Landfill Liner Interface Strengths from Torsional-Ring-Shear Tests

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Abstract: A torsional-ring-shear apparatus and test procedure are described for measuring soil/geosynthetic and geosynthetic/geosynthetic interface strengths. Typical interface strengths are presented for a double-composite liner system and the relevancy of ring-shear strengths is illustrated using the slope failure at the Kettleman Hills Waste Repository, Kettleman City, Calif. The results of undrained ring-shear tests show that for a clay/geomembrane interface: (1) Interface strength depends on plasticity and compaction water content of the clay, and the applied normal stress; (2) interface strengths measured with the torsional-ring-shear apparatus are in excellent agreement with back-calculated field strengths; and (3) peak and residual interface failure envelopes are nonlinear, and the nonlinearity should be modeled in stability analyses instead of as a combination of cohesion and friction angle. Design recommendations for interface strengths and stability analyses are also presented.

INTRODUCTION

Hazardous-waste landfills and new municipal solid-waste landfills in this country are required to have a low-hydraulic-conductivity liner and a drainage system consisting of compacted clay and geosynthetic materials. The stability of these systems is controlled by the shear strength of each component and the various component interfaces. The importance of interface strengths was illustrated by the slope-stability failure in Phase IA of Landfill B-19 at the Kettleman Hills Class I hazardous-waste treatment-and-storage facility in Kettleman City, Calif. The landfill area is an oval-shaped bowl carved into an existing valley to a depth of approximately 30 m and covering an area of 120,000 m^2. A slope-stability failure occurred during filling on 19 March 1988 that resulted in 11 m lateral displacements of the waste fill and 4.3 m vertical settlements. Byrne et al. (1992) concluded that sliding primarily occurred along the 1.1-m-thick secondary clay liner/secondary high-density polyethylene (HDPE) geomembrane interface in the double-composite liner system (Fig. 1). As a result, the majority of the ring-shear tests described here focus on the secondary clay liner/secondary HDPE geomembrane (SC/SG) interface. This paper reviews current interface test procedures, describes the torsional-ring-shear apparatus and test procedure used to measure interface strengths of various soil/geosynthetic and geosynthetic/geosynthetic interfaces, presents some typical ring-shear test results, and illustrates the relevancy of ring-shear interface-strengths using the Kettleman Hills case history.

REVIEW OF DIRECT-SHEAR INTERFACE TESTS

Direct-shear and pullout tests have been widely used to measure liner interface strengths (Martin et al. 1984; Saxena and Wong 1984; Koerner

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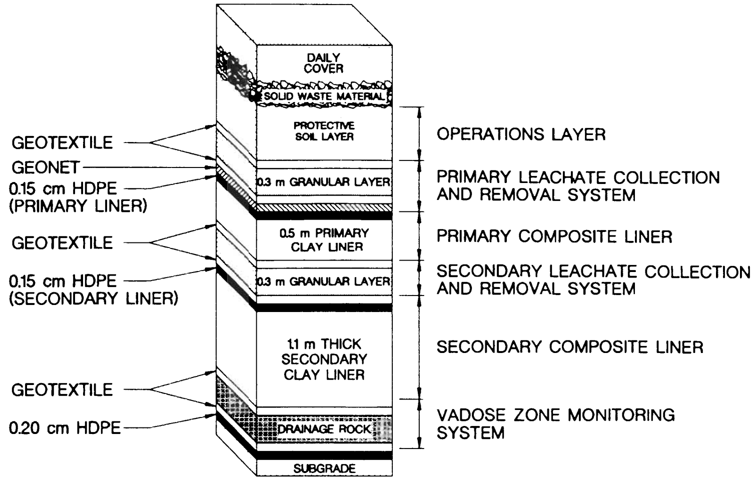

FIG. 1. Composite Double-Liner System at Base of Landfill (from Mitchell et al. 1990)

et al. 1986; Williams and Houlihan 1986; Seed et al. 1988; Giroud and Beech 1989; O'Rourke et al. 1990; Bove 1990; Seed and Boulanger 1991; Takasumi et al. 1991). Conventional direct-shear specimens range in size from 7 cm × 7 cm to 10 cm × 10 cm. Therefore, the specimen is subjected to only 0.6–0.8 cm of continuous displacement. Since the peak interface-strength is usually mobilized at a shear displacement of less than 0.5 cm, conventional direct-shear tests provide a good estimate of the peak strength. However, a shear displacement of 40–60 cm is typically required to mobilize a residual interface-strength in the ring-shear tests described here. To achieve these large displacements, the shear box must be reversed a number of times. These reversals do not apply continuous displacement in one direction, and thus do not simulate field shearing conditions that lead to a residual strength-condition.

In an effort to increase the magnitude of continuous-shear displacement, larger direct-shear boxes have been developed. These large-scale shear boxes range in size from 30 cm × 30 cm to 28 cm × 43 cm. ASTM Test Standard D5321 ("Determining" 1992) requires a shear box with a minimum dimension of 30 cm to be used to measure the coefficient of soil/geosynthetic or geosynthetic/geosynthetic friction. In this test, a geosynthetic is secured with glue to a horizontal substrate (e.g., plywood) and the shear box containing soil or a geosynthetic is moved along the substrate for a displacement of 2.5–7.5 cm. As a result, virgin geosynthetic material is sheared along the bottom substrate instead of being sheared along the same interface. Other disadvantages of 30 cm × 30 cm direct-shear boxes include the lack of vertical displacement information during consolidation or shear; the cost of the apparatus; and the need to secure large geosynthetics, compact large soil-specimens, and apply large normal forces.

