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

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Received 3 December 2004, revised 27 June 2005, accepted 16 September 2005

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

KEYWORDS: Geosynthetics, Landfill stability, Probability of failure, Reliability methods, Interface shear strength

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

1. INTRODUCTION

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

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

However, before design engineers can use reliability-based stability analysis, guidance is required on quantifying variability of interface shear strength and on use of outputs from such analyses, in conjunction with traditional factors of safety, in the decision-making process leading to design of stable slopes. This paper presents information on the variability of measured strengths obtained from a large data set for interfaces commonly encountered in landfill lining systems. Interfaces involving geosynthetic clay liners (GCL) are excluded from this paper as these have been considered in detail by McCartney *et al.* (2004) using a similar approach. The use of reliability assessment in landfill stability is demonstrated through consideration of two common landfill design cases: veneer stability and waste slope stability. Veneer stability has previously been used by Koerner and Koerner (2001) and McCartney *et al.* (2004), and waste slope stability by Sabatini *et al.* (2002), to demonstrate the sensitivity of landfill design to interface variability. These two design cases were selected for use in this study in order to add to the existing published information on relationships between probability of failure and traditional factors of safety. The aim is to produce a body of information that can be used by engineers to carry out and interpret reliability-based landfill designs.

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

2. STATISTICAL ANALYSIS OF INTERFACE STRENGTH VARIABILITY

2.1. General

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

$$X_k = X_m - 0.5\sigma_m \tag{1}$$

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

2.2. Derived interface shear strength parameters

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

As the quantification of interface shear strength requires two parameters (α and δ), variability of measured shear strengths requires consideration of linked pairs of these parameters. Dixon *et al.* (2002) proposed an approach based on calculating the variability of measured shear strengths for each normal stress and using these data to derive the appropriate shear strength parameters for use in design. For example, Figure 1 shows how characteristic values can be obtained for use in a limit equilibrium analysis.

2.3. Statistical data for measured interface shear strengths

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

A second approach is to carry out a limited number of tests to obtain site-specific strength values and to obtain information from the literature on possible variability for that specific type of interface. However, a limitation of this approach is that there is no information available to indicate whether the measured site-specific strengths are representative of mean values. If, in comparison with the estimated mean values (i.e. using data from previous tests on similar materials), the measured strengths are considered to be high, or there is limited experience of testing the interface, then further tests should be conducted and the first approach described above must be used. Design based wholly on literature values should not be attempted

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

$$V = \frac{\sigma_m}{X_m} \quad (2)$$

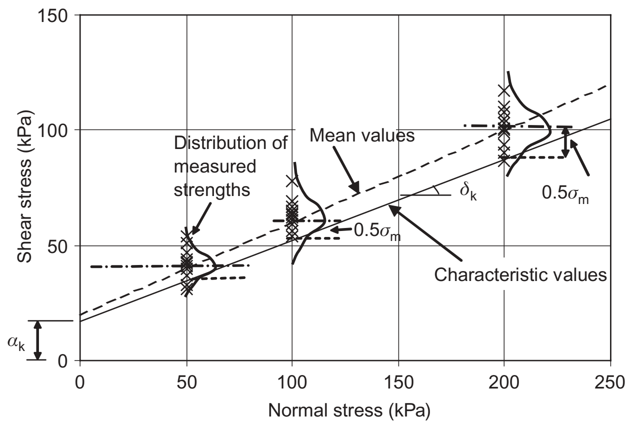

Figure 1. Derivation of interface shear strength parameters from measured shear strengths

3. VARIABILITY OF MEASURED INTERFACE SHEAR STRENGTH

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

3.1. Published information on variability of interface shear strength

The international literature contains many papers that report measured shear strengths for geosynthetic/geosynthetic and geosynthetic/soil interfaces. The best-controlled studies are those in which materials from one source (i.e. roll of geosynthetic and bulk sample of soil) have been used in direct shear tests repeated on one device, using the same test standard and carried out by the same personnel. Such studies have been reported by Dixon *et al.* (2000) for both smooth and textured geomembranes in contact with nonwoven geotextile tested at low normal stresses (i.e. appropriate for cap design), and by Criley and Saint John (1997) for both fine and coarse soils in contact with textured geomembrane.

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

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

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

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

3.2. Distribution of measured shear strengths with normal stress

The data presented in Figures 2 to 8 show a large variability in the number of tests and their distribution across the range of normal stresses. This is to be expected as the data sets are, in the main, compilations of tests conducted for different and specific purposes. However, despite this, there are sufficient data to demonstrate general trends. It was anticipated that data sets would show ranges of variability dependent upon the number of variables involved in testing (e.g. test equipment, personnel, test specification and material). For example, literature data sets would be expected to show greater variability than interlaboratory data sets for material from one source. However, the data do not show this trend (Figures 3 and 6). Apart from the repeatability results, the other data sets for a given interface (both peak and residual) define comparable ranges of shear strength with respect to normal stress. This is surprising, because it indicates that differences in measured strengths resulting from material variability (e.g. from type of texturing, type of soil/conditions etc.) represented in the literature and internal databases are of the same order as that resulting from carrying out tests on the same materials at different laboratories. It can be concluded that, for a given generic type of interface, test conditions have the most significant influence on observed variability of measured shear strengths.

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

3.3. Trends in variability of interface strength

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

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

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

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

[a]GM, geomembrane; [b]NW GT = nonwoven geotextile; [c]summary data not given owing to range of test conditions used.

