Pierce Chandler

Kettleman Hills Waste Landfill Slope Failure. I: Liner-System Properties

By James K. Mitchell,[1] Fellow, ASCE, Raymond B. Seed,[2] Associate Member, ASCE, and H. Bolton Seed,[3] Honorary Member, ASCE

Abstract: A slope-stability failure occurred in a 15 acre hazardous-waste landfill (90 ft high) in which lateral displacements of up to 35 ft and vertical settlements of up to 14 ft were measured. Failure developed by sliding along interfaces within the composite, multilayered geosynthetic-compacted clay liner system beneath the waste fill. The testing, analyses, and related studies made to determine the cause of the failure are the subject of this and a companion paper (Seed et al. 1990). The present paper presents details of a direct shear and pullout testing program undertaken to determine liner-system-interface shear-strength characteristics. The interfaces between the various geosynthetics, and between these materials and the compacted clay in the liner system, are characterized by low frictional resistance, with values of interface-friction angle as low as 8° for some combinations. The most critical interfaces were determined to be those between high-density polyethylene (HDPE) geomembrane and geotextile, HDPE geomembrane and geonet, and HDPE geomembrane and saturated compacted clay. Representative values of interface shear-strength parameters were obtained for use in the stability analyses described in the companion paper. The variations in measured strength parameters for the different interfaces in the liner system indicate the desirability of conducting similar test programs for proposed new facilities to establish design parameters.

Introduction

Landfill Unit B-19, covering an area of about 36 acres, forms part of a Class I hazardous-waste treatment-and-storage facility at Kettleman City, California. The waste repository essentially consists of a very large, oval-shaped bowl excavated in the ground to a depth of about 100 ft, into which the waste fill is placed. The "bowl" has a nearly horizontal base, and side slopes of 1 on 2 or 1 on 3. To prevent the escape of hazardous materials into the underlying and surrounding ground and the ground water below, the base and sides of the excavation are lined with a multilayer system of impervious geomembranes, clay layers, and drainage layers. An overall view of the facility is shown in Fig. 1.

For operational reasons, the lining of the northern end of the "bowl," designated Phase 1-A and covering approximately 15 acres, was completed first and placement of solid hazardous waste was initiated in this section of the facility in early 1987. At the same time, the liner systems for other phases of the project were being completed. The placement of waste in Phase 1-A may be seen in Fig. 1, in addition to a thin covering of soil over the

[1]Prof. Civ. Engrg., Dept. of Civ. Engrg., Univ. of California, Berkeley, CA 94720

[2]Assoc. Prof. Civ. Engrg., Dept. of Civ. Engrg., Univ. of California, Berkeley, CA.

[3]Deceased, formerly Cahill Prof. of Civ. Engrg., Univ. of California, Berkeley, CA.

Note. Discussion open until September 1, 1990. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the ASCE Manager of Journals. The manuscript for this paper was submitted for review and possible publication on December 7, 1988. This paper is part of the *Journal of Geotechnical Engineering*, Vol. 116, No. 4, April, 1990. ©ASCE, ISSN 0733-9410/90/0004-0647/$1.00 + $.15 per page. Paper No. 24577.

FIG. 1. Photo Looking Northwest Showing the Unit B-19, Phase I-A Landfill

base of the remainder of the facility. The surface topography in Phase 1-A as it existed on March 15, 1988 is shown in Fig. 2(*a*); a cross section through the center of the fill in this zone showing the base and sides of the excavation and the liner system is shown in Fig. 2(*b*).

On Saturday, March 19, 1988, a slope-stability failure occurred that resulted in lateral displacements of the surface of the waste fill of up to 35 ft and vertical settlements of the surface of the fill of up to 14 ft. Surface cracking was clearly visible, as also were tears and displacement on the exposed portions of the liner system.

