.1 Mineral lining systems

"Christmas tree" system

This lining system is constructed in lifts ahead of the waste. The clay is compacted in layers and brought up in stages against the quarry wall as the depth of waste increases, see Figure 12.2. This system relies on the waste for its stability and integrity. There are several problems associated with this design in particular the inner part of the base of each lift is placed on waste. As the waste compresses and degrades it undergoes significant settlement resulting in large stresses in the clay liner, which will deform and probably shear. This would then lead to uncontrolled escape of landfill gas and leachate through the sheared zones. The EPSRC funded project outlined above is monitoring a lining system of this design. Failure of this type of liner system is presented in Section 4.3 (Case history No. 3).

A variation on this design is the use of a geomembrane on the front face of the mineral liner (i.e. a composite lining system). Deformation of the barrier into the waste will also result in tearing of the geomembrane and hence a loss of integrity. In this case, the addition of an extra barrier layer is unlikely to result in any significant increase in the level of protection. Any lining system that utilises geosynthetics must afford protection to the geomembrane. A heavy non-woven geotextile is often used as a protection layer as is a layer of sand. Such protection layers would operate satisfactorily in the short-term, but any movement of the waste could lead to differential movement of the geotextile/sand and this could leave the lining system exposed to the waste and hence to damage.

Unsupported mineral liner

Concern over the performance of the 'Christmas tree' type design has led to unsupported mineral lining systems being used. A mineral liner of uniform thickness is constructed in lifts against the side slope. Often the size of the compaction plant needed to achieve the required properties (i.e. density) means that a greater width of mineral liner than is required is constructed and it is then cut back to the desired width. Each lift of mineral liner is placed on the previous lift and there is no construction of liner directly over waste. There will therefore be no liner damage due to waste settling from beneath, however the magnitude of lateral support provided by the waste is still a major concern.

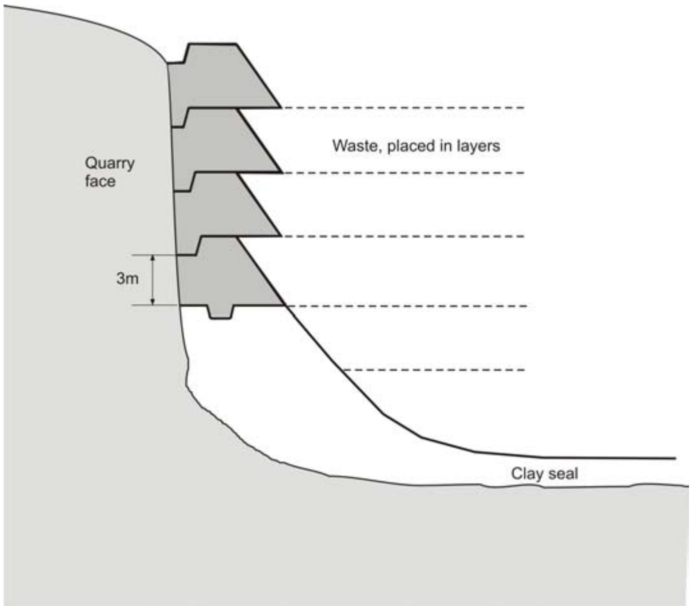


Figure 12.2 Cross-section through a 'Christmas tree' lining system

Engineered fill supported mineral liner

Instability of unsupported miner liners has led to the use of systems that introduce engineered fill between the liner and the waste and a support layer. In this system, a mineral liner of uniform thickness is constructed as described above, and as each lift of barrier is built a wedge of engineered granular fill (i.e. placed to achieve specified shear strength and compressibility) is placed against the mineral liner to ensure stability and control deformations. A cross-section through the liner is shown in Figure 12.3. The disadvantages of this system are that the engineered fill can be expensive and it takes up a significant percentage of the void available for landfilling. There is also the possibility that as the mineral liner consolidates it can get 'hung up' on the quarry wall causing differential settlement and this can lead to the integrity of the liner being compromised. While this approach has been used for a number of UK landfills, the loss of void space has resulted in the Christmas tree type design often being preferred even though it is demonstrably unstable.

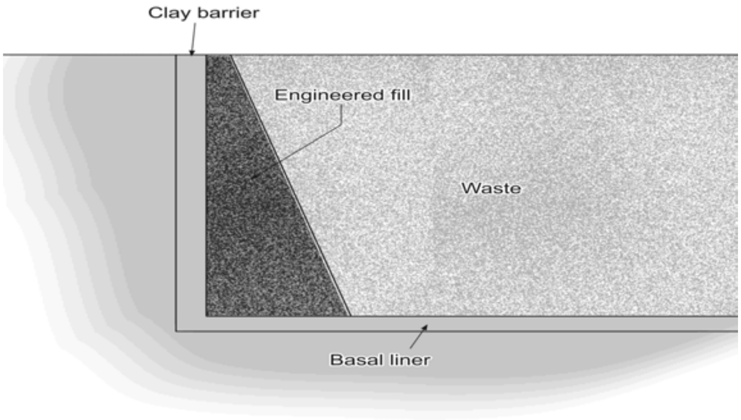


Figure 12.3 Cross-section through a mineral lining system supported by engineered fill

12.3.2 Geosynthetic lining systems

There has been an increase in the number and type of geosynthetic based lining systems used in the UK over the last ten years. They have been constructed as small-scale trials, prototypes and complete lining systems. Many are subject to patent applications. Gallagher *et al.* (2000) have produced a useful review of the systems and construction methods. At the date of publication they report that they have been used in 11 UK landfills. The systems are described below (after Gallagher *et al.* 2000).

Vertical systems

A prototype at one site has been trailed for 18 months. It comprises a triple row of vertical HDPE pipes, the central row of which is filled with low permeability slurry. The design is for vertical sided voids and it cannot be used on a site with benches. The deformation of the system during interaction with waste has not been published. Figure 12.4 shows the triple row of pipes following installation (Gallagher *et al.* 2000).

Figure 12.4 Photograph showing a prototype vertical barrier system using HDPE pipes (after Gallagher *et al.* 2000)

Revetment systems

Typically inclined at 50° to 90° to the horizontal. The main systems are double rows of gabions sandwiching a geomembrane, frames rock bolted to the side slope with a geomembrane placed in front supported and protected by either gabions or sacks of fill, and no-fines concrete and shotcrete lined slopes overlain by a geomembrane that is protected by a geotextile layer.

The common features of this family of systems was listed by Gallagher *at al.* (2000) as:

- a single geomembrane liner;
- generally high protection constraints;
- significant engineering;
- significant input required during design and construction; and
- increasing flexibility in following the quarry wall leading to gains in available void.

Gabion basket systems have been used to form a surface for geomembrane lining. An example is the use of two rows of gabions constructed parallel to the quarry face with a geomembrane placed between the gabions. Sand can be used to backfill around the geomembrane, or geotextile can be placed either side, to protect it from the steel mesh of the gabions. This system isolates the waste from the geomembrane and hence provides protection. Such a system is very costly and labour intensive since the gabion baskets are filled by hand. There are concerns regarding both the short and long-term stability of the system due to the instability of a high and thin gabion wall. The long-term stability is also dependent upon the waste/gabion wall interaction. This design is inherently unstable and its use is not recommended.

Framework systems can be bolted to the quarry wall in order to form a planar surface for geomembrane lining. The mesh covered frame is self-supporting but all systems use either gabions or bulk sacks in front of the geomembrane to protect it from the waste. These gabion/sack systems are dependent upon the waste for stability. Geotextile protection layers are required both between the frame and the geomembrane and the geomembrane and the support system. Free draining material is filled between the frame and quarry wall to stabilise the frame. The protection layers, geomembrane and support system are placed in a series of lifts working from the waste. Figure 12.5 shows a cross-section through a bulk sack based system.