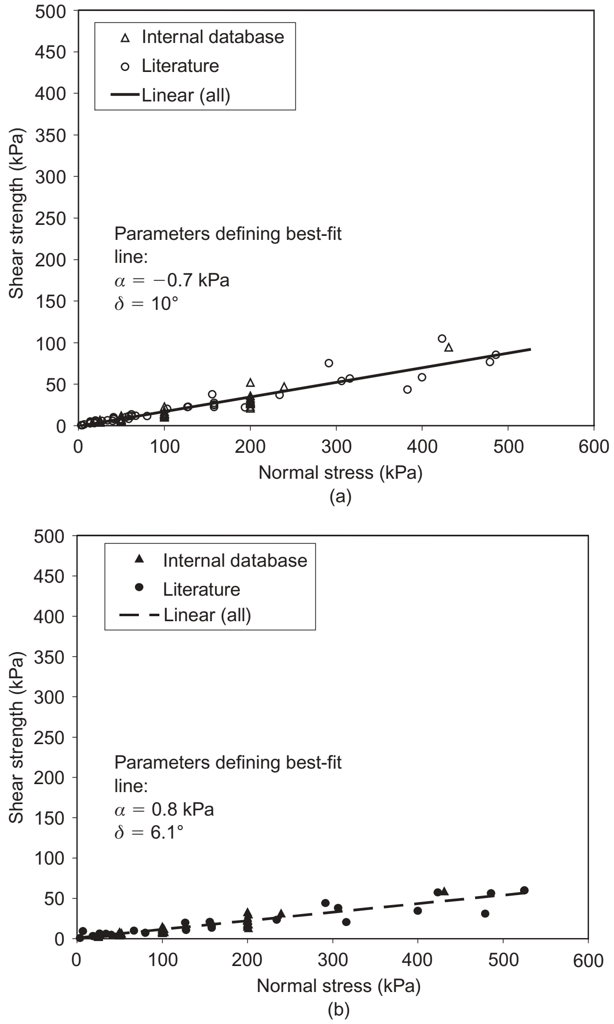

Figure 2. Shear strength against normal stress for smooth HDPE geomembrane/nonwoven geotextile from internal database and literature: (a) peak; (b) large displacement

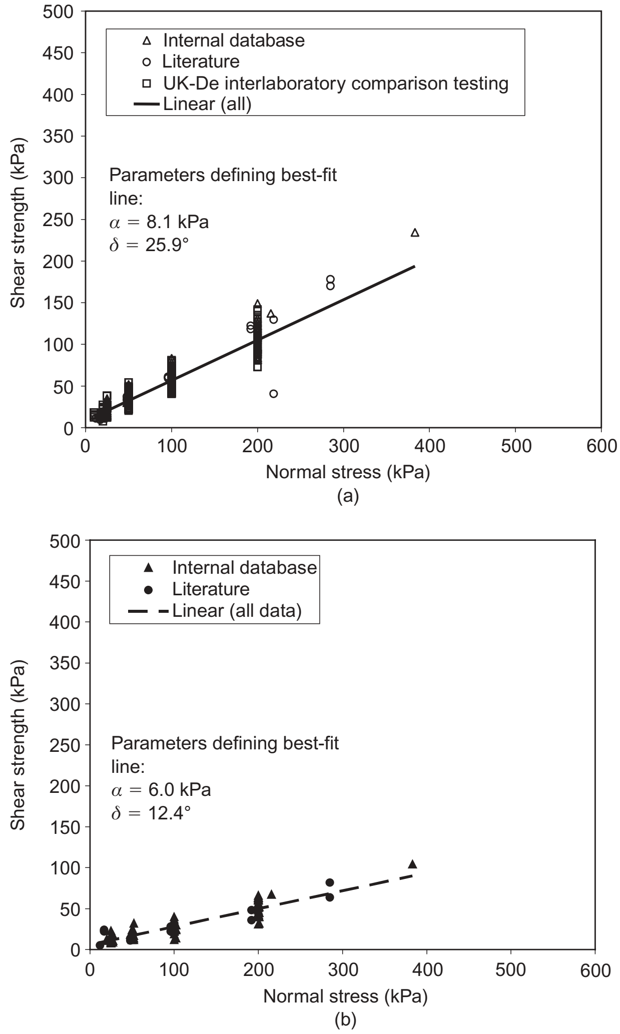

Figure 3. Shear strength against normal stress for textured HDPE geomembrane/nonwoven geotextile from internal database, interlaboratory comparison testing and literature: (a) peak; (b) large displacement

4. PROBABILITY OF FAILURE STABILITY ANALYSIS

4.1. Analysis method for probability of failure

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

4.2. Veneer stability

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

Effective stress analyses have been carried out for a 1.0 me thick soil veneer with porewater pressures on the interface calculated using a parallel submergence ratio (PSR) of 0.5. Slope angles (β) between 14° (1 in 4) and 33.7° (1 in 1.5) have been analysed. Only the variability of interface shear strength has been considered in these analyses; however, the method outlined by Duncan (2000)