Because of the danger of a break having occurred in the liner system, a major investigation was undertaken to determine both the cause of the failure and appropriate methods of testing and analysis to preclude the possibility of similar failures at other facilities. The testing, analyses, and related studies to determine the cause of the failure are the subject of this and a companion paper (Seed et al. 1990). In the present paper, the failure is briefly described, a testing program to evaluate the shear resistances along interfaces in the composite liner system is summarized, and conclusions are drawn concerning the properties appropriate for evaluation of the waste-landfill stability. These properties are used in stability analyses, described in the companion paper, to provide a probable explanation for the cause of the failure.

Landfill Failure

By mid-March 1988, the waste pile in the Phase I-A portion of the Unit B-19 landfill had reached a maximum fill height of about 90 ft. Up to this point no evidence of instability had been observed. According to eyewitness reports, a crack of 1/2 in. or less was noticed on the truck ramp on the

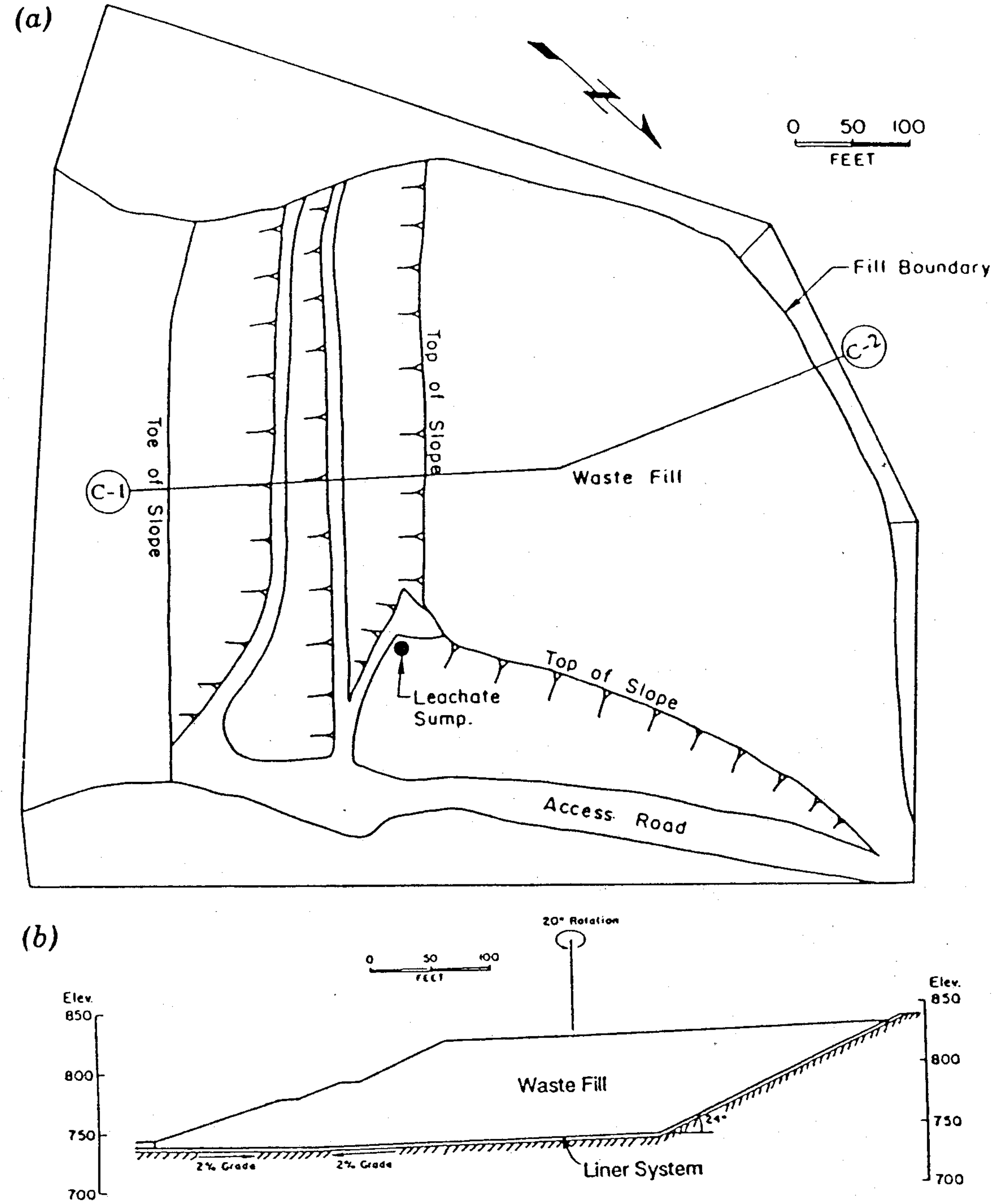

FIG. 2. Preslide Surface Topography and Cross Section C-1/C-2 for the Unit B-19, Phase I-A Landfill (March 15, 1988): (*a*) Surface Topography, March 15, 1988; and (*b*) Cross section C-1/C-2

northeast corner of the landfill at about 6:30 a.m. on Saturday, March 19, 1988, and at about 9:30 a.m., a major crack was observed along the top edge of the north and west sides, where the side slope is 2 horizontal to 1 vertical. Estimates of the movements on the north and west sides at that time ranged from a few inches to a few feet. By noon, movements of several feet were evident, and a truck had become trapped inside the landfill area. The main failure, which brought the landfill to its final postslide geometry, is reported to have occurred by early afternoon. No subsequent movements were measured.

There is no record of significant seismic activity in the vicinity or any other abnormal events at the time of failure or for several months preceding. Pumping from the leachate-collection system had remained almost steady at about 4,500 gal per month. The fluid level in the leachate-collection and removal system varied between 10 and 30 in., with an average of about 20 in. over this period. The fill-placement rate had been essentially constant since filling began almost a year prior to the failure.