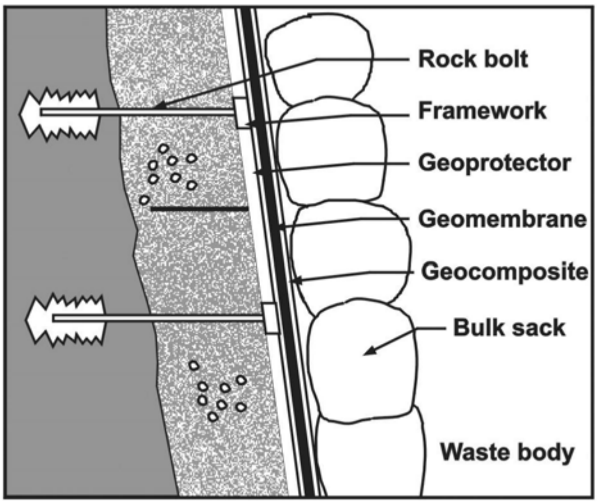


Figure 12.5 Cross-section through a frame supported geomembrane system (after Gallagher *et al.* 2000)

There is no information in the literature detailing the geotechnical performance of the frame systems, although there is anecdotal evidence that the geomembrane can become stressed and that the structural integrity of the frame can be compromised. This is a promising approach but in service monitoring is required to demonstrate long-term performance. Gallagher *et al.* (2000) provide a more detailed explanation of the development of these frame systems and discuss the merits of the various designs.
It is not known whether a no-fines concrete geosynthetic system has been used in the UK although it has been employed at one the large strategic Hong Kong landfills. It comprises placing a geosynthetic lining system onto a previously cast wall made of no-fines concrete. No-fines concrete is placed against the quarry face using temporary formwork in the conventional manner to form a smooth surface for the geosynthetics. Due to its porous nature, it can be used to drain any seepage in the quarry face and prevent any build up of water pressure behind the lining system. The geosynthetics comprise two heavy non-woven geotextiles either side of a mono-textured geomembrane, see Figure 12.6.

The lower geotextile provides protection for the geomembrane from any irregularities within the no-fines concrete, while the upper geotextile provides protection from the waste, in addition to the sand layer. The smooth surface of the mono-textured geomembrane is placed against the upper geotextile such that as the waste settles and drags down the sand, little stress can be transferred into the geomembrane. This approach is used in many of the steep slope lining systems. The tensile force induced in the geomembrane governs the height to which such a lining system can be designed. Forces can be transmitted to the geomembrane through shear stresses at the interfaces, and for heights over 10m the self-weight of the geomembrane can become significant.

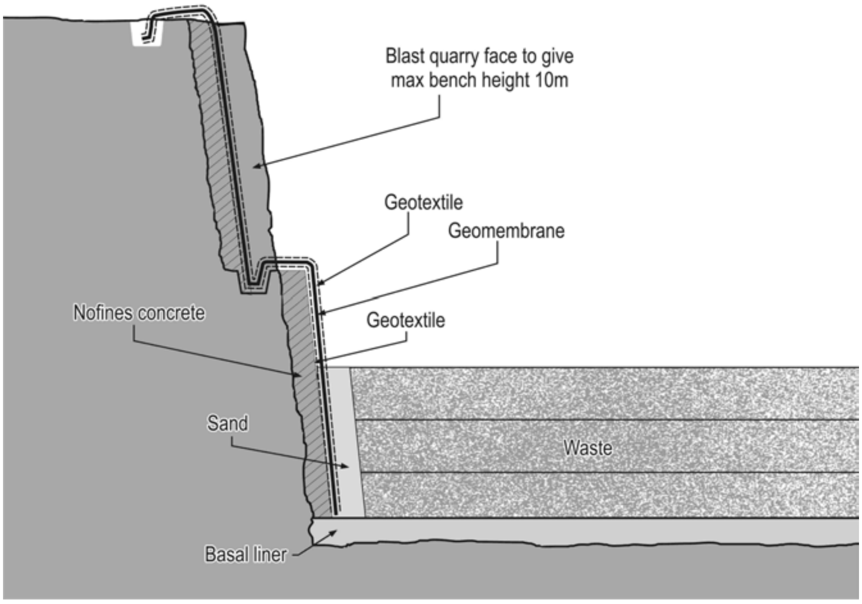


Figure 12.6 Example of a no-fines concrete supported geomembrane system

A shotcrete geosynthetic system is a similar approach to the no-fines concrete method. It has been considered for use in one of Hong Kong's strategic landfills. A relatively smooth surface for lining is created by spraying shotcrete onto the rock face.

Reinforced soil systems

Used to line several sites with side slopes up to 70° from the horizontal. The method relies on a reinforced earth slope to create a stable surface for geomembrane lining, and is typically designed for a 3m lift. The reinforcement generally used is a geogrid and relatively short lengths are required for stability of a 3m lift (depending on the engineering properties of the fill used). The lining system can comprise a mono-textured geomembrane with the textured side down, together with a heavy non-woven geotextile on top for protection. A sand protection layer is required to separate the waste from the geosynthetics. Designing such a system on a lift-by-lift basis requires confidence that the waste will provide sufficient support for the lower layers during construction of subsequent lifts and for the long-term case. At present there is no field evidence for the geotechnical performance of such structures following waste placement (i.e. measured deformations). It is vital for such systems to have smooth front faces for geomembrane placement. Overlapping geogrids at the surface can produce "pinch points" that will act as points for stress concentration in the geomembrane. Methods of creating the front face that have been used include mobile ply and steel formwork, permanent steel formwork and permanent polystyrene formwork. The use of polystyrene offers several advantages such as ease of construction, but has numerous durability issues in terms of chemical compatibility and heat resistance that do not seem to have been addressed by the designers. A cross-section through a reinforced soil system with polystyrene formers is shown in Figure 12.7 (after Gallagher *et al.* 2000).

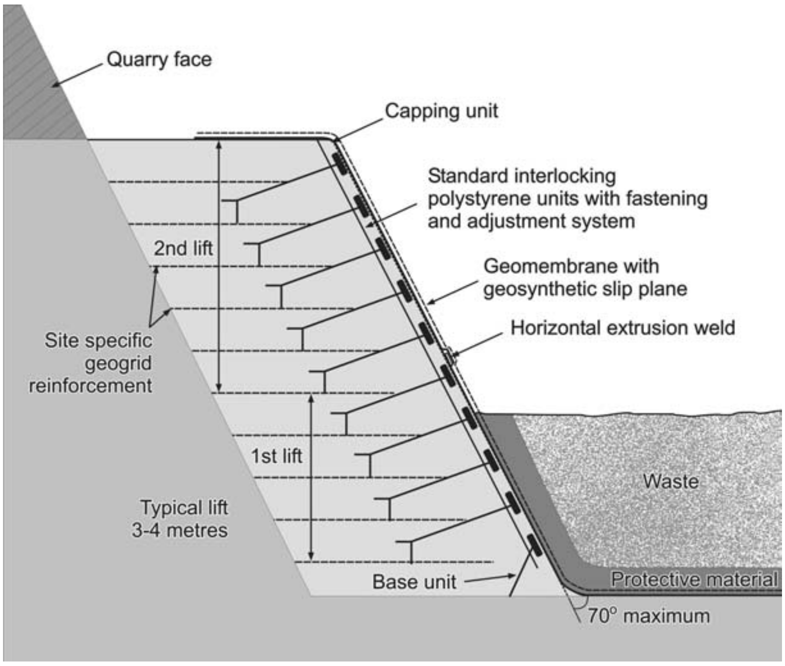


Figure 12.7 Cross-section though a typical reinforced soil supported lining system

Dense asphaltic concrete

The use of dense asphaltic concrete (DAC) as a landfill barrier is now accepted in Switzerland and Germany for side slopes up to 45 degrees. It is placed using specialist plant using winches that has been developed for earth dam applications. There is a possibility of placing the DAC on steep slopes, up to vertical, for landfill applications although some form of lateral support would be required. The main reason for this is the large creep strain that the material undergoes when unsupported. DAC has been shown to have the low permeability and high chemical resistance required for use as a landfill barrier (Christie & Pfiffner 1993). It has the advantage that it is more robust than a geosynthetic lining system and does not need to be protected from the waste, although a thin separation layer is recommended. It can be placed in thinner layers than clay. Specialist plant would be required for vertical lining systems, as well as a requirement for a buttressing layer. In order to develop DAC as a potential system for steep sided landfills, further investigation is required into its creep strain and the support required to prevent this creep.