Fig. 2 presents typical shear stress-displacement relationships from large-scale (30 cm × 30 cm) direct-shear tests on the SC/SG interface from landfill B-19 at Kettleman Hills (Byrne et al. 1992; "Draft" 1991). The water content of the clay specimens ranged from 30.0% to 30.6%. The tests were stopped

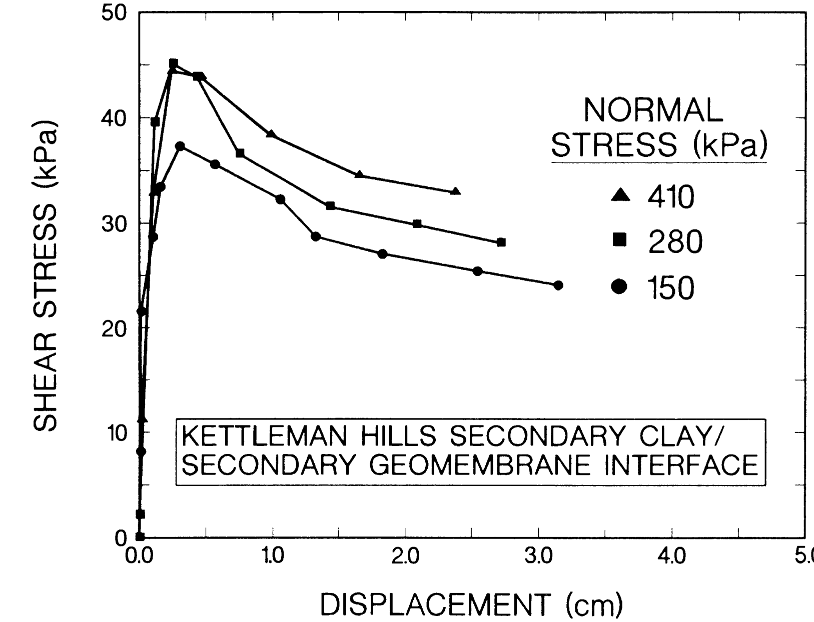

FIG. 2. Large-Scale Direct-Shear Tests Results for Secondary Clay/Secondary Geomembrane Interface (after Byrne et al. 1992)

at a shear displacement of approximately 2.5–3.0 cm, even though the shear-stress/horizontal-displacement relationships had not reached a minimum or residual strength condition. Therefore, large-scale direct-shear boxes appear applicable to the measurement of the peak interface strength, however, the limited shear displacement restricts their use in measuring the residual interface-strength.

Due to the limitations of current direct-shear testing, the suitability of a torsional-ring-shear apparatus to measure peak and residual interface-strengths was investigated. Negussey et al. (1989) also used a torsional-ring-shear apparatus to measure the strength of sand/geomembrane and nonwoven geotextile/HDPE geomembrane interfaces.

TORSIONAL-RING-SHEAR APPARATUS

The main advantage of the torsional-ring-shear apparatus is that unlimited continuous-shear displacement can be applied in one direction to achieve a residual strength condition. Other advantages of the ring-shear apparatus include continuous-shear displacement along the same interface, a constant cross-sectional area during shear, minimal laboratory supervision (because there is no reversal of the shear box), applicability of data acquisition techniques, and a smaller specimen, which allows greater control of compaction conditions and the securing of the geosynthetics. However, the small specimen size is also a disadvantage for materials that are significantly anisotropic. Larger ring-shear specimen containers are being developed to investigate scale effects of some interfaces.

The Bromhead ring-shear apparatus is based on the original design developed by Bromhead (1979) and manufactured by Wykeham-Farrance Ltd., Slough, England. The ring-shear specimen is annular with an inside diameter of 7 cm and an outside diameter of 10 cm. In the original apparatus, drainage is provided by two knurled, bronze, porous stones screwed to the bottom of the specimen container and to the top loading platen. The specimen is confined radially by the 0.5-cm-deep specimen container.

The Bromhead ring-shear apparatus was modified to facilitate testing of soil/geosynthetic and geosynthetic/geosynthetic interfaces. A specimen container that could accommodate a 1-cm-deep specimen was fabricated, and a Lucite ring was used to facilitate securing geosynthetics to the top platen with glue. In tests on soil/geosynthetic interfaces, the bottom porous stone was replaced with a knurled stainless-steel ring to minimize drainage. In tests on geosynthetic/geosynthetic interfaces, the bottom, bronze, porous stone was replaced by another Lucite ring to aid in securing the bottom geosynthetic with glue. Digital dial gages and a microcomputer data acquisition system were used to monitor vertical displacement and shear stress during the test. Average horizontal displacements were calculated by multiplying the average circumference of the specimen by the measured angular displacement.

RING-SHEAR TEST PROCEDURE AND SPECIMEN PREPARATION

Based on water contents of the secondary clay liner, which was measured during placement and after the slope failure, and based on the determination that the dissipation of excess pore-water pressures in the vicinity of the secondary-clay/secondary-geomembrane interface would be negligible prior to failure, the unconsolidated-undrained shear strength of the secondary clay/secondary geomembrane interface was determined to be representative of the Kettleman Hills field conditions (Mitchell et al. 1990; Byrne et al. 1992). As a result, an unconsolidated-undrained test procedure was used by Mitchell et al. (1990) and in "Draft" (1991) for direct-shear tests on the SC/SG interface. For consistency, an unconsolidated-undrained test procedure was used for the ring-shear tests performed on the SC/SG interface. Since the ring-shear specimen is not enclosed in a membrane, a displacement rate of 4.4 cm/min was applied to obtain an undrained condition in the ring-shear apparatus. This displacement rate is based on the coefficient of consolidation reported by Seed et al. (1988) of 3.5×10^{-3} cm^2/min, the procedure developed by Gibson and Henkel (1954), and a degree of consolidation of 0%. It is faster than the displacement rate in previous Kettleman Hills testing (Mitchell et al. 1990; "Draft" 1991), because the ring-shear specimen is thinner than previous direct-shear specimens.

All geosynthetic/geosynthetic interfaces were sheared immediately after application of the normal stress at a displacement rate of 0.1 cm/min. This displacement rate is in agreement with direct-shear testing performed by Mitchell et al. (1990) and "Draft" (1991), in which displacement rates of 0.01 cm/min to 0.1 cm/min, and 0.1 cm/min, respectively, were applied.

The secondary clay liner at Kettleman Hills consists of on-site claystone, siltstone, and sandstone with 2–5% sodium bentonite by weight added to decrease hydraulic conductivity. The clay-liner sample used in the ring-shear tests was obtained from on-site stockpiles during construction of the liner. The secondary clay classifies as a high-plasticity clay (CH) according to the Unified Soil Classification System with a liquid limit of 65% and a plasticity index of 44. Hydrometer tests conducted during this study revealed that

84% of the clay liner passes U.S. Standard sieve No. 200 and the clay-size fraction (percent by weight finer than 0.002 mm) is 49%. Byrne et al. (1992) reported that the as-built secondary clay liner at Kettleman Hills had a liquid limit of 60–70% and a plasticity index of 40–50; thus, the soil tested appears representative of average field conditions. Standard Proctor compaction tests revealed that the optimum water content of the secondary clay liner from Kettleman Hills is 22% and the maximum dry-unit weight is 15.6 kN/m^3. Quality-assurance records show that the secondary clay layer was placed at water contents from 27% to 33% with a median value of 29.7% (Byrne et al. 1992).