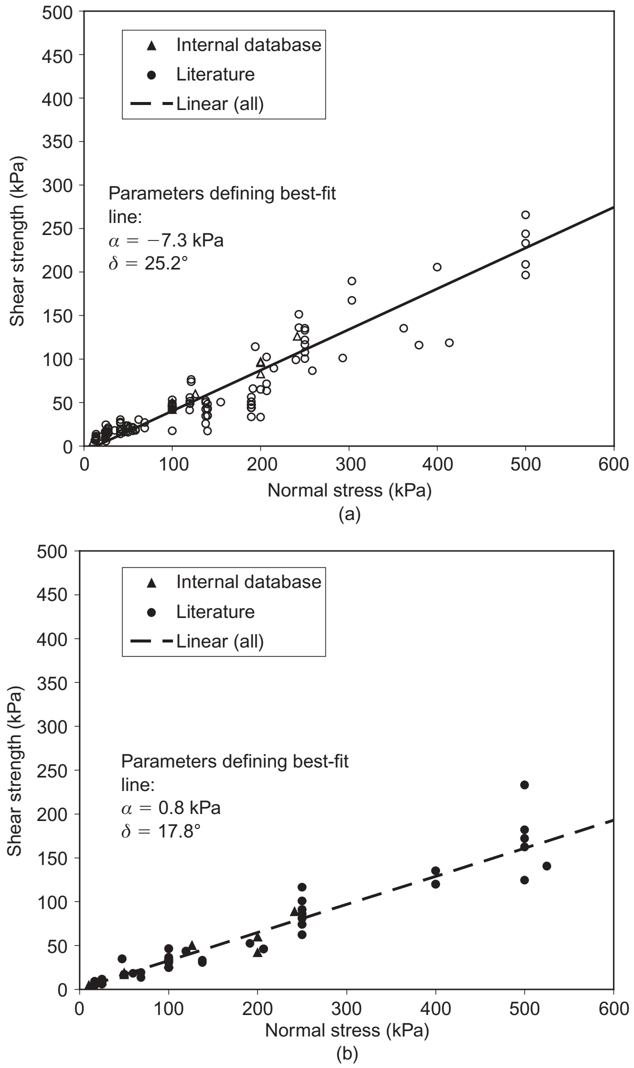

Figure 4. Shear strength against normal stress for smooth HDPE geomembrane/coarse soil from literature and internal database: (a) peak; (b) large displacement

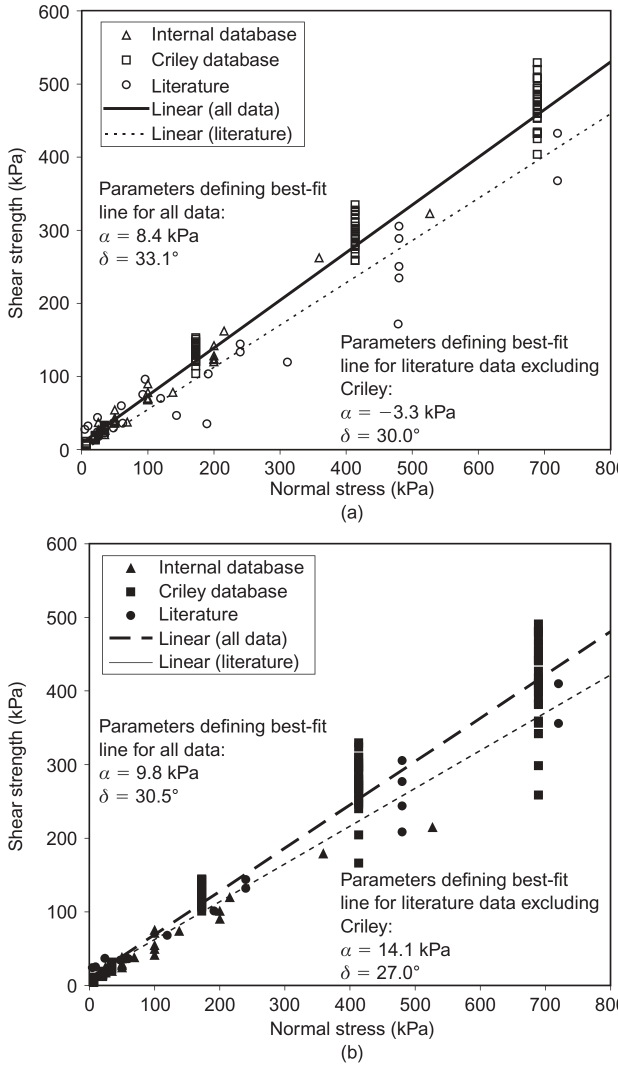

Figure 5. Shear strength against normal stress for textured HDPE geomembrane/coarse soil from literature, internal database, and Criley and Saint John (1997) repeatability results: (a) peak; (b) large displacement