Cast in situ concrete with embedded geomembrane

The use of no-fines concrete to form a smooth surface for lining and to act as a back drainage layer has possibilities in addition to those described above. The major disadvantage of the standard approach is that since the geomembrane is fixed at the top, there is a limit to the height that can be constructed in one lift. If a suitable system was developed in which the geomembrane was fixed continuously, or at regular intervals, on the face of the no-fines concrete, then there would not be the requirement for benches. A system that may be viable is based on the use of the concrete protective liner materials. This material is designed for casting into concrete structures and consists of a regular series of protrusions on one side to enable the bonding with the concrete. The usual protection layers would be required.

12.4 Stability and Integrity: Factors Controlling Failure

The main design issues related to stability and integrity of steep slope lining systems in both the short and long-term are summarised below. Barrier types have been categorised into self-supporting systems and those that rely on the waste for support.

12.4.1 Self supporting systems

These include systems that are stable at full height without the presence of waste and the temporary condition of lifts of waste supported systems prior to waste placement.

Stability issues

Assessment of structural instability of lining systems is required for short-term conditions and should include shear failure mechanisms in mineral liners (e.g. lifts of 'Christmas tree' and engineered fill buttressed systems), tensile failure of geomembranes under self weight and loads from other liner components (e.g. revetment systems) and structural instability of support systems (e.g. lifts of reinforced soil, frame systems bolted to rock face and no-fines concrete).

Structural stability assessment is required, taking into consideration the long-term interaction between the barrier and waste (i.e. large waste settlements adjacent to the lining system). This should include assessment of shear failure mechanisms in the engineered fill buttressed mineral liners, possible tensile failure of geomembranes through over-stressing as a result of interaction between the lining components and settling waste body and structural instability of support systems under the imposed waste loads (e.g. failure of reinforced soil elements).

Integrity issues

Assessment of the long-term integrity of lining systems requires an understanding of the interaction between the lining system and waste and hence quantification of the strains in the lining components (i.e. even if they do not lead to instability). Differential deformation of a mineral liner (e.g. engineered fill buttressed system) can result in the formation of tension cracks and shear zones that increase its permeability to leachate and gas. Desiccation of the mineral liner will also result in an increased permeability. Differential deformation of support systems (e.g. reinforced soil and support frame) can strain the overlying geomembrane resulting in the development of stress cracking in the long-term. Differential movement of the waste can lead to a loss of protection to the geomembrane through tensile failure of geotextile layers and loss of continuity of sand protection layers. This will then expose the geomembrane to mechanical damage from gravel drainage material and waste.

12.4.2 Waste supported systems

Assessment of stability during construction (i.e. pre-waste placement against a lift) is discussed above. The issues relating to stability post waste placement and integrity in the long-term are similar to those for self-supporting systems but the role played by the waste body, and hence the magnitude of liner deformations, is more significant.

Stability issues

Structural stability assessment is required to take into consideration the long-term interaction between the barrier and waste, which can result in large deformations of the lining system into the waste body in an attempt to mobilise lateral restraint and hence establish a condition of limiting equilibrium. The size of the deformations required to establish equilibrium may result in a condition of lining failure. If equilibrium is established, then the integrity of the lining system must be considered (see below). Failure conditions that could occur include shear failure in mineral liners via the mechanisms identified by Edelmann *et al.* (1999) and shown in Figure 12.1, tensile failure of geomembranes through over-stressing as a result of interaction between the lining components and waste and structural instability of support systems under the imposed waste loads (e.g. failure of reinforced soil elements).

Integrity issues

Assessment of the long-term integrity of lining systems requires an understanding of the interaction between the lining system and waste, even if the system is considered to be 'stable', and hence quantification of the strains in the lining components. Deformation of a mineral liner into the waste body can result in the formation of tension cracks and shear zones that increase its permeability to leachate and gas. Desiccation of the mineral liner will also result in an increased permeability. Differential deformation of support systems (e.g. reinforced soil and support frame) can strain the overlying geomembrane resulting in the development of stress cracking in the long-term. Differential movement of the waste can lead to a loss of protection to the geomembrane through tensile failure of geotextile layers and loss of continuity of sand protection layers. This will then expose the geomembrane to mechanical damage from gravel drainage material and waste.

12.5 Stability: Analysis Methods

12.5.1 Limit equilibrium approach

The use of limit equilibrium slope stability methods is relevant for a number of configurations of mineral, geosynthetic and composite lining systems. The methods of analysis outlined in Section 11.4 should be used (i.e. based on the method of slices). Configurations of steep slope liner systems that require assessment of slope stability as part of the design process include:

- short-term (pre-waste placement) stability of mineral liners such as lifts of both "Christmas tree" and engineered fill buttressed systems (see Figure 12.8a, upper failure);
- long-term (post-waste placement) stability of mineral, geosynthetic and composite systems in cases where the adjacent waste body has a slope profile close to the barrier (see Figure 12.8b), and failure occurring into the waste body (Figure 12.8b, upper mechanism).

As with failure of shallow slope lining systems, critical shear surfaces will follow weak layers and interfaces and hence will often be non-circular in shape. Examples are interfaces between geosynthetics and geosynthetics/mineral liners (i.e. in composite systems). Preferential swelling, and hence softening, can take place in clay liners adjacent to drainage systems. This will result in a weaker layer of clay that could control the position of a critical shear surface (i.e. forming within the softened clay and following the boundary of the clay layer). When analysing the stability of mineral liners during construction, or shortly following waste placement, it is common practice to use undrained shear strength parameters in a total stress analyses. An assessment must be carried out to demonstrate that this is an appropriate approach. Consideration should be given to the rate of construction in relation to the swelling rate of the material forming the mineral liner. It should be remembered that steep slopes in cohesive soils are kept stable by pore water suctions and that as these suctions dissipate, stability decreases (see Chapter 9). Circular shear surfaces can be assumed in certain analyses (e.g. Figure 12.8a) and therefore stability charts could also be used in these circumstances. However, the use of these simplified methods must always be justified.

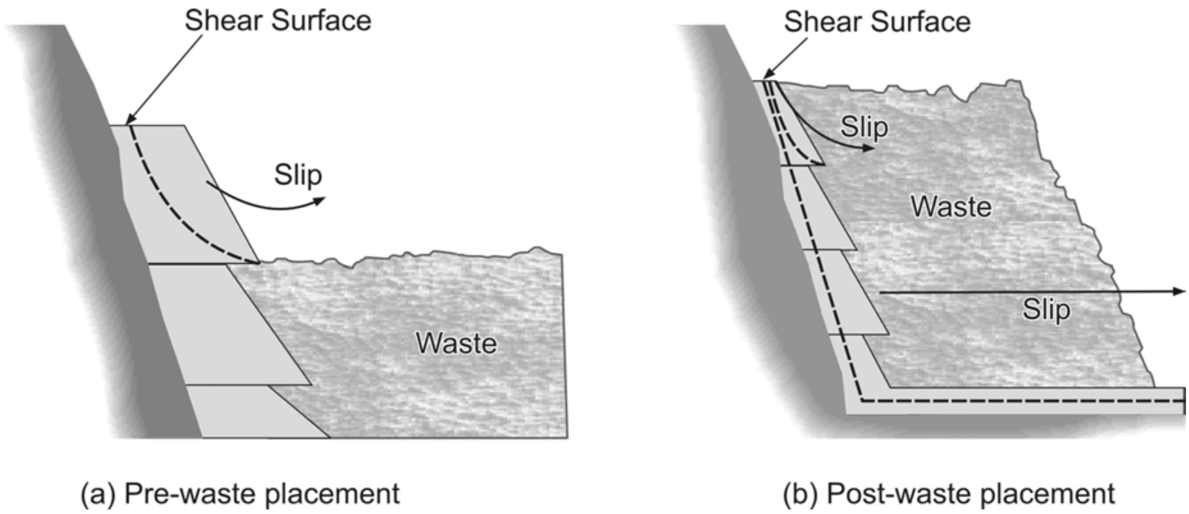


Figure 12.8 Possible slope failure modes of steep wall barrier systems

Analysis of the failure mode shown in Figure 12.8b can be carried out following the guidance given for shallow slopes (see Section 11.4). Analysis of slope failure mechanisms involving movement into the waste body (e.g. the upper failure in Figure 12.8b) can be assessed using the method outlined by Jones and Dixon (1997). Stresses in the waste body calculated from the self-weight (vertical stresses) and K_o values (used to calculate the horizontal stresses, see Section 8.6) are applied to the external slope of the barrier. A traditional limit equilibrium stability analysis can then be carried out.