The ring-shear specimens were obtained by air drying a portion of the 18 kg sample of the secondary clay liner from landfill B-19. The air-dry soil was crushed with a ceramic pestle and processed through U.S. Standard sieve No. 40. The clay was mixed to the desired water content using distilled water. The ring-shear specimens were compacted directly into the ring-shear specimen container using a Harvard miniature compactor. The desired dry-unit weight was obtained by compacting the appropriate weight of moist soil into the specimen container using two lifts. After compaction, the top platen with a secured geosynthetic was placed on the compacted clay, and the specimen container was installed in the ring-shear apparatus. The desired normal stress was applied within 5 min and shearing commenced after the normal stress was obtained. The observations were made to ensure that specimen extrusion did not occur during application of the normal stress. The specimen was not inundated prior to or during shear. Moist cotton batting, however, was placed around the top platen to minimize changes in moisture content during testing. In accordance with field water contents and dry-unit weights, the ring-shear specimens were compacted to dry-unit weights of 14.5–15.4 kN/m^3 and initial water contents of 18–33% to investigate the effect of compaction water content on interface-shear strength.

The geosynthetics were glued to the Lucite ring using a thin coat of epoxy cement and allowed to cure for 24 h under a normal stress of 300 kPa. This aided bonding of the geosynthetics and minimized vertical displacements due to the glue during testing. The specimen container and geosynthetic were always marked to ensure that the geosynthetic did not slip during shear. The surface of the geomembrane was also wiped, if necessary, using a paper towel to minimize the effect of fingerprints and perspiration on the interface strengths (Yegian and Lahlaf 1992). The ring-shear tests were conducted at a laboratory temperature of 20°C.

The following interfaces were tested during this investigation: (1) Secondary clay/secondary HDPE geomembrane; (2) geonet/HDPE geomembrane; and (3) nonwoven geotextile/HDPE geomembrane. The other interfaces shown in Fig. 1 (i.e., geonet/nonwoven geotextile and nonwoven geotextile/secondary clay) were not tested, because Seed et al. (1988) concluded that these interfaces exhibited a large interface shear strength. The geomembrane used in the present study is a smooth 1.5-mm-thick HDPE liner (Gundle Lining Systems, Inc., Houston, Tex.). The HDPE drainage geonet is the Gundle Gundnet XL4 net with a thickness of 5 mm. The nonwoven geotextile used in the present study is Polyfelt TS 600, which is a nonwoven needle-punched geotextile made of polypropylene fibers. These geosynthetics are compatible with the geosynthetics used in the double-composite liner system at Kettleman Hills Landfill B-19.

RING-SHEAR TEST RESULTS ON CLAY/GEOMEMBRANE INTERFACE

Fig. 3 presents a typical shear-stress/horizontal-displacement relationship for an unconsolidated-undrained ring-shear test on the SC/SG interface at a normal stress of 48 kPa. The final water content of the clay was approximately 32.5%. The interface exhibits a peak shear strength of approximately 13.5 kPa and a residual strength of about 8.5 kPa. The peak SC/SG-interface strength was usually mobilized at a shear displacement of 0.2–0.4 cm (Fig. 4). This is in good agreement with the large-scale direct-shear test results in Fig. 2 that indicate the peak interface strength is mobilized at a shear displacement of 0.2–0.3 cm. Fig. 3 also shows that the residual interface strength is reached at a horizontal displacement of approximately 60 cm. However, the majority of the strength loss occurs within a horizontal displacement of 35–40 cm. This displacement is larger than the 2.5–5 cm that is usually applied in a large-scale direct-shear apparatus. In addition, the specimen underwent a vertical displacement of less than 0.01 cm, less than 1% of the initial height, during undrained shearing. This small vertical deformation is probably due to a slight extrusion of soil during undrained shearing.

La Gatta (1970) recommended plotting the shear-stress/horizontal-displacement relationship from ring-shear tests using the logarithm of horizontal displacement. This plotting technique accentuates the slope of the shear stress-displacement curve at large deformations, allowing the horizontal portion of the curve to be clearly defined. Fig. 4 presents a semilogarithmic representation of the ring-shear test on the SC/SG interface presented in Fig. 3. The shear stress has reached a constant or residual value using a logarithmic scale. Therefore, to ensure that a residual strength condition is reached before a ring-shear or direct-shear test is stopped, it is recommended that the shear stress be plotted using the logarithm of horizontal displacement. Once the shear stress becomes essentially constant on a semilogarithmic plot, the test can be stopped.

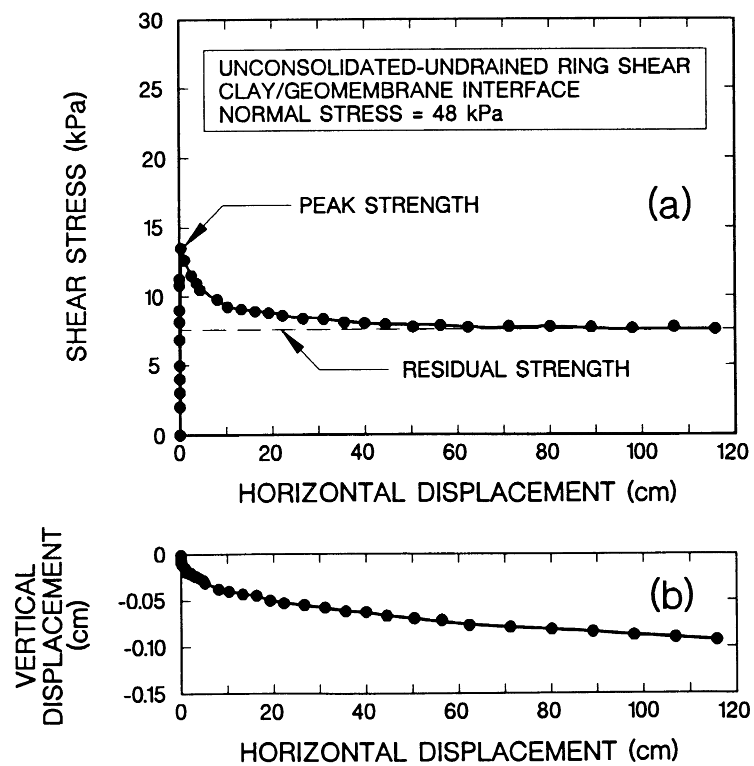

FIG. 3. Typical Undrained Ring-Shear Test Results for Secondary Clay/Secondary Geomembrane Interface: (*a*) Shear Stress; (*b*) Vertical Displacement