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

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

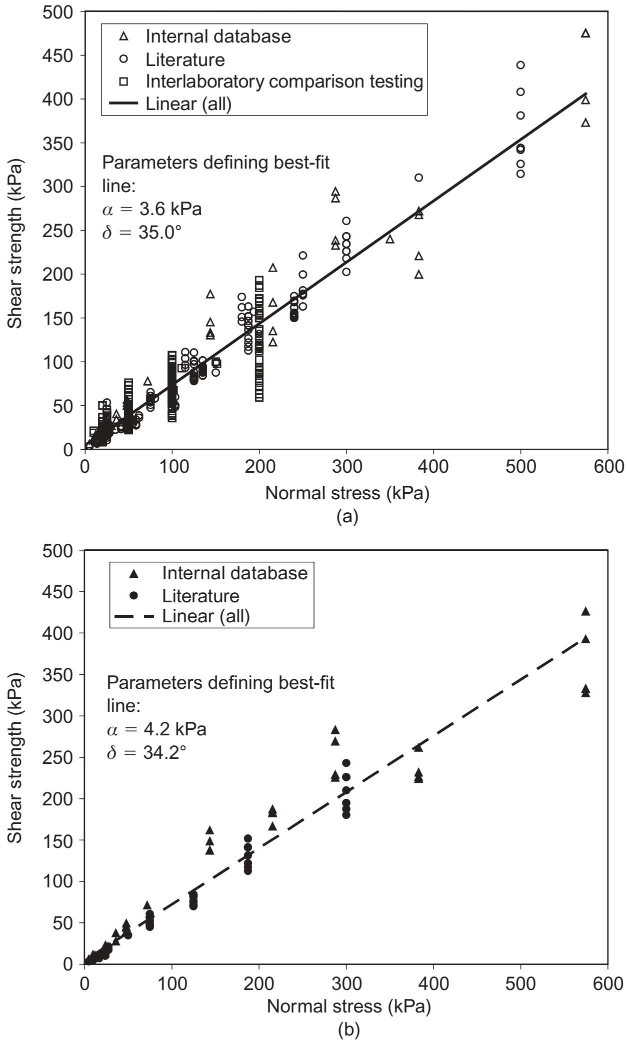

Figure 6. Shear strength against normal stress for nonwoven geotextile/coarse soil from literature, internal database and interlaboratory comparison testing: (a) peak; (b) large displacement

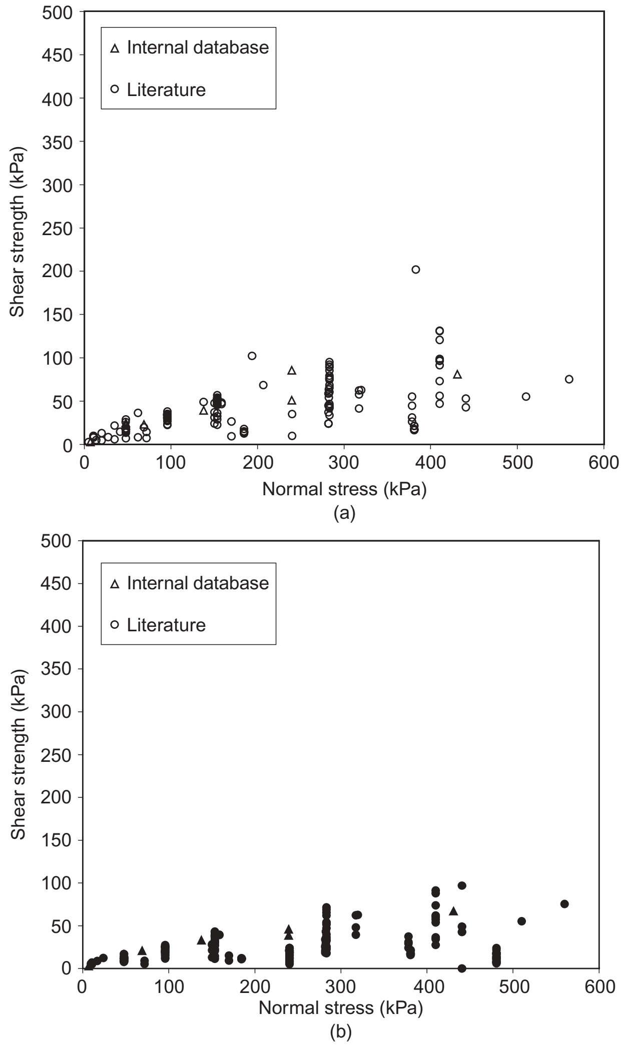

Figure 7. Shear strength against normal stress for smooth HDPE geomembrane/fine soil from literature and internal database: (a) peak; (b) large displacement

variability when compared with probability of failure values.

4.3. Waste body stability

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

Zero porewater pressures have been assumed on the interface owing to the presence of the drainage layer. Slope height has been varied between 30 and 60 m. Only the variability of interface shear strengths has been considered in this analysis. Sliding has been analysed for two interfaces: nonwoven geotextile/coarse soil and textured HDPE geomembrane/nonwoven geotextile. Each analysis has the same interface on the base and side slope. Mean peak shear strength parameters have been obtained from the best-fit lines calculated from combined data sets and shown in Figures 6a and 3a, respectively. The standard deviations of measured shear strengths have been taken from Figures 13b and 10b, respectively. Analyses have been carried out using the mean standard deviations of shear strength from combined data sets. There are currently no repeatability data sets available for these interfaces. Both mean and standard deviation values have been taken over the appropriate normal stress range for the problem (i.e. 100 to 300 kPa). Shear strength parameters (α and δ) for mean, $+1\sigma_m$ and $-1\sigma_m$ measured shear strengths have been calculated for each interface. Table 2 shows the shear strength input parameters for each interface, and Figure 18 shows plots of P_f against FS_k and FS_{MLV} for nonwoven geotextile/coarse soil and textured HDPE geomembrane/nonwoven geotextile interfaces.