12.5.2 Reinforced soil design

The stability of a reinforced soil support system prior to waste placement must be demonstrated as part of the design process. Design and assessment of these structures should be carried out in accordance with *BS 8006: 1995 Code of practice for Strengthened/reinforced soils and other fills*. Both the ultimate limit states (i.e. associated with collapse) and the serviceability limit states (i.e. relating deformations to prescribed limits) should be considered. This is required to demonstrate that the reinforced soil system is stable and has the required rigidity to support the lining systems (e.g. limit strains in geomembranes). Performance of the reinforced soil structure post-waste placement should be considered in terms of both stability (i.e. under the weight of waste) and integrity by considering the interaction between the reinforced soil structure, the lining system and the waste (see section 12.6). Potential variations that result from the construction process should be included (e.g. variations in geometry and in soil density).

12.5.3 Structural support systems (revetments)

Analyses are required to demonstrate the structural stability of revetment type support systems pre-waste placement (e.g. frames, gabions, no-fines concrete). This means assessing their stability under self-weight loading. Consideration should be given to the design of rock bolt systems (frames), internal shear failure of no-fines concrete and the stability of gabion type systems (i.e. assessing over turning, bearing capacity and buckling modes of failure). In all cases the influence of any granular backfill must be included.

Structural stability must also be considered post-waste placement. The forces imposed by the waste on the support system should be quantified. These result from the in situ waste stresses and are modified by waste settlement. Frame systems must have an adequate factor of safety against collapse. Gabion wall type structures must be structurally stable under the stresses imposed by waste settlement. Assessment of stability post-waste placement requires a detailed understanding of interaction with the waste and hence is more easily considered along with integrity issues employing numerical modelling techniques (see Section 12.6). The likely 'as built' wall profile should not be over simplified in the analysis as this can lead to important mechanisms of failure being missed (see Section 12.6.7).

12.6 Integrity: Analysis Methods

As discussed in Section 12.1, barrier deformation post-waste placement is controlled by the waste/lining system/support system interaction (assuming that the sub-grade is stable and relatively incompressible). Assessment of the magnitude and distribution of deformations in lining system components (e.g. barrier and protection layers) is required if the long-term integrity is to be demonstrated (e.g. over-stressing of geomembrane and loss of protection, and the formation of tension cracks and shear zones in mineral liners). This leads to a requirement to use numerical modelling methods. An introduction to this approach and details of a common analysis method (FLAC) are provided in Section 11.4.4. This section also demonstrates the importance of carrying out such analyses. Key issues relevant to numerical modelling of steep side slope lining systems are discussed below.

12.6.1 Waste mechanics properties

Behaviour of the waste body controls the performance of the lining system. Appropriate ranges of the waste material properties must be used in any analysis in conjunction with a constitutive model that enables the observed mechanical behaviour of waste to be represented. Specifically, the material model should reflect the volumetric strain hardening behaviour of waste (i.e. increasing stiffness and shear strength resulting from decreasing volume of material caused by increasing stress). A key outcome of using such a model is that the main parameters of unit weight, stiffness and strength increase with depth of burial (i.e. increased stress). However, even using such a relatively sophisticated material model, consideration should be given to the likely ranges of the main parameters. Sensitivity analyses must be conducted to gain an understanding of the significance and possible variation of the predicted barrier deformations. Some models (e.g. FLAC) allow the input of a random spatial distribution of the material properties. This enables the consequences of waste heterogeneity to be studied.

12.6.2 Strain-softening interfaces

The importance of material strain incompatibility and strain-softening interfaces is discussed in detail in Section 11.4 in relation to the performance of shallow side slope lining systems. Strain-softening interfaces must be modelled, if present. They influence the magnitude and distribution of both stresses in and deformations of, the lining system components. For example, a low strength strain-softening interface between a smooth geomembrane and a geotextile can be used to isolate the geomembrane from the large settlements in the waste, and hence to minimise the stresses in the geomembrane. If this interface were not modelled correctly, stresses in the geomembrane would be over predicted.

12.6.3 Structural support system

As discussed in Section 12.5.3 the performance of the lining support system under waste loading has an important influence on the stresses and strains in the lining components. In many instances it will not be possible to model the support elements directly (e.g. a three-dimensional frame rock-bolted to the quarry face). In these cases it is acceptable to model the support system using a region beneath the lining with material properties that represent the mass behaviour of the support. Developing such models requires significant levels of specialist knowledge and experience. All models by necessity include approximations and simplifications. The most important often being the use of a two-dimensional model to represent a three-dimensional problem. If these approximations and simplifications are not taken into consideration, interpretation of results will be flawed and this could lead to unsafe designs. An example is given in Section 12.6.7.

12.6.4 Staged construction

Waste is placed in layers and most barriers are constructed in stages. In a numerical analysis the stages of construction must be modelled in order to obtain representative results. Lining systems constructed in stages can deform significantly during the construction process because the layers of waste initially placed against the lining provide low levels of support. This is because at shallow depths of burial the waste has a low stiffness and shear strength. These increase with depth of burial (i.e. increased vertical stress). Simplifying a model to place all construction stages of the lining, and the waste, in one event will lead to an underestimate of the lining deformations, and hence an overestimate of the lining integrity.

12.6.5 Waste degradation

Demonstrating long-term integrity of the lining system requires an assessment of any changes that occur in response to the waste settlement that accompanies degradation. At present there is inadequate information on the likely changes in material properties that occur in response to degradation. However, an analysis should attempt to assess the response of the liner to the predicted waste settlements. The simplified approach described in Section 11.4.4 generates the magnitude of likely settlements but not the correct mechanism. The degradation process and its effect on the waste engineering properties are not specifically modelled.

12.6.6 Properties of lining components

Appropriate material models (i.e. stress/strain relationships) must be selected for the lining components as it is the stresses and strains in these elements that are investigated to assess integrity. Whether clay liners are modelled as either drained or undrained, the selected approach must be justified. Other aspects of material behaviour must be included in the model as appropriate. For example, if assessing the performance of a dense asphaltic concrete lining, the creep behaviour of the DAC should be incorporated and modelled.

12.6.7 Representation of site variability in the model

As discussed in Section 12.6.3, simplification of the physical site is necessary to produce a model but this can also lead to inaccurate and misleading results. Particular care should be exercised in simplifying the quarry side slope geometry and the likely ‘as constructed’ lining profile. Modelling slopes as having constant angles can result in an underestimate of stability and overestimate of integrity (i.e. both on the unsafe side). Changes in the angle of a slope along its length can introduce mechanisms of failure that are not present in a constant angled slope. Figure 12.9a shows a cross-section through an idealised quarry slope, which could be used in a numerical model. Figure 12.9b shows a cross-section through the more complex actual slope. If the slope were lined with a compacted mineral liner, a numerical analysis would be required to assess the long-term stability and integrity of the liner (post-waste placement). A liner on the planar slope shown in Figure 12.9a has a higher degree of stability than the same liner on the actual slope shown in Figure 12.9b. The changes in slope cause concentrations of shear stresses, and hence strains, at the locations shown in Figure 12.9b. Using a constant angle and therefore simplifying the model could result in potentially important modes of behaviour being missed.