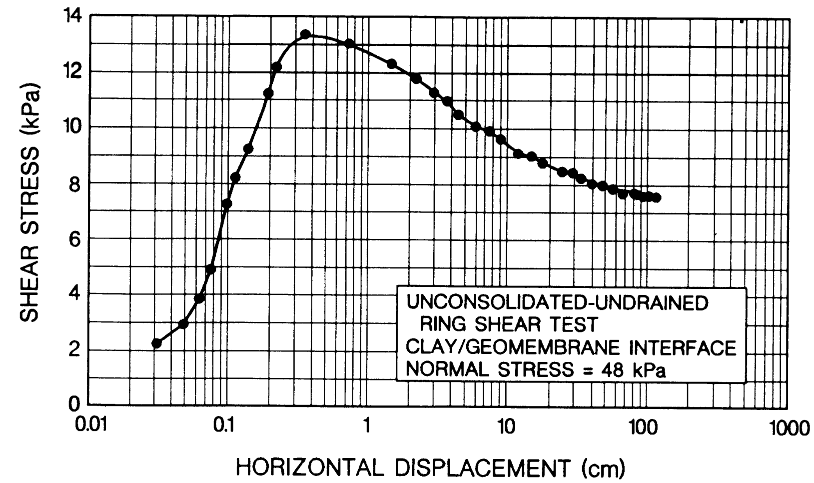

FIG. 4. Semilogarithmic Presentation of Secondary Clay/Secondary Geomembrane Ring-Shear Interface Test

Fig. 5 presents the peak SC/SG-interface strengths for the range of normal stresses and final water contents considered by Mitchell et al. (1990) and Byrne et al. (1992). At low normal stresses the peak interface strength is slightly influenced by final water content. However, as the normal stress increases, the peak interface strength becomes sensitive to compaction water content. The optimum water content of the Kettleman Hills secondary clay liner is 22%. Therefore, at placement water contents of 31–32% (9–10% above optimum), the peak interface strength converges to a range of 15–35 kPa. This is in good agreement with research on compacted cohesive soils presented by Seed et al. (1960). In summary, near the optimum water content the clay is stiffer and has more frictional resistance, which results in higher peak interface strengths. As the water content increases on the wet side of optimum, the peak interface strength decreases.

Fig. 6 presents the residual SC/SG-interface strengths for the same range of normal stresses and final water contents as Fig. 5. At low normal stresses, the residual interface strength is slightly influenced by final water content. As the normal stress increases, however, the residual interface strength becomes sensitive to compaction water content. At a water content of 31–32% the residual interface strength converges to a range of 8–24 kPa. These residual interface strengths are 30–50% lower than the peak interface strengths presented in Fig. 5.

Byrne et al. (1992) reported that the median-placed moisture content of

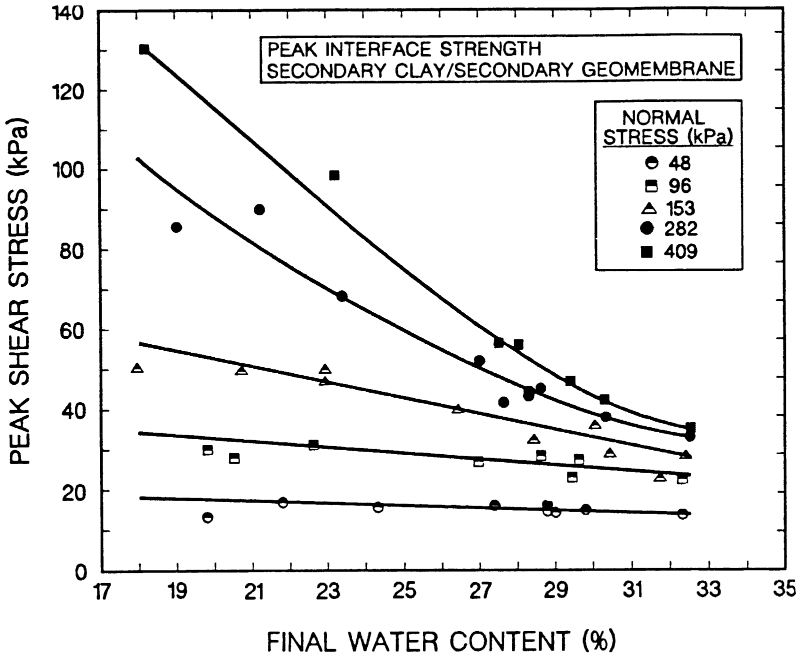

FIG. 5. Peak Secondary Clay/Secondary Geomembrane Interface Strengths

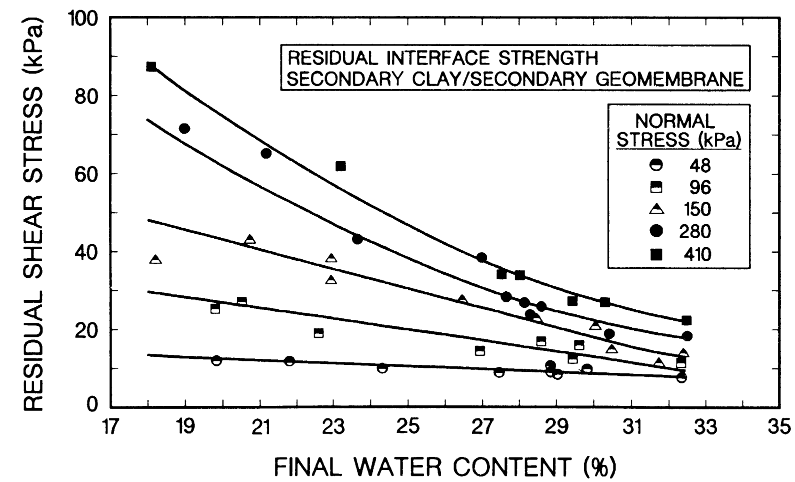

FIG. 6. Residual Secondary Clay/Secondary Geomembrane Interface Strengths

the secondary clay liner at Kettleman Hills was 29.7%. Therefore, peak and residual failure envelopes (Fig. 7) were obtained using Figs. 5 and 6 and a water content of 29.7%. Since ring-shear tests were not conducted at a water content of exactly 29.7%, data points are not shown on the ring-shear failure envelopes. Also the peak and residual ring-shear failure en-