For limit equilibrium analyses using mean shear strengths, FS_{MLV} values greater than 2.6 and 2.0 are

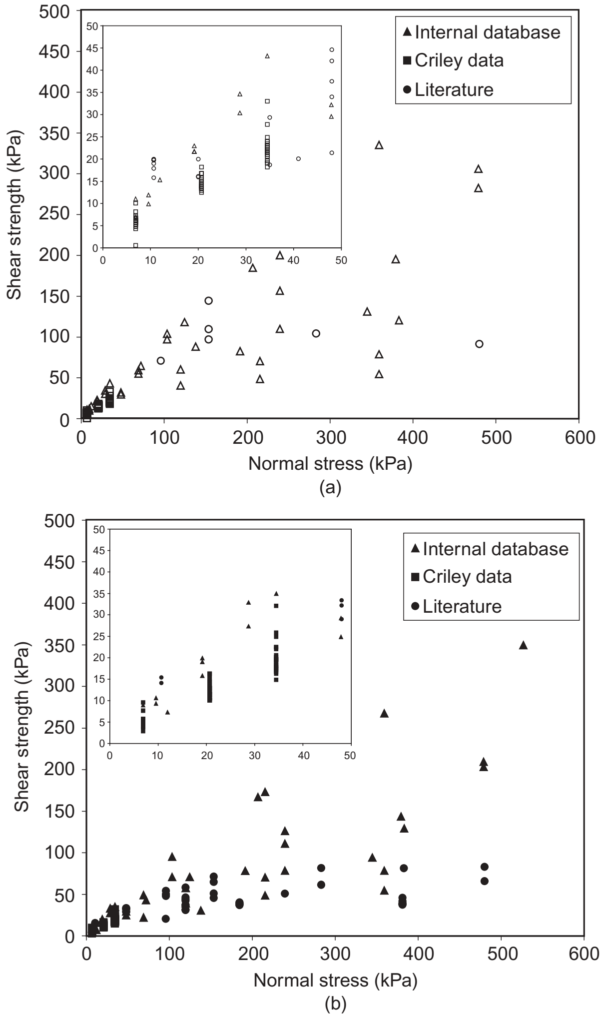

Figure 8. Shear strength against normal stress for textured HDPE geomembrane/fine soil from literature, internal database and Criley and Saint John (1997) repeatability results: (a) peak; (b) large displacement

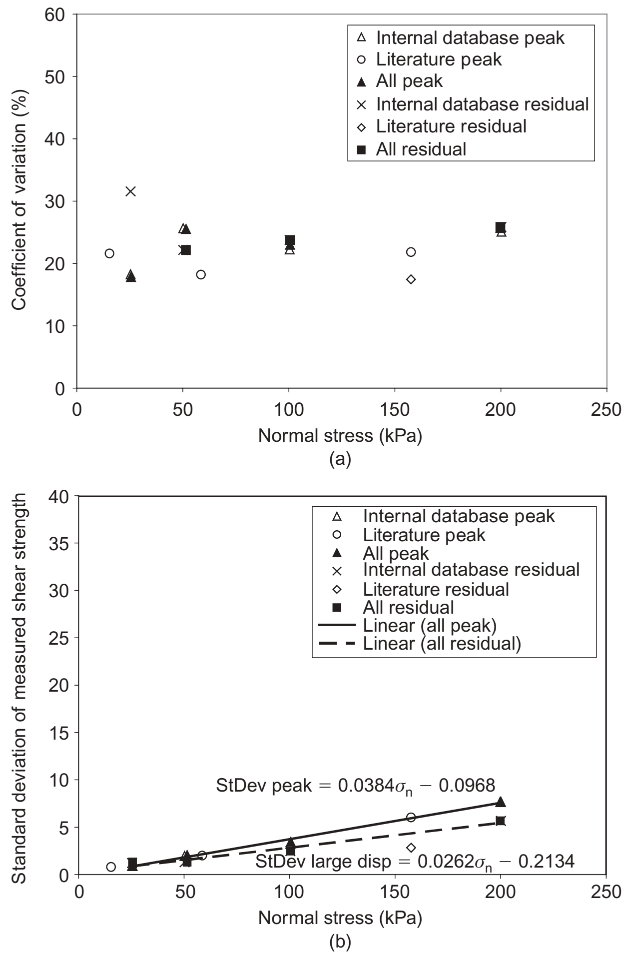

Figure 9. Smooth HDPE geomembrane/nonwoven geotextile from internal database, literature, and combined for peak and large displacement: (a) coefficient of variation against normal stress; (b) standard deviation of measured shear strength against normal stress

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

5. RELIABILITY OF LANDFILL STABILITY ANALYSIS

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

In order to enable probability of failure analysis to be used as a decision-making tool it is necessary to relate calculated values with consequences of failure, and hence to provide guidance on required values of P_f. Koerner and Koerner (2001) suggested boundary values based on the consequence of failure for a particular geosynthetic application being low, medium or serious. For barrier applications such as landfill lining systems Koerner and Koerner