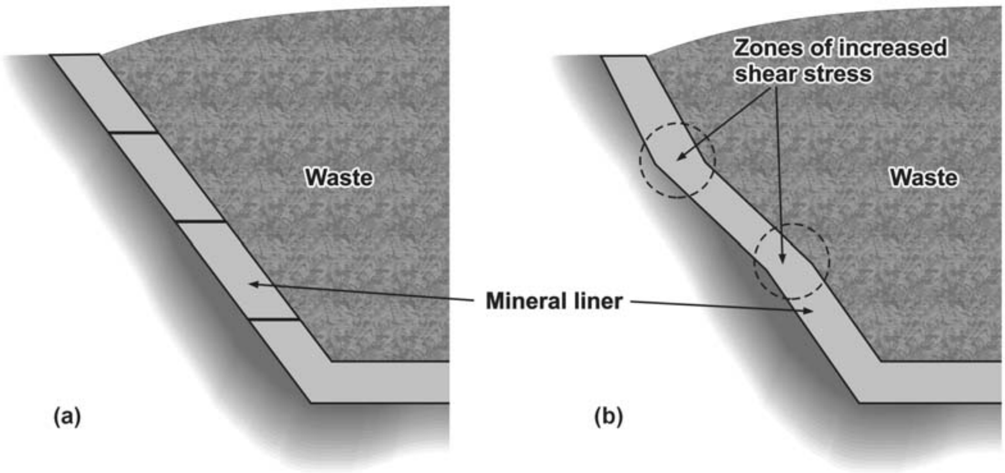


Figure 12.9 Cross-sections though a) the idealised slope and b) the actual slope, demonstrating that over simplification of geometry can lead to unsafe results

12.7 Summary of Key Issues

Current designs for steep slope lining systems can be divided in two. Those that are self supporting, and hence could be constructed to full height prior to waste placement, and those that rely on waste for stability. Those in the latter category are the most common in the UK, but there are significant concerns over their long-term stability and integrity. This is due to the low stiffness of the waste that is relied upon for support. A full assessment of the performance of all types of lining system can only be made by considering the waste/barrier interaction. This requires the use of numerical modelling techniques, and hence a relatively high level of knowledge on the mechanical properties of the waste. For self supporting systems the structural stability of the liner sub-grade (i.e. reinforced earth, steel frame etc.) must be assessed as part of the design, including performance under loading from the waste during settlement. The stiffness of the support systems should be considered to ensure that long-term deformations do not lead to straining of the liner and hence loss of integrity.

Compacted clay liner systems that rely on waste for support (e.g. 'Christmas' tree) are used in the UK, but there are severe concerns regarding both their short- and long-term performance. Unless proof of performance can be given, they should not be used.

13. WASTE SLOPE STABILITY

13.1 Introduction

Instability of waste slopes that involves elements of the lining system and/or the sub-grade has been discussed in detail in Section 11.4. This is by far the most common mode of failure. However, it is possible for the slope failure to be entirely within the waste body. Waste slopes must be designed to ensure an adequate factor of safety against the occurrence of this mode of failure both in temporary and long-term slopes.

13.2 Stability: Factors Controlling Failure

13.2.1 Engineering properties of waste

For analysis of slope stability, characteristic values of the main engineering properties of waste must be selected. The issues surrounding their selection are discussed in Section 11.4. The key parameters are unit weight and shear strength.

13.2.2 Leachate

As discussed in Section 11.4.1, leachate distributions in waste bodies can be complex. Particular attention should be given to the possibility of perched (or localised) leachate being present, especially above cover soil layers. These can have an important impact on the location of the critical shear surface and the factor of safety against failure. The potential for the presence of significant pore pressures in the waste due to stress dependent permeability, even if the base drain is functioning correctly, should also be considered. Note that the higher the leachate pore pressures the lower the effective stresses in the waste and hence the lower the waste shear strength.

13.2.3 Location and shape of potential shear surface

The location and shape of the critical shear surface controlling stability is likely to be influenced by the anisotropic strength of MSW (i.e. related to the method of placement and compaction) and the presence of daily cover soil layers. The layered structure of MSW will dictate that the shear strength along sub-horizontal planes within the waste will be lower than on an inclined plane. The reinforcement produced by elongated components of waste will be a minimum along sub-horizontal planes and on inclined planes between phases of filling. The important role of such reinforcement, and hence tensile strength, on the angle of the rear part of the shear surface is discussed in Section 11.4.1).

The waste mass is often stratified with daily cover soil layers. Depending on the site practices for waste placement, and the depth of burial, these layers of soil can have a vertical spacing in the order of 1m. They are usually sub-horizontal and extend over a significant area. Depending upon the thickness of the layer and the soil type (i.e. cohesive or granular) it is possible for these to form preferential paths for the critical failure surface. A contributing factor is that these layers often have permeabilities lower than the waste and this can cause the perched leachate. This in turn can reduce the effective stresses in the soil and hence can result in lower shear strengths. Waste anisotropic strength and daily cover soil layers will result in the critical shear surface being non-circular in many cases (i.e. with a planar sub-horizontal basal section). Figure 13.1 shows an example of how the location and shape of a shear surface could be influenced by the structure of the waste. The shear strength at interfaces between a cohesive cover soil and waste can also be reduced due to the strain incompatibility between the two materials (see Section 11.2).

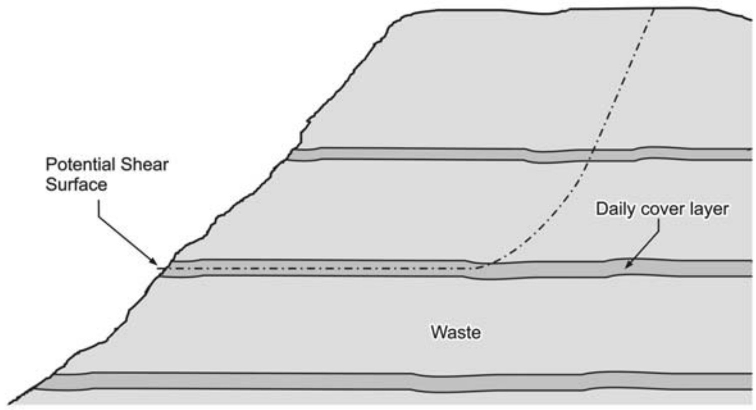


Figure 13.1 A cross-section showing the influence of waste structure on the location and shape of a shear surface

13.3 Stability: Analysis Methods

Stability analyses should be conducted using the limit equilibrium approach outlined in Sections 11.4. Analysis should be based on the method of slices in order to represent the change in shear strength around the shear surface resulting from changes in effective stress. The likelihood that the failure surface will be non-circular in shape means that the simpler 'circular' techniques may not be appropriate. The possible variability of the input parameters must be considered by undertaking a sensitivity analysis. In addition, a range of leachate pressure distributions should be assessed and a rigorous search must be conducted to find the location of the critical shear surface (i.e. the surface that gives the lowest factor of safety). Loads from construction plant and surcharge from temporary stockpiles (e.g. stores of cover soils) must be considered.

13.4 Impact of Changes in MSW Composition

As discussed in Sections 8.1.3 and 11.4 the distribution and type of constituents in MSW are constantly altering due to the impact of legislation and changes in lifestyle. This will have an impact on the safe angle of future waste slopes. The removal or reduction of reinforcement type elements (e.g. paper, garden waste and plastics) will result in a reduction in shear strength. It is possible that shallower waste slopes will have to be constructed. Of particular note is that past experience of designing stable waste slopes will not be relevant in the future, and may in fact lead to the construction of unstable, and hence, unsafe slopes.

At present there is insufficient information to assess whether degradation causes a reduction in shear strength of MSW. If degradation causes physical modification of some reinforcing elements (e.g. paper and wood) it is possible that the resulting reduction in shear strength will balance any increase due to higher densities, and hence that the net effect of degradation may be a reduction in shear strength. The shear strength parameters of MSW used in design must relate to site-specific waste materials and not rely on general information in the literature, unless it can be demonstrated that such data is relevant. Parameters used in design must also be consistent with predicted changes in MSW constituents during the time scale of filling (i.e.

the impact on shear strength of possible changes in MSW over this time scale should be considered).

13.5 Summary of Key Issues

Although failures that occur wholly in the waste body are rare, this condition must be checked and temporary and permanent waste slopes should be designed to ensure stability. Many of the factors controlling failure are the same as those discussed in Section 11.4. The possible magnitude and distribution of leachate pressures should be assessed, with particular attention given to the formation of perched leachate above cover soil layers. Consideration must also be given to the role that waste anisotropy and the cover soil layers play in the location and shape of the critical shear surface. A rigorous search must be conducted to demonstrate that the minimum factor of safety against slope failure has been calculated. Limit equilibrium analysis methods can be employed to assess stability but it is likely that non-circular shear surfaces will control stability, and hence that the more complicated non-circular analysis techniques will have to be used. Possible changes in MSW constituents during the time scale of the filling operations should be considered in order to assess whether the waste shear strength will reduce. The design must use the worst-case strength for the life of the slope. The influence of degradation on shear strength is at present unknown.