Based on field observations, photographic and survey records, and stability analyses, it is clear that the failure developed by sliding along interfaces within the multilayer liner system, within the clay layers that form part of the liner system, or along combinations of liner interfaces and through the clay. As the geometry of the landfill and the liner at the time of failure are known, stability analyses can be made for various sections through the landfill. However, to perform such analyses, requires a knowledge of the frictional resistance between the different components of the liner system and of the shear strength of the compacted liner clay.

Liner System

The composite double-liner system used under the base of the Phase I-A portion of the B-19 landfill is generally as shown in Fig. 3. The liner system complies with the 1986 Hazardous and Solid Waste Ammendments to the Resource Conservation and Recovery Act, which mandate leachate-collection systems and both geomembrane and compacted-clay leachate-containment barriers.

A liner configuration similar to that shown in Fig. 3 was used on the side slopes, with the following exceptions.

- There is no vadose-zone monitoring system.
- The primary clay liner does not extend up the slope.
- The 1 ft thick granular layers were not included.
- An additional geont layer occurs immediately above the secondary high-density polyethylene (HDPE) liner.
- A 60 mil HDPE geomembrane liner was placed over the geotextile layer of the primary leachate-collection and removal system.

The geomembrane is a nontextured ("smooth") 60 mil Gundle high-density polyethylene (HDPE) liner. The geonet is Polynet 300, which is also a high-density polyethylene. The geotextile is Trevira Spunbond No. 1145 filter fabric. Samples of each of these types of geosynthetics were available for testing.

The clay-liner material was a mixture of on-site claystone, siltstone, and sandstone with approximately 5% bentonite, except in the vicinity of the leachate-collection sump, where the bentonite content was approximately 10%. The resulting prepared materials contained 63–97% finer than the #200 sieve, and the plasticity index varied between 22% and 46%. The soil-clay mixture was compacted to an average relative compaction of 94% based on the Standard Proctor (ASTM D-698) Compaction Test at a water content averaging about five percentage points above the Proctor optimum.