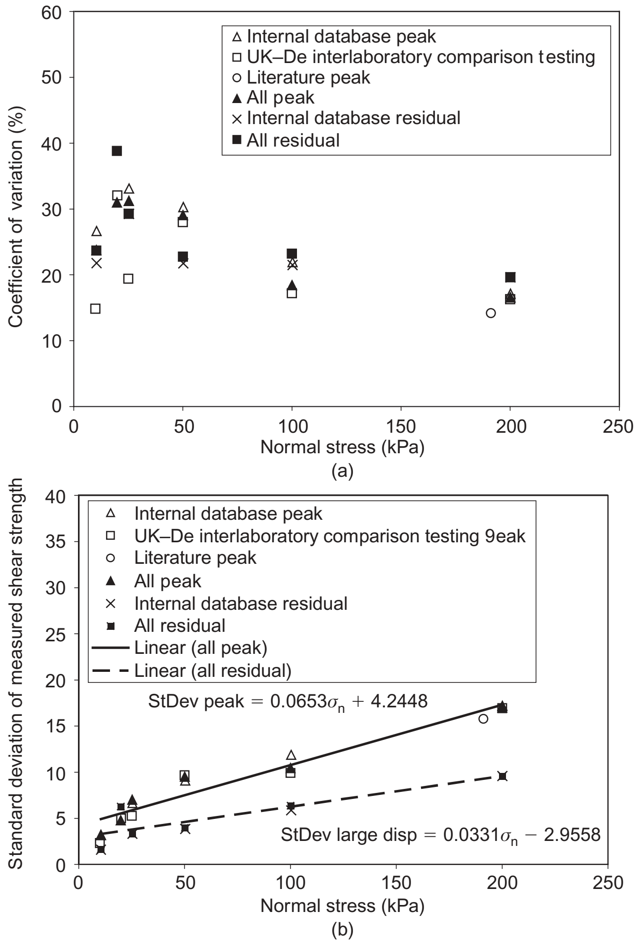

Figure 10. Textured HDPE geomembrane/nonwoven geotextile from internal database, literature, interlaboratory comparison tests and combined for peak and large displacement: (a) coefficient of variation against normal stress; (b) standard deviation of measured shear strength against normal stress

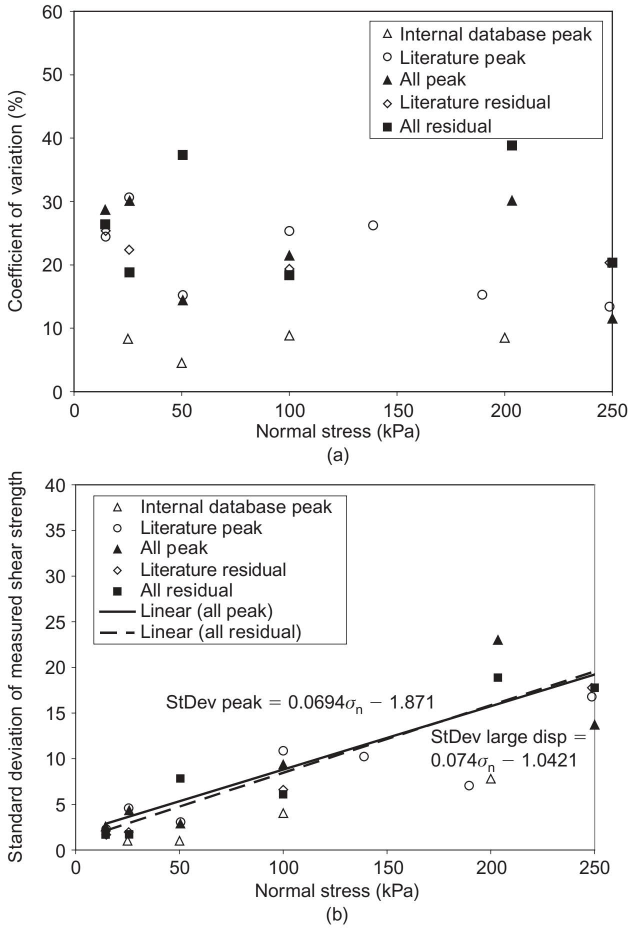

Figure 11. Smooth HDPE geomembrane/coarse soil from internal database, literature and combined for peak and large displacement: (a) coefficient of variation against normal stress; (b) standard deviation of measured shear strength against normal stress

(2001) proposed values of 0.3%, 0.05% and 0.01% for low medium and high consequences of failure, respectively. For landfill design, low consequence could relate to instability of a soil veneer during side slope construction (e.g. a drainage layer). This type of failure typically can be repaired at relatively low cost and does not result in any uncontrolled discharge of gas or leachate into the environment. Medium consequences could relate to capping failure and slippage of a temporary waste slope. Cost of repair may be higher than side slope veneer instability but is still low in relation to a serious failure. However, environmental damage could occur owing to escape of landfill gas. Serious consequence of failure could relate to slippage of a waste body that has an impact outside the site. This is likely to be disruptive to site operation and costly to repair, and can cause serious damage to the environment through pollution of groundwater by leachate and escape of landfill gas.

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

Higher P_f values could be considered appropriate for veneer stability analyses (i.e. 0.05% for capping failure and 0.3% for slide slope veneer failure). However, all of the analyses giving FS_{MLV} or $FS_k = 1.5$ have P_f values above these suggested boundary values, including analyses using Criley and Saint John (1997) repeatability data sets (Figures 15 and 16). This is a surprising result and indicates either poor current design practice or that the medium and low consequence acceptable values are too low. McCartney *et al.* (2004) reported factor of safety values corresponding to a P_f of 1% for GCL/textured HDPE geomembrane interfaces in an infinite slope veneer stability analysis with associated factors of safety calcu-