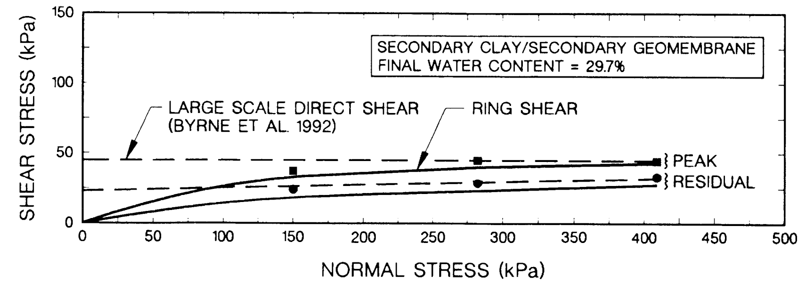

FIG. 7. Undrained Secondary Clay/Secondary Geomembrane Interface Failure Envelopes

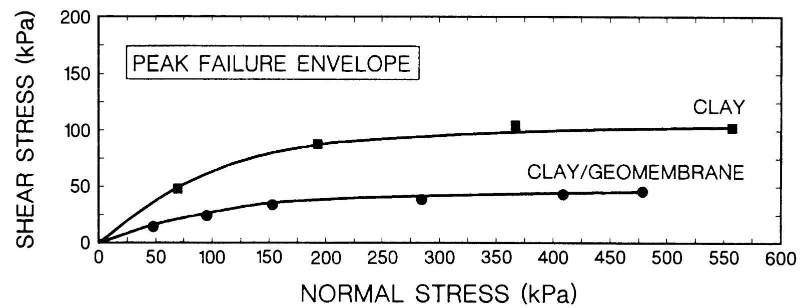

FIG. 8. Comparison of Peak Clay and Peak Clay/Geomembrane Interface Strengths

velopes are nonlinear. For comparison purposes, if a secant failure envelope is assumed to pass through the origin and the shear stress at a normal stress of 410 kPa, the peak friction angle is approximately 6°, and the residual friction angle is about 4°. Mitchell et al. (1990) reported that conventional direct-shear tests on the Kettleman Hills SC/SG interface yielded peak and residual friction angles of 13.6 ± 2.4° and 12.4 ± 1.1°, respectively, for water contents between 27–31%. However, an undrained residual strength of 43 ± 12 kPa was used by Seed et al. (1990) in post-failure stability analyses. Byrne et al. (1992) used "Draft" (1991) data from large-scale direct-shear tests to develop the following peak (τ_{peak}) and residual ($\tau_{residual}$) strength relations for the SC/SG interface at a water content of 29.7%:

$$\tau_{peak} = 45 \text{ kPa} \quad (1)$$

$$\tau_{residual} = 23.2 \text{ kPa} + 0.022\sigma_n \quad (2)$$

where σ_n = normal stress.

The failure envelopes from the large-scale direct-shear tests performed in "Draft" (1991) are superimposed on the ring-shear envelopes in Fig. 7. The three normal stresses used in "Draft" (1991) are 150, 280, and 410 kPa (Fig. 2), and the data points in Fig. 7 correspond to these tests. The ring-shear peak strength is slightly lower than the large-scale direct-shear tests at normal stresses greater than 150 kPa. This is in good agreement with previous research (Bishop et al. 1971; La Gatta 1970), suggesting that peak strengths measured in a ring-shear apparatus are slightly lower than direct-shear values for materials exhibiting a large post-peak strength loss. This is due to the nonuniformity of shear displacements radially across the specimen. If the material does not exhibit a significant post-peak strength loss, the peak strengths from direct-shear and ring-shear tests will be equal. The ring-shear residual interface strengths are 15–20% lower than the large-scale direct-shear test results for normal stresses greater than 150 kPa. The difference in residual interface strengths is probably due to the larger continuous-shear displacements that can be applied to the ring-shear tests. For stability analyses, there exist a number of cohesion and friction angle combinations that can be used to model the nonlinear peak and residual failure envelopes in Fig. 7. Therefore, it is recommended that the entire envelope or an appropriate value of the friction angle be incorporated in a stability analysis.

Fig. 8 provides a comparison of the peak SC/SG-interface failure envelope and the peak failure envelope of the clay-liner material at a water content of 29.7%. The envelopes have similar shapes but the SC/SG-interface strength is significantly less than the peak strength of the clay liner. If a secant envelope is assumed to pass through the origin and the shear stress at a normal stress of 480 kPa, the peak SC/SG-interface friction angle is about 6°, and the peak clay-liner friction angle is approximately 12°. These failure envelopes can be used to estimate the efficiency of the SC/SG interface by dividing the tangent of the interface friction angle by the tangent of the clay-liner friction angle. This yields an interface efficiency of 50–55% for normal stresses greater than approximately 190 kPa. This is in good agreement with the 45–60% range that has been reported for direct-shear tests on other clay-geomembrane interfaces at a normal stress near 500 kPa (Long et al. 1993). Therefore, it may be concluded that the peak strengths measured using a ring-shear apparatus are in agreement with direct-shear test results.

Fig. 9 provides a comparison of the residual SC/SG-interface strength and the residual strength of the clay-liner material. The envelopes have similar shapes but the SC/SG-interface strength is again significantly less than the residual clay-liner strength. A comparison of Figs. 8 and 9 reveals that the clay-liner material does not exhibit a large post-peak strength loss, thus the peak and residual failure envelopes of the clay liner are similar. If a secant envelope is assumed to pass through the origin and the shear stress at a normal stress of 480 kPa, the residual SC/SG-interface friction angle is approximately 4°, and the residual clay-liner friction angle is about 11°.

The residual interface efficiency was calculated to be approximately 35% at normal stresses greater than approximately 190 kPa, which is lower than the peak interface efficiency of 55%. Therefore, the SC/SG interface exhibits a larger post-peak strength loss than the clay-liner material. This is probably caused by some clay particles being oriented parallel to the direction of shear and by significant polishing of the geomembrane that occurs at large continuous-shear displacements. It should be noted that the excavated secondary HDPE geomembrane at Kettleman Hills exhibited a highly polished surface. Typical residual interface efficiencies reported for direct-shear tests range from 45% to 55% (Long et al. 1993). The difference between the ring-shear and direct-shear residual interface efficiencies is probably due to the larger continuous-shear displacement that can be applied in the ring-shear tests.