Available information in the literature concerning friction angles and coefficients between different geosynthetic materials, and between these materials and compacted clay, is limited. Such data as were available, e.g., Mar-

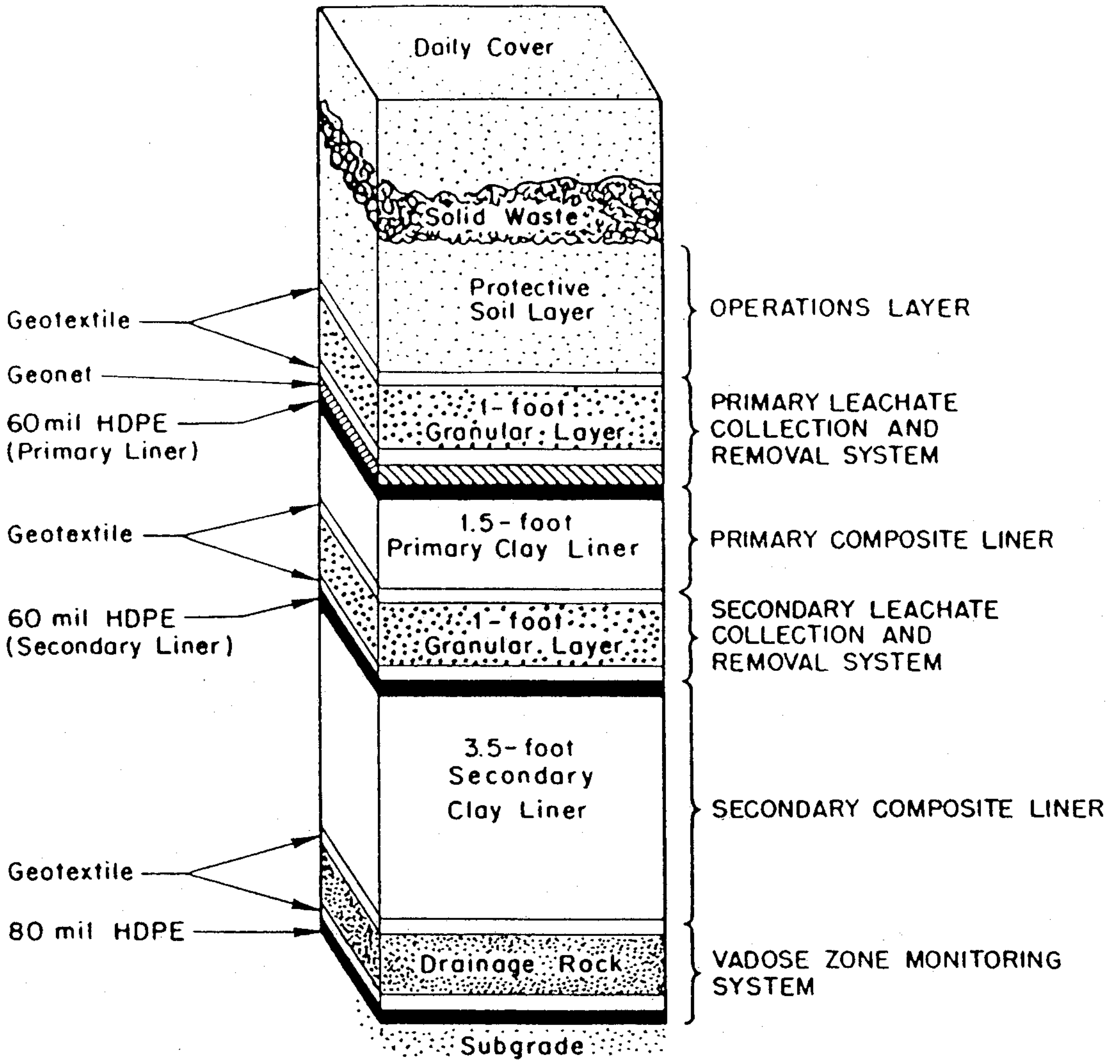

FIG. 3. Schematic Illustration of Multilayer Liner System at Base of Landfill

tin et al. (1984), indicate significant ranges of values as shown in Tables 1, 2, and 3. Preliminary stability analyses indicated that the more specific values for the materials at this site would be needed if reliable analyses of the slope failure were to be made, and thus a comprehensive study was under-