14. SUMMARY

14.1 Major Issues Identified in the Literature

This report has presented the results of a detailed literature review of the issues associated with the stability of landfill lining systems. Summaries of the key issues identified within each chapter are given at the end of the chapter. The major issues have been identified as:

- the stability of landfill lining systems can be considered in terms of stability failure and integrity failure;
- there are significant difficulties in the measurement of interface shear strength;
- characteristic values should be used in design;
- waste properties are important; they control long-term lining system stability and integrity;
- difficulty in obtaining relevant waste parameters and their variability;
- soil mechanics principles are important for landfill design;
- uncontrolled groundwater causes many failures during construction;
- sub-grade stability must be assessed as part of the design;
- long-term deformations of sub-grade can compromise the integrity of lining systems by excessive settlement, development of voids and basal heave;
- weak layers and interfaces are important for stability;
- leachate pressures often control instability;
- seepage pressures are important in veneer stability;
- waste/lining system interaction controls mobilised strength in lining materials (strain softening behaviour);
- waste/lining system interaction controls the stability and integrity of steep slope lining systems; and
- the need to monitor the in-service structural performance of lining systems.

The information gained in this literature review has been assimilated to produce guidance on the stability of landfill lining system, and this is presented as Report No. 2.

14.2 Limitations of Current Knowledge and Practice

14.2.1 Current knowledge

From the literature review the following limitations of current knowledge can be identified:

- internal strength of geocomposites and geosynthetic clay liners;
- interface shear strength:
    - methods of measurement in particular at low normal stresses;
    - creep behaviour of interfaces;
- selection of characteristic values for use in design;
- mechanical properties of waste and their change in response to the changing waste stream;
- integrity of mineral liners subjected to strains;

- mobilisation of post peak shear strengths in shallow side slope lining systems;
- guidance for the use of limit equilibrium techniques for liner integrity assessment; and
- structural behaviour of steep side slope lining systems:
  - waste support condition and strain compatibility;
  - strains induced by waste settlement;
  - constitutive model for waste.

14.2.2 Current practice

In addition to the above limitations in knowledge, the following limitations in current practice have been noted:

- assessment of stability is often not carried out by a suitably qualified person;
- conventional geotechnical investigation and analytical methods are often not applied;
- integrity of lining systems is seldom assessed and use of numerical modelling techniques is limited;
- selection of factors of safety is rarely justified in relation to the available knowledge and consequence of failure; and
- structural performance monitoring of lining systems post waste placement is not carried out.

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GLOSSARY

Term	Definition
Adhesion	Shear strength at zero normal stress between soil and another material (e.g. geosynthetic, pile or retaining wall). See also Cohesion.
Aggregate	The constituents, comprising sand and gravel used in the manufacture of dense asphaltic concrete and other types of concrete.
Anisotropic	The property of a material (e.g. permeability) which varies with the direction of measurement at a point in the medium (e.g. vertical versus horizontal).
Asphalt	Well graded aggregate with a bituminous binder and filler.
Aquifer	A geological formation that is capable of yielding usable quantities of groundwater to wells or springs. Movement is principally in a horizontal direction through porous underground strata.
Aquitard	A relatively low permeability stratum from which it is relatively difficult to extract significant volumes of water.
Artesian pressure	Water pressure in a confined aquifer with a hydraulic head above the ground surface.
Back-analysis	Obtaining material or problem related parameters after an event by using the observed behaviour.
Barrier layers	Materials forming part of the lining system which impede migration of leachate and gas (e.g. geomembrane, compacted clay liner).
Basal heave	Upward movement of the base of the landfill/ excavation.
Basal liners	Lining systems used on the base (and sides) of the landfill cell.
BAT	Best available technique. The most effective technology and method of operation, available within a relevant industry sector, that provides a high general level of protection of the environment. The techniques should be technically and economically viable, and be reasonably accessible to the operator.
Bentonite	A type of clay composed primarily of montmorillonite with a high affinity for water giving a high swelling and shrinkage potential.
Berm (or Bund)	An artificial ridge of earth or other material used to mitigate against visual and/or noise effects or, within a landfill, to contain leachate on an interim basis or provide stability to the toe of a landfill slope.
BES	Bentonite enriched soils. Bentonite is added to soils (usually sand) in order to produce a low permeability material that can be used as a barrier layer.
Biodegradable	The ability of a substance to be broken down physically and/or chemically by micro-organisms.
Borehole	A hole made in a geological formation by drilling. It is used to determine soil and rock characteristics, and also permits the installation of instruments for monitoring groundwater and ground deformations.
Capping liner	A layer forming part of the capping system used as a low permeability barrier on the surface of the waste body in order to control the ingress of water and uncontrolled escape of landfill gas.

Cell	A recognisable independent unit of a landfill.
Characteristic values	A cautious estimate of the value affecting the occurrence of the limit state (e.g. strength).
Christmas tree liner system	A steep slope lining system based on lifts of compacted clay formed against the sub-grade slope, buttressed by lifts of waste and overhanging the waste at each lift.
Clay	Soil size particles smaller than 0.002 mm comprising clay minerals.
Clay geosynthetic barrier	See GCL.
Closure	The period of a landfill where no further waste is accepted at the site for disposal, but the waste still has a potential to cause pollution.
Coefficient of variation	Calculated for a set of data as standard deviation of the data divided by the mean of the data.
Cohesion	Shear strength of a soil at zero normal stress (*c.f.* adhesion).
Cohesive soil	A soil formed primarily from clay and silt sized particles (*c.f.* cohesionless soil, granular soil).
Commercial waste	Non-hazardous solid waste generated by business activities.
Compaction	Reduction in bulk of fill through removal of air via rolling, tamping or other mechanical means.
Compaction curve	The curve showing the relationship between dry unit weight (density) and the moisture content of a soil for a given compactive effort.
Compaction test	A laboratory procedure to obtain the compaction curve (dry density/moisture content relationship), see BS1377.
Confined slope	A slope buried under a body of material (e.g. waste).
Confined aquifer	An aquifer which is overlain by an aquitard.
Confining layer	A body of geological materials (aquitard) in the subsurface which is of sufficiently low permeability to limit significantly the flow of water into or out of the underlying aquifer.
Consolidation	The process whereby the application of pressure on a soil layer over a period of time causes a reduction in its volume by expelling fluid from the pores and the packing of soil particles closer together.
Constrained modulus	A measure of the stiffness of a layer when subjected to a one dimension change in stress. Calculated from change in stress divided by change in vertical strain.
COPA	Control of Pollution Act 1974.
Cover (daily and intermediate)	Material that is placed on the waste during construction of the landfill to minimise impacts due to: the blowing away of waste, birds vermin and odour.
Cover (final)	Materials (e.g. soil, geosynthetics) placed over the waste after completion (of a portion) of the landfill. This represents the final surface of the landfill and is intended to a) control the infiltration of water into the landfill, and b) prevent the uncontrolled escape of landfill gas from the landfill.
CQA	Construction quality assurance the process of checking quality of materials, construction and compliance with design.