RING-SHEAR TESTS ON GEOSYNTHETIC/GEOSYNTHETIC INTERFACES

Torsional-ring-shear tests were also conducted on geonet/geomembrane and nonwoven geotextile/geomembrane interfaces. Fig. 10 presents the peak

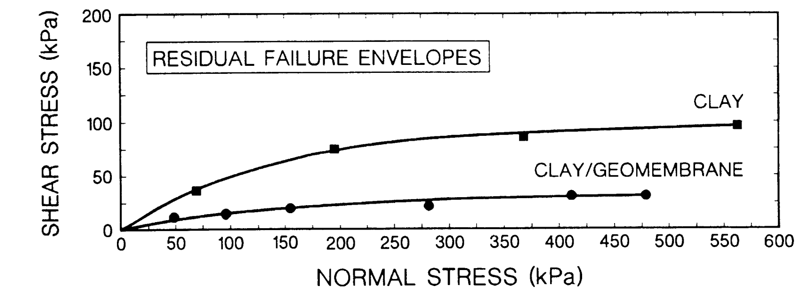

FIG. 9. Comparison of Residual Clay and Residual Clay/Geomembrane Interface Strengths

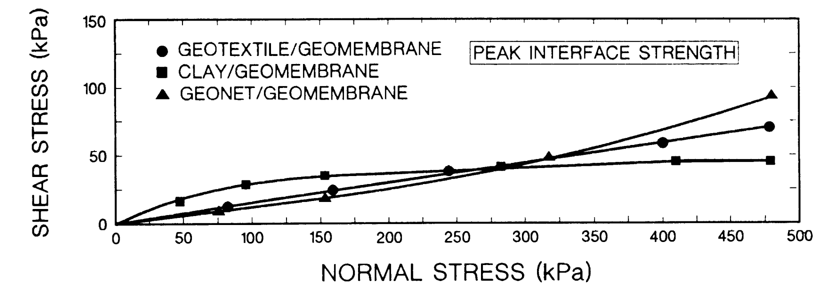

FIG. 10. Peak Failure Envelopes for Various Liner Interfaces

interface failure envelopes for these interfaces together with the SC/SG interface at a water content of 29.7%. The synthetic interfaces are critical (exhibit the lowest peak shear resistance) at normal stresses less than approximately 280 kPa. However, at normal stresses greater than approximately 280 kPa, the SC/SG interface is critical. The normal stress acting on the base of the Kettleman Hills Landfill at the time of failure ranged from 420 to 480 kPa. These stresses are based on a unit weight of 15.7–17.3 kN/m^3 and a fill depth of 27–28 m at the time of failure. Fig. 10 indicates that sliding should occur at the SC/SG interface for these normal stresses. This is in excellent agreement with field observations along the base of the landfill that showed striations only occurred along the SC/SG interface (Byrne et al. 1992).

Fig. 11 presents the residual interface failure envelopes for the geosynthetic/geosynthetic interfaces and the SC/SG interface at a water content of 29.7%. Fig. 11 suggests that in the upper 10–11 m of the sideslopes, that is, in areas with a normal stress less than or equal to 190 kPa, sliding could have occurred along any of these interfaces. However, below a depth 10–11 m along the sideslopes, that is, in areas with normal stresses greater than approximately 190 kPa, sliding should have occurred along the SC/SG interface. Again, this is in excellent agreement with field observations, which clearly showed sliding along the SC/SG interface at a depth of 10–15 m below the landfill rim. Above this depth, sliding occurred along the primary geomembrane/secondary geotextile interface.

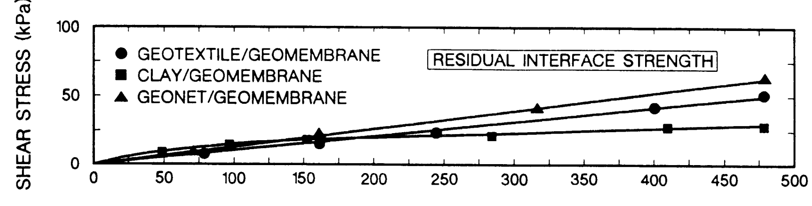

FIG. 11. Residual Failure Envelopes for Various Liner Interfaces

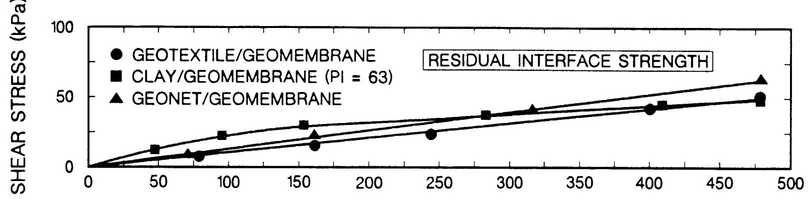

FIG. 12. Effect of Plasticity on Residual Clay/Geomembrane Interface Strength

EFFECT OF PLASTICITY ON CLAY/GEOMEMBRANE INTERFACE STRENGTH

For design considerations, the effect of increasing the percentage of sodium bentonite by weight in the secondary clay-liner mixture on the SC/SG-interface strength was also investigated. Additional sodium bentonite provides a lower hydraulic conductivity, but it may also decrease the interface strength. Fig. 12 presents the residual strength of a higher plasticity clay/geomembrane interface at a water content of 29.7%. This higher plasticity clay was obtained from the failed secondary clay liner on the base of the Kettleman Hills Landfill after waste excavation. A greater percentage of sodium bentonite was probably added to decrease hydraulic conductivity in this liner sample. This clay classifies as a CH according to the Unified Soil Classification System, having a liquid limit of 87% and a plasticity index of 63. Since both clay-liner samples classify as CH, this sample will be referred to as the higher plasticity clay liner. Hydrometer tests conducted during the present study revealed that 92% of the higher plasticity clay liner passes the U.S. Standard sieve No. 200 and the clay-size fraction (percent by weight finer than 0.002 mm) is 73%. For comparison purposes, the residual geosynthetic/geosynthetic failure envelopes from Fig. 11 are also presented in Fig. 12. The geotextile/geomembrane interface is critical for normal stresses less than approximately 450 kPa. Therefore, increasing the plasticity and clay-size fraction of the clay increases the residual interface strengths at a water content of 29.7%. The increase in interface strength with an increase in plasticity is attributed to differences in the optimum water content of the two clays. The optimum water content of the higher plasticity clay liner is 26% versus 22% for the secondary clay liner described previously. Therefore, a water content of 29.7% is slightly greater than the optimum water content of the higher plasticity clay liner. This higher optimum water content causes more frictional resistance, and thus higher interface strengths for the higher plasticity clay liner. Based on these results, it appears that clay/geosynthetic interface strengths are clay/site specific. Testing should be conducted to determine the critical interface for landfill-stability analyses.