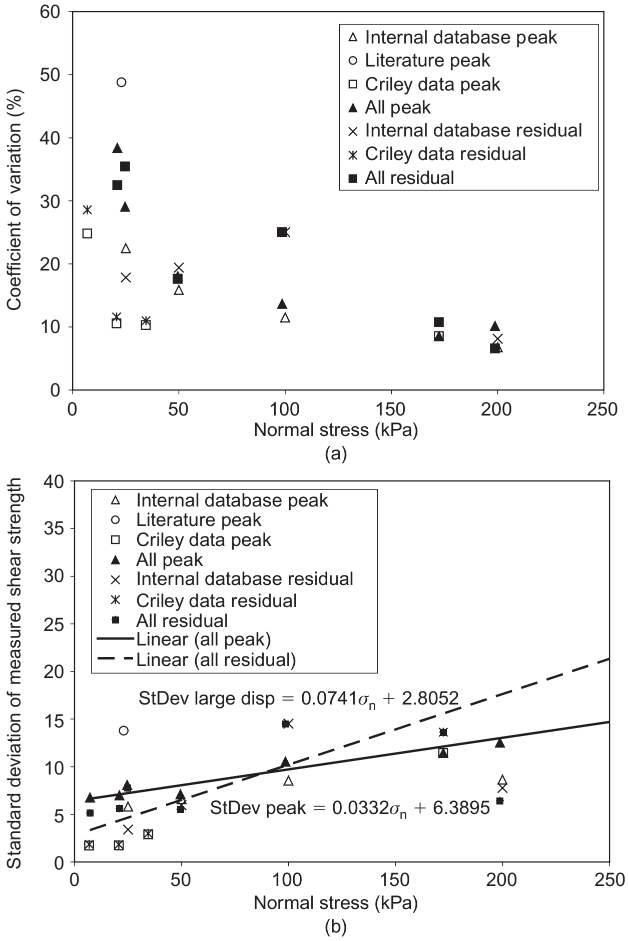

Figure 12. Textured HDPE geomembrane/coarse soil from internal database, literature, Criley and Saint John (1997) repeatability results and combined for peak and large displacement: (a) coefficient of variation against normal stress; (b) standard deviation of measured shear strength against normal stress

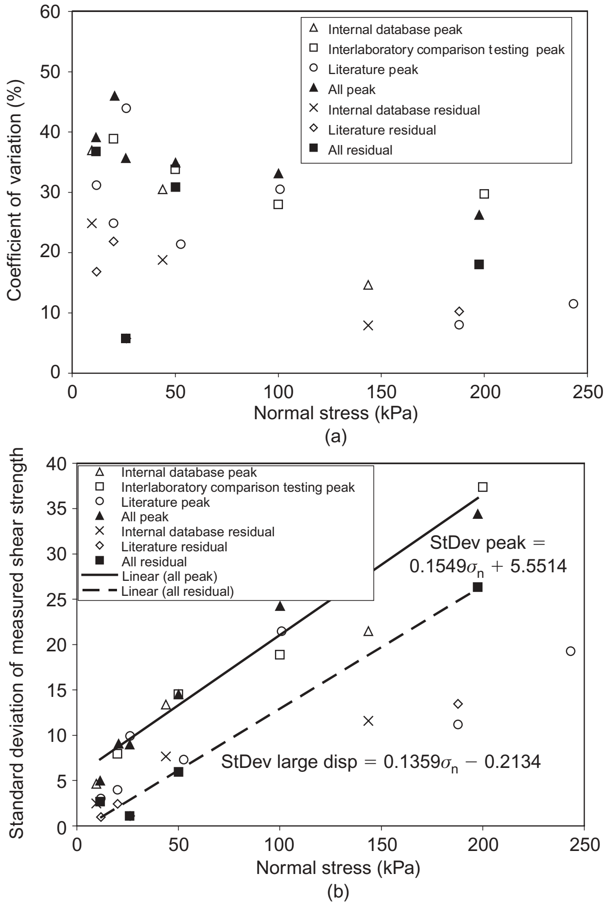

Figure 13. Nonwoven geotextile/coarse soil from literature, internal database, interlaboratory comparison testing and combined for peak and large displacement: (a) coefficient of variation against normal stress; (b) standard deviation of measured shear strength against normal stress

lated between 1.23 and 2.25 (depending upon the number of variables influencing the test results). They concluded that values of factor of safety associated with a $P_f = 1\%$ can be significantly greater for slopes incorporating GCL interfaces than the typical design target value of 1.5. The findings of the current study for a range of typical interfaces are consistent with the findings of McCartney *et al.* (2004).

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

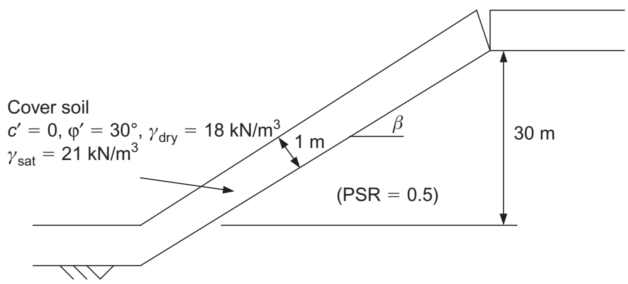

Figure 14. Diagram of the model used in the veneer stability analysis

6. CONCLUSIONS

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

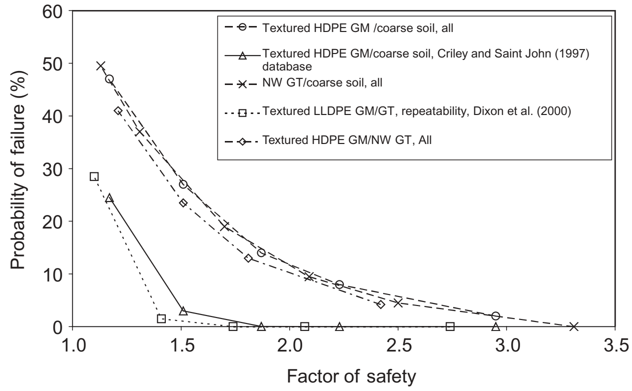

Figure 15. Probability of failure against factor of safety from veneer stability analysis, presenting data from combined data sets, Criley and Saint John (1997) and Dixon *et al.* (2000)