Creep	Time dependent deformation of a material under a condition of constant stress.
DAC	Dense asphaltic concrete. A low permeability material used as a barrier layer in water retaining structures and landfills.
Density	The ratio of mass of a substance to its volume.
Desiccated	Dry and friable due to removal of moisture.
Direct shear test	Procedure used to measure the shear strength of a material or the interface between two materials. Shearing at a pre-determined location (i.e. the plane between the top and bottom parts of the apparatus) is caused to occur at a constant rate of strain. Values of peak shear strength are obtained.
Discontinuities	Any mechanical discontinuity within a soil or rock mass (e.g. joint, bedding plane, fissure).
Domestic waste	Solid non-hazardous waste generated from households. Also referred to as residential waste or municipal solid waste (MSW). It does not include liquid waste or hazardous waste.
Drainage layers	High permeability materials (e.g. granular soils) that form part of the lining system. They are used to transmit fluids to collection points where they can be removed from the landfill. Hence they reduce the build up of fluid pressures on the barrier layers.
Drained	Containing pore water pressures that are in equilibrium with the hydraulic boundary conditions.
Dry density	The mass of mineral matter divided by the total volume it is within.
Dry of optimum	See optimum moisture content.
Dry unit weight	The weight (force) of mineral matter per unit total volume.
DSA	Direct shear apparatus used in the direct shear test.
Effective stress	The stress carried by the soil particles, being the difference between the total stress (from self weight and external loads) and the pore water pressure. Effective stress controls the shear strength of a soil.
Engineered clay	Clay compacted to achieve required engineering parameters such as density, shear strength and stiffness.
Engineering properties	Properties of a material that define its mechanical behaviour (e.g. in response to a change in stress).
Factor of safety	Numerical expression of degree of confidence against failure.
Failure envelope	The relationship between shear strength and normal stress defining the failure state for the material.
Field vane	A field test used to measure the undrained shear strength of cohesive soils.
Fissure	A narrow opening, cleft or crevice.
FML	Flexible membrane liner. Term previously in use (see geomembrane).
Gabion baskets	Rock filled rectangular wire mesh boxes which may be laid like bricks to form a structure for retaining soil.
Gas flux	Flow rate of gas.
Gas well	Structure used to extract landfill gas in order to control gas pressures within the landfill.

Geocomposite	Manufactured, assembled material using at least one geosynthetic product among the components.
Geocomposite clay liner	See GCL.
Geogrid	Planar, polymeric structure consisting of a regular, open network of integrally connected tensile elements and whose openings are much larger than the constituents, used for reinforcement in geotechnical and civil engineering applications.
Geological barrier	The in situ geological formation underneath the constructed liner that provides sufficient attenuation to ensure that no unacceptable discharges are made.
Geomembrane	A very low permeability sheet used as a liquid and vapour barrier in geotechnical and civil engineering applications, e.g. HDPE, PP, etc. (historically called FML).
Geonet	Planar, polymeric structure consisting of a regular, dense network of integrally connected overlapping ribs, used for liquid and vapour transmission in geotechnical and civil engineering applications.
Geosynthetic	A polymeric material, synthetic or natural, used in geotechnical and civil engineering applications.
Geosynthetic clay liner (GCL)	A low permeability sheet constructed from a thin layer of clay bonded to either one or two layers of geosynthetic material that is used as a liquid and vapour barrier in geotechnical and civil engineering applications.
Geotextile	Planar, polymeric (synthetic or natural) textile material, which may be woven, non-woven or knitted, used in geotechnical and civil engineering applications. Its functions include separation, protection reinforcement and filtration.
Grading	Distribution of particle sizes within a representative sample of a soil.
Granular soil	Soil formed from particles predominantly greater in size than 0.06 mm e.g. sand, gravel (*c.f.* cohesive soil).
Groundwater	All water which is below the surface of the ground in the saturation zone and in direct contact with the ground or subsoil.
HDPE	High density polyethylene. Resistant to chemicals typically found in leachate. Used to form geomembranes, geogrids, geotextiles etc.
Heterogeneous	A property within a medium (e.g. permeability) that varies with location in a diverse way.
Homogeneous	A property within a medium (e.g. permeability) that is the same at all locations.
Horizontal in situ stress	Stress acting within a medium in the horizontal direction (i.e. acting on a vertical plane). In a granular material (e.g. soil) it is a function of soil strength and vertical stress.
Hydrogeology	The study of the occurrence, movement and chemistry of groundwater in relation to the geologic environment
Hydraulic conductivity	Ability of a soil or rock to transmit water. The ratio of flux to hydraulic gradient. The higher the hydraulic conductivity, the greater the ability to transmit water.
Impermeable	Adjective used to indicate that a soil, rock, geomembrane etc. has a very low capacity to transmit fluid (i.e. having a very low permeability).

Index testing	Used to obtain parameters as an indication/inference of performance where performance testing is unfeasible or inappropriate.
Industrial waste	Non-hazardous solid waste generated as a result of industrial processes.
Inert waste	Waste which- (a) does not undergo any significant physical, chemical, or biological transformations; (b) does not dissolve, burn or otherwise physically or chemically react, biodegrade or adversely affect other matter with which it comes into contact in a way likely to give rise to environmental pollution or harm to human health; and (c) its total leachability and pollutant content and the ecotoxicity of its leachate are insignificant and, in particular, do not endanger the quality of any surface water or groundwater.
In-situ density	The density of a soil sample in the field.
Integrity	Wholeness, soundness (e.g. as applied to a landfill liner).
Interface friction	The frictional strength at an interface between two materials.
Interface shear strength	The shear strength (i.e. maximum shear stress) that can be mobilised at an interface between two materials. (e.g. between soil and a geosynthetic).
Internal strength	The strength between two materials bonded together (i.e. glued, stitched, heat bonded).
IPPC	Integrated Pollution Prevention and Control Directive (see PPC)
Isotropic	The property of a material (e.g. permeability) that is the same when measured in every direction.
Landfill gas	Any gas generated from landfilled waste.
Landfill Regulations	The Landfill (England and Wales) Regulations 2002 Statutory Instrument 2002 no. 1559.
Large strain shear strength	Ultimate (i.e. lowest) shear strength for a material or interface. Mobilised after large relative displacement (strain) on a slip zone or shear surface (see residual shear strength).
Lateral stiffness	Stiffness of a material in a horizontal direction.
LDPE	Low density polyethylene. Used in the manufacture of some geomembranes.
Leachate	Leachate is defined by the Landfill Regulations as any liquid percolating through the deposited waste and emitted from or contained within a landfill.
Leachate head	Hydraulic head (depth) of leachate acting at a point (e.g. on the basal liner). Note, this is not the same as hydraulic gradient.
Leachate well	Structure used to pump leachate from a landfill in order to control the leachate head on the liner.
LfD	European Council Directive 1999/31/EC of 26 April 1999 on the landfill of waste.
Lift	Term used to describe a layer of placed waste.

Limit equilibrium stability methods	Analysis method for assessing stability based on equating stabilising forces, or moments, acting on a defined body of soil with the destabilising forces, or moments, acting on the body of soil. Does not analyse magnitude of strains/movements.
Liner	A relatively thin structure of compacted natural clayey soil or manufactured material (e.g. geomembranes, geosynthetic clay liner) that serves as a barrier to control the migration of leachate or gas.
Linear regression	Method of obtaining the equation for a best-fit-straight-line through a series of data plotted in x, y space.
Lining system	A system typically comprising a series of layers that function as liner, protection layer, drainage layer and reinforcing layer.
LLDPE	Linear low density polyethylene. Used in the manufacture of flexible geomembranes.
Long-term	Used in soil mechanics this term means that sufficient time has elapsed to allow all transient pore pressures to dissipate, and hence the water pressures in the soil are in equilibrium with the external hydraulic boundary conditions (see also drained).
Mean	A number, or quantity, representative of a set of numbers, or quantities. Arithmetic mean – the sum of a series of values divided by the quantity of values.
Mechanical distortion	Change in the shape of a solid through the application of stress (force).
Method of slices	A technique used in the analysis of slope stability.
Moisture content	Ratio of the mass of water in a material to the mass of solids in the material.
Monte Carlo simulation	A probabilistic analysis technique used to consider possible outcomes for a problem from using input variables with statistical distributions.
Montmorillonite	A clay mineral that readily adsorbs water, causing very high susceptibility to expansion, swelling and shrinkage. The primary constituent of bentonite.
Municipal solid waste (MSW)	Waste from households as well as other waste which because of its nature or composition is similar to waste from households.
Needle punched	A method of entangling fibres in the manufacture of a geotextile by repeated penetration of a mat with barbed needles.
No-fines concrete	Concrete made with a mixture of cement and aggregate comprising only coarse granular particles, forming a material with high voids ratio and hence high permeability.
Non-woven	A method of forming a geotextile other than by weaving (e.g. needle punched and heat bonded geotextile are non-woven).
Normal stress	The stress applied perpendicular to a plane or surface.
Numerical modelling	An approach for assessing the behaviour of a zone of material, or a physical system, by using mathematical expressions to relate the stresses and strains in the materials with the applied forces.
Optimum water content	The moisture content at the peak dry density for a given compaction energy (value obtained from a Compaction Test).
Peak shear strength	The maximum shear stress that can resisted by a material.
Perched leachate	Leachate lying above a low permeability layer and separate from and above the leachate body immediately above the liner.