STABILITY ANALYSIS OF KETTLEMAN HILLS PHASE IA OF LANDFILL B-19

Two- and three-dimensional slope-stability analyses of the Kettleman Hills failure were conducted to investigate the effect of complex landfill geometries on the calculated factor of safety and relevancy of ring-shear interface strengths. A three-dimensional analysis was conducted using a limit equilibrium technique based on Janbu's (1973) method. This analysis neglects the vertical component of interslice-shear forces and satisfies overall vertical and horizontal force equilibrium. The three-dimensional microcomputer slope-stability program, LF, developed by Golder Associates, Seattle, Wash., and the three-dimensional geometry in Fig. 13 were used for the analysis. The computer program allows the nonlinear-interface failure envelopes to be modeled using a trilinear failure envelope.

The two-dimensional cross section shown in Fig. 13 was used by Byrne et al. (1992) and was analyzed during the present study. Janbu's (1973) stability method was also used for the two-dimensional analysis. For comparison purposes, the two-dimensional hand calculations did not utilize Janbu's correction for neglecting the vertical component of interslice-shear forces. Therefore, the identical slope-stability method and assumptions were applied in the two- and three-dimensional stability analyses to permit comparison of the factors of safety. The same trilinear failure envelope was used to model the nonlinear-interface failure envelopes in the two- and three-dimensional analyses. The two- and three-dimensional stability analyses were performed assuming an average unit weight of the waste fill of 17.3 kN/m^3, a value based on bulk unit weight determinations conducted during waste excavation (Byrne et al. 1992). The basal operations layer, the primary

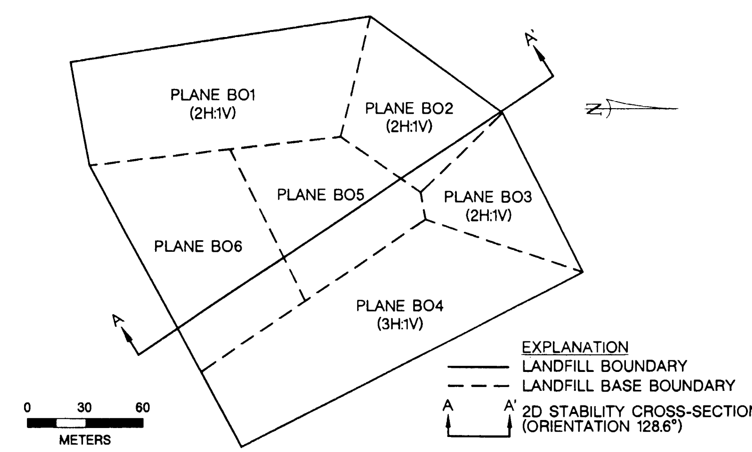

FIG. 13. Kettleman Hills Landfill B-19, Phase IA Geometry for Stability Analyses (after Byrne et al. 1992)

and secondary gravel-drainage layers, and the primary clay overlying the secondary clay/secondary geomembrane interface (Fig. 1) were also assigned an average unit weight of 17.3 kN/m^3.

Various combinations of peak and residual SC/SG-interface failure envelopes were applied in the analysis to determine the interface strength mobilized at the time of the Kettleman Hills failure. Byrne et al. (1992) reported that inspection of the liner system during removal of the waste from Phase IA of Landfill B-19 at Kettleman Hills showed that sliding occurred along the SC/SG interface on the base of the landfill. Fig. 13 presents the geometry of Kettleman Hills Landfill B-19 and the location of the basal panels BO5 and BO6. The basal panels are inclined at a 2% grade. Sliding along the northwest and southwest sideslopes (panels BO1 through BO3) was also concentrated on the SC/SG interface, except in the upper 10–15 m of the slope, where sliding occurred along the primary geomembrane/secondary geotextile interface. Inspection of the northwest and southwest sideslope toe areas showed evidence of a more complex sliding mechanism due to the sharp change in slope angle at the toe of the slope. Sliding appeared to have occurred on synthetic interfaces as well as on the SC/SG interface in these toe areas in order to accommodate the kinematic constraints of the grade change. Sliding along the northeast sideslope (Panel BO4) was concentrated on the SC/SG interface and was approximately parallel to the strike of this sideslope.

To model field conditions, the residual failure envelope for the primary geomembrane/secondary geotextile interface was assigned to the slip surface in the upper 10–15 m of the sideslope. There is only a slight difference between the residual failure envelopes for the SC/SG and geomembrane/geotextile interfaces in Fig. 12 for normal stresses less than 190 kPa. As a result, interchanging these failure envelopes had a negligible effect on the calculated factors of safety.

Table 1 presents the three- and two-dimensional stability analyses. The three-dimensional factor of safety for the post-failure geometry and residual interface strengths assigned to all landfill panels is 0.95. This is in excellent agreement with field observations and confirms the relevancy of residual interface strengths measured using the torsional-ring-shear apparatus. The three-dimensional factors of safety for the prefailure geometry range from 0.73 to 1.26 when the residual and peak interface strengths, respectively, are assigned to all sliding surfaces. This suggests that Phase IA of landfill B-19 was marginally stable during construction. If the peak interface strength is assigned to the basal panels (BO5 and BO6 in Fig. 13), and the residual interface strength is assigned to all of the sideslopes (BO1 through BO4), then the three-dimensional factor of safety for the prefailure geometry is 0.92. This factor of safety is slightly less than unity, but still in excellent agreement with the 19 March 1988 slide. Possible reasons for this factor of safety being slightly less than unity include neglecting the complex sliding that occurred in the toe areas on the northwest and southwest sideslopes and the interface strength being slightly greater than the residual value near the toe of the sideslopes. However, a factor of safety of 0.92 is in excellent agreement with field observations, and thus it is recommended that peak and residual interface strengths be assigned to the landfill base and sideslopes, respectively, for general design purposes.