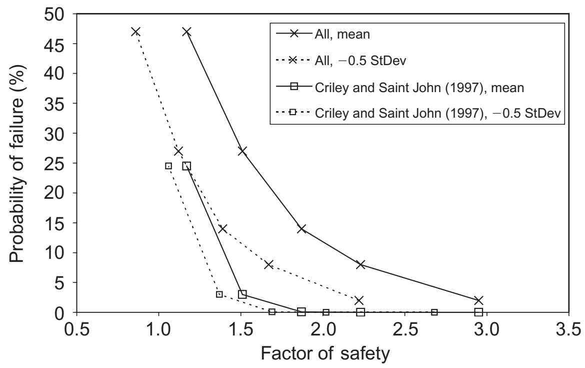

Figure 16. Probability of failure against factor of safety for veneer stability, showing the relationship between the mean and characteristic values for factor of safety, based on combined data and Criley and Saint John (1997) for textured HDPE geomembrane/coarse soil

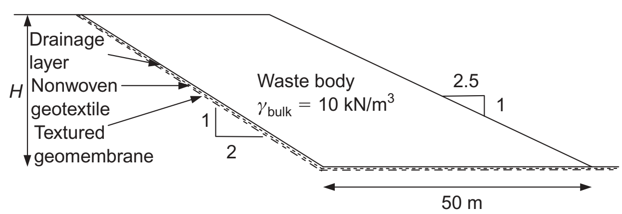

Figure 17. Diagram of the model used in the waste mass stability analysis.

shear strength. These can then be used to calculate shear strength parameters for use in stability assessments.

Current practice is to carry out a limit number of site-specific tests, but this provides insufficient information for the variability of interface strength to be considered in design. It is recommended that a sufficient number of site-specific direct shear interface tests be carried out to provide statistical data for use in traditional limit equilibrium analyses using characteristic values, and probability of failure analyses using the simple procedure described by Duncan (2000). In some cases, literature values are being used in lieu of site-specific test results, and this is considered be unacceptable and likely to lead to unreliable designs, as demonstrated by the analyses presented in this paper.

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

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

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

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

Table 2. Shear strength input parameters

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

[(a)]GM, geomembrane; [(b)]NW GT, nonwoven geotextile.

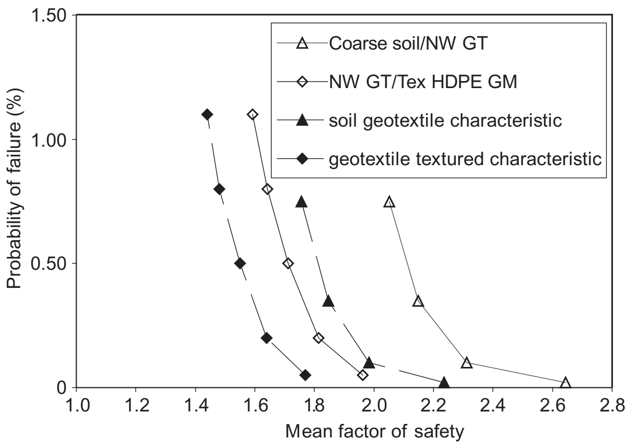

Figure 18. Probability of failure against factor of safety for waste body stability, showing the relationship between the mean and characteristic values for factor of safety, based on combined data

NOTATIONS

Basic SI units are given in parentheses.

FS_k	factor of safety using characteristic shear strengths (dimensionless)
FS_{MLV}	most likely (or traditional) value of factor of safety (dimensionless)
FS_i^+	factor of safety calculated with the specific variable (i.e. shear strength) increased by one standard deviation (dimensionless)
FS_i^-	factor of safety calculated with the specific variable (i.e. shear strength) decreased by one standard deviation (dimensionless)
P_f	probability of failure (dimensionless)
PSR	parallel submergence ratio (dimensionless)
V	coefficient of variation (dimensionless)
X_k	characteristic value (dimensions depending on parameter)
X_m	mean value (dimensions depending on parameter)
α	apparent adhesion defining Coulomb failure envelope for interface shear strength (Pa)
β	slope angle (degrees)
δ	slope angle defining Coulomb failure envelope for interface shear strength (degrees)
σ_m	standard deviation of measured value (dimensions depending on parameter)
σ_{MLV}	standard deviation of FS_{MLV} (dimensionless)
ΔFS_i	$FS_i^+ - FS_i^-$ for each variable (dimensionless)

Subscripts

k	characteristic value
p	peak
r	residual
+, −	plus and minus one standard deviation

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APPENDIX

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

Step 1 Assemble the mean value and standard deviations of the major variables that are to be used in the design method.

Step 2 Calculate the most likely value of factor of safety (FS_{MLV}) using the mean values (i.e. following standard design methods).

Step 3 Calculate the standard deviation (σ_{MLV}) and coefficient of variation (V_{MLV}) of the FS_{MLV} using the standard deviation of all the major design variables.

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

$$V_{MLV} = \frac{\sigma_{MLV}}{F_{MLV}} \tag{4}$$

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

Step 4 Using the values of F_{MLV} and V_{MLV}, determine the probability of failure (P_f) using Koerner and Koerner (2001 Table 1), which shows the probabilities that the factor of safety (FS_{MLV}) is smaller than 1.0 based on a lognormal distribution for the factor of safety. Alternatively, the analytical approach given by Duncan (2000) could be used.

Step 5 Assess the calculated factor of safety in respect of the P_f value. A P_f of 0% means there is no likelihood of failure.

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