TABLE 1. Soil-to-Geomembrane Friction Angles (after Martin et al. 1984)

	Soil Types		
Geomembrane (1)	Concrete sand (ϕ = 30°) (2)	Ottawa sand (ϕ = 28°) (3)	Mica Schist sand (ϕ = 26°) (4)
EPDM	24° (0.80)[a]	20° (0.71)[a]	24° (0.92)[a]
PVC			
Rough	27° (0.90)[a]	—	25° (0.96)[a]
Smooth	25° (0.83)[a]	—	21° (0.81)[a]
CSPE	25° (0.83)[a]	21° (0.75)[a]	23° (0.88)[a]
HDPE	18° (0.60)[a]	18° (0.64)[a]	17° (0.65)[a]

[a]Efficiency, defined as the ratio of interface-friction angle to soil-friction angle.

TABLE 2. Geomembrane-to-Geotextile Friction Angle (after Martin et al. 1984)

Geotextile (1)	GEOMEMBRANE EPDM (2)	PVC Rough (3)	PVC Smooth (4)	CSPE (5)	HDPE (6)
CZ 600	23°	23°	21°	15°	8°
Typer 3401	18°	20°	18°	21°	11°
Polyfilter X	17°	11°	10°	9°	6°
500 X	21°	28°	24°	13°	10°

TABLE 3. Soil-to-Geotextile Friction Angle (after Martin et al. 1984)

Geotextile (1)	Soil Types: Concrete sand (ϕ = 30°) (2)	Ottawa sand (ϕ = 28°) (3)	Mica Schist sand (ϕ = 26°) (4)
CZ 600	30° (1.00)[a]	26° (0.93)[a]	25° (0.96)[a]
Typer 3401	26° (0.87)[a]	—	—
Polyfilter X	26° (0.87)[a]	—	—
500 X	24° (0.80)[a]	24° (0.86)[a]	23° (0.88)[a]

[a]Efficiency, defined as the ratio of the interface-friction angle to the soil-friction angle.

taken to determine the strength characteristics of the various components of the liner system.

Laboratory Investigation of Liner System

Laboratory tests were performed to evaluate (1) The interface-shear-strength characteristics of the various components of the multilayer liner system; and (2) the shear-strength and consolidation characteristics of the compacted clay that forms two layers within the multilayer liner system shown in Fig. 3.

Based on previous test data for granular soil/geotextile liner interfaces, shear failures along surfaces between granular soil or drainage rock and geotextile were considered unlikely. Accordingly, the following interface combinations occurring within the liner system were tested to evaluate their shear strength and shear-stress versus shear-displacement characteristics:

- HDPE liner/geotextile interface.
- HDPE liner/compacted clay liner interface.
- HDPE liner/geonet interface.
- Geotextile/compacted clay liner interface.
- Geotextile/geonet interface.

In addition to these liner-interface combinations, tests were also performed to evaluate the shear-resistance characteristics of HDPE liner/HDPE liner interfaces. Although this type of interface does not occur in the multilayer liner system underlying the landfill, these additional tests were of interest because the actual liner-interface combinations of lowest shear strength were found to be those of HDPE liner/geotextile, HDPE liner/geonet, and HDPE liner/compacted clay liner. The surface shear-slippage characteristics of the HDPE liner were thus of significant potential interest in understanding the overall interface-shear behavior of the