Perched water table	Groundwater lying above a low permeability layer and separate from and above another water table.
Performance testing	Testing conducted using site specific materials and boundary conditions in order to assess performance under in service conditions.
Permeability	The capacity of a porous medium to transmit a liquid or gas; hydraulic conductivity is permeability coefficient with respect to water.
Phreatic surface	The level of the water surface in an unconfined aquifer; see water table.
Piezometric surface	The notional surface formed from the pressure heads in a confined aquifer (i.e. the surface of zero pressure in the aquifer) that lies above the upper boundary of the aquifer. Note that artesian conditions exist where the piezometric surface lies above ground level).
Plane strain	A condition where the strain in one direction is zero. This condition is found perpendicular to a cross-section through a long uniform slope or retaining structure.
Plasticity	The ability to deform without cracking and rupturing. Used to describe cohesive soils.
Plastic limit	The lowest moisture content at which a soil can be deformed plastically (i.e. if the moisture content is below this value, when deformed the soil will crack and rupture).
Pore water pressure	Pressure of water in void spaces between soil particles.
PPC	The Pollution Prevention and Control Regulations 2000 no. 1973.
Pressure head	A measurement of pressure in a fluid system expressed as the height of an enclosed column of fluid which can be balanced by the pressure in the system.
Reinforced soil	Addition of members (e.g. geosynthetics) that have tensile strength. These interact with the soil through interface friction to produce a mass of soil with increased stability.
Residual shear strength	Ultimate (i.e. lowest) shear strength for a material or interface. Mobilised after large relative displacement (strain) on a slip zone or shear surface (see also large strain shear strength).
Revetment lining systems	Steep side slope lining system based on forming a stable structure on the quarry wall with an outer planar surface against which the liner can be placed (e.g. frame system).
Ring shear apparatus	A laboratory test device for measuring residual shear strength of cohesive soils and of some geosynthetic interfaces.
Saturation	The amount of moisture in the voids of a medium, equal to the volumetric moisture content divided by the porosity. The saturation ranges from 0 for a dry condition to 1 (or 100%) for a completely saturated condition.
Self-supporting liner	A lining system that is stable to the full height of the side slope without the presence of waste.
Sensitivity analysis	An evaluation conducted to assess the impact of changes in the values of specific parameters.
Seepage force	The force in a soil body resulting from the seepage of water.

Settlement, differential	Variation in settlement between two or more points.
Settlement, primary (waste)	Resulting from compression of waste due to a change in applied stress (mechanical compression). In a saturated waste deposit it also includes consolidation (a time dependent process).
Settlement, secondary (waste)	Resulting from a combination of time dependent creep under a constant applied stress and loss of volume due to waste degradation
Settlement, total	The sum of the primary and secondary settlement.
Shallow side slope	Classification relating to stability conditions defining a landfill side slope shallower than 30º from the horizontal.
Shearing rate	The rate at which shear strains are applied to a material (e.g. in a shear box test).
Shear modulus	A parameter used to define the stiffness of a material in shear.
Shear strength	Resistance of a material to formation of a shear (rupture) plane.
Shear surface	A surface along which there has been relative displacement, parallel to the surface, between the materials on either side of the surface. Also called failure surface.
Short-term	Used in soil mechanics this term means that insufficient time has elapsed to allow dissipation of any excess pore pressures, and hence the water pressures in the soil are not in equilibrium with the external hydraulic boundary conditions. (see also undrained).
Smooth geomembrane	A geomembrane with a planar smooth surface (i.e. as opposed to geomembranes that are textured to increase interface friction).
Staged construction	Construction of a system in a series of clearly defined phases (e.g. construction of a steep slope lining system in five 3 metre lifts to the height of 15 metres).
Standard deviation	A statistical measure of the spread of a set of values.
Steep side slope	Classification relating to stability conditions defining a landfill side slope steeper than 30º from the horizontal.
Stiffness	The resistance to deformation of a material (i.e. the ratio of a change in stress to a change in strain).
Stitch bonded	A method of connecting two or more pieces of geosynthetic together by stitching (e.g. stitched bonded GCLs rather than needle punched).
Strain compatibility	Materials experiencing the same strains in response to the application of a change in stress.
Strain hardening	Increase in shear strength accompanying increase in strain (work hardening).
Strain softening	Reduction in shear strength accompanying increase in strain after the peak shear strength has been achieved.
Strain softening interfaces	Interfaces that exhibit a reduction in shear stress for an increase in shear strain after the peak shear strength has been achieved (common to many soil/geosynthetic and geosynthetic/geosynthetic interfaces).
Stress history	The changes in stress that a body of soil has experienced in the period since formation.
Stress relief	A reduction in stress usually caused by excavation of overlying or laterally adjacent soil.

Sub-grade	Soil and rock beneath the lining system. This can be undisturbed in situ material or fill. Surface on which construction commences.
Submerged	Beneath a water table or surface of a body of water.
Subsidence	The process of settling.
Surface drainage	The overland movement of surface water.
Textured geomembrane	A geomembrane that has a roughened surface to increase its interface friction with adjacent materials (i.e. soil or geosynthetic). Texturing can be by one of several processes.
Tilting table	A laboratory test device used to measure geosynthetic vs. soil and geosynthetic vs. geosynthetic interface shear strengths at low normal stresses.
Topsoil	Uppermost layer of organic rich soil which is capable of supporting good plant growth.
Total stress	Sum of the effective stress (taken by the soil particles) and pore water pressure. Caused by the self weight of the soil, including the pore water, and external applied loads (e.g. foundations).
Triaxial compression test	A laboratory test method used to measure the shear strength of soil samples (see BS 1377).
Unconfined slope	A slope that does not have overlying material (e.g. no waste placed against it).
Under-drainage	Downward seepage of groundwater to an underlying permeable layer.
Undrained	Containing pore water pressures that are not in equilibrium with the boundary conditions. With time flow of water will occur into or out of the soil until the pore pressures are in equilibrium. The soil will then be in the drained state. (see also short-term).
Unit weight	Weight per unit volume (with this definition the use of the term weight means force).
Variability	A quantity or condition susceptible to fluctuations in value or magnitude.
Vertical compressibility	Change in vertical thickness of a layer of material in response to a change in applied vertical stress.
Vertical stress	Stress on a horizontal plane within a material. Calculated using the depth below ground surface, the unit weight of the materials overlying the plane and external loads.
VFPE	Very flexible polyethylene. Generic term sometimes used to describe flexible polyethylene geomembranes, e.g. LDPE, LLDPE and VLDPE.
VLDPE	Very low density polyethylene. Used in the manufacture of flexible geomembranes.
Voids ratio	Ratio between volume of voids in a material and volume of the solids forming a material. The higher the voids ratio the larger the voids in the material and hence greater the permeability.
Waste-supported liner	A steep slope lining system that is only stable at full height of the side slope following placement of waste against the lining system (i.e. without the waste the system will not be stable).
Water level	The measurement of the top of groundwater. The water level is reported as an elevation related to a datum to provide a common and comparative reference point.

Water table	The surface of underground, gravity controlled water; the surface of an unconfined aquifer at which pore water is at atmospheric pressure. It is generally located at the top of the zone of saturation in an unconfined aquifer.
Well graded	A term used to describe of the range of particle sizes present in a sample of soil. Well graded refers to their being particles of all sizes present in equal quantities between the largest and smallest sizes.
Wet of optimum	A soil compacted 'wet of optimum' is compacted at a water content higher than the optimum water content for the soil. See compaction test.
Woven	A method of manufacturing geotextiles by weaving (*c.f.* non woven, heat bonded, needle punched).
Young's modulus	A measure of stiffness obtained by relating a change in strain to a change in applied stress.

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