TABLE 1. Results of Two- and Three-Dimensional Slope-Stability Analyses of Kettleman Hills Landfill B-19

Slide geometry (1)	Interface strength (2)	3D factor of safety (3)	2D factor of safety (4)
Postfailure	Residual	0.95	0.88
Prefailure	Residual	0.73	0.71
Prefailure	Peak	1.26	1.18
Prefailure	Peak and residual	0.92	1.03

The two-dimensional factors of safety are lower than the three-dimensional values and in excellent agreement with field observations. The two-dimensional factor of safety for the postfailure geometry and a residual interface strength assigned to all sliding surfaces is 0.88. This factor of safety is lower than 0.95 because of the absence of three-dimensional effects. This suggests that the residual interface strengths should be higher for field agreement. Neglecting three-dimensional effects, however, yields back-calculated strengths that are too high. In summary, it may be concluded that the residual interface strengths measured using a torsional-ring-shear apparatus are in excellent agreement with field observations. The two-dimensional factors of safety for the prefailure geometry range from 0.71–1.18 when the residual and peak interface strengths, respectively, are assigned to all sliding surfaces. It is important to note that these two-dimensional factors of safety are less than the corresponding three-dimensional factors of safety.

When peak and residual interface strengths are applied to the base and sideslopes, respectively, the two-dimensional factor of safety for the prefailure geometry is slightly larger than the three-dimensional factor of safety. It is anticipated that this result is due to the percentage of area that is assigned peak and residual strengths in the two- and three-dimensional analyses. For example, 35% of the slide surface was assigned a peak interface strength, while 65% received a residual interface strength in the three-dimensional analysis. Conversely, in the two-dimensional analysis, 36% of the slide surface was assigned the residual interface strength, while 64% of the base received the peak value. Further evidence of the importance of area balance is that the two- and three-dimensional factors of safety yield a consistent pattern when the same interface strength is applied to the entire slip surface.

Based on this reevaluation of the Kettleman Hills failure, it appears that a two-dimensional stability analysis can be used with a suitable factor of safety for landfill design. Considerable engineering judgment should be used, however, to determine the critical two-dimensional section. A three-dimensional analysis may provide insight into the location of the critical two-dimensional section. But a more important parameter in landfill-stability analyses is the measurement and selection of the interface-shear strength parameters. The difference in the two- and three-dimensional factors of safety is 10–15% (Table 1), while the variability in interface strengths is significantly larger (Figs. 5 or 6). Therefore, the majority of the design effort should focus on site-specific testing and careful selection of interface strengths.

SELECTION OF INTERFACE STRENGTHS FOR SLOPE-STABILITY ANALYSES

The regressive analysis of the Kettleman Hills failure indicates that peak and residual interface strengths are mobilized along a landfill's base and sideslopes, respectively, during construction and waste placement. Waste is typically placed in landfills using a 2-m-thick lift with little compaction. As a result, the waste usually settles a considerable amount during the filling operation. Review of field settlements from several landfills indicates that municipal solid-waste landfills usually settle approximately 10% of the initial height because of placement and decomposition (Chang and Hannon 1976; York et al. 1977; Dodt et al. 1987; Coduto and Huitric 1989). It was found that hazardous-waste landfills typically settle 2–6% of the initial height (Gray and Lin 1972; Seals et al. 1972; Leonards and Bailey 1982; McLaren and DiGioia 1987). Settlement of the fill induces shear stresses in the sideslope liner system, all of which tends to displace the liner downslope. These shear stresses induce shear displacements along specific interfaces in the liner system that may lead to the mobilization of a residual interface strength. In addition, thermal expansion and contraction of the sideslope liner system during construction and filling may also contribute to the accumulation of shear displacements and the mobilization of a residual interface strength. Therefore, it is recommended that a residual interface strength be assigned to all sideslopes for design purposes.

Fill settlement, and thus liner shear displacements, decrease with depth along the sideslopes. Therefore, at the toe of the sideslopes, the residual interface strength may not be mobilized due to small displacements. For design, however, it is recommended that the entire sideslope be assigned the residual interface strength. Since shear displacements may be small along the base of the landfill, it is possible that the peak interface strength can be mobilized along the base of the landfill. In summary, it appears that peak and residual interface strengths should be assigned to the base and sideslopes, respectively, for design purposes.

A second design scenario involves assigning ring-shear residual interface-strengths to all slip surfaces and requiring a factor of safety greater than unity. This scenario should be considered, because the interface peak strength is usually mobilized at a small laboratory displacement (Fig. 3). Since field interface displacements and the effect of progressive failure are not known, it is prudent to also consider this scenario. If the residual interface strength is measured in a direct-shear apparatus, a factor of safety greater than unity may be required to compensate for the limited continuous-shear displacement applied in the apparatus.

CONCLUSIONS

The following conclusions are based on the analysis, data, and interpretation presented in this paper:

1. Clay/geomembrane interface strengths are a function of compaction water content, normal stress, and soil type. Therefore, a geosynthetic/geosynthetic interface may be more critical than the clay/geomembrane interface depending on placement water content, applied normal stress, and clay composition. Site-specific testing should be conducted to identify the critical interface as a function of normal stress.
2. Based on field observations, the torsional-ring-shear apparatus appears to provide an excellent estimate of field peak and residual interface-shear strengths. The main advantage of a ring-shear apparatus is that large continuous-shear displacement can be applied in one direction in order to achieve a residual strength condition.
3. Undrained clay/geomembrane-interface failure envelopes are nonlin-

ear. It is recommended that the entire failure envelope or an appropriate value of the friction angle be incorporated in stability analyses.
4. Design-stability analyses should utilize peak and residual interface strengths on the landfill base and sideslopes, respectively. Since field interface displacements and the effect of progressive failure are not known, a factor of safety greater than unity with a ring-shear residual interface strength assigned to all slip surfaces should also be satisfied.
5. A two-dimensional stability analysis can be used for the design of landfills with complex geometries. However, considerable judgment should be used in determining the critical two-dimensional cross section. A three-dimensional analysis may aid the selection of the critical two-dimensional cross section.
6. The most important aspect of a landfill-stability analysis is measurement and selection of the interface-shear strengths. Significant time and effort should be expended on estimating the critical interface-shear strengths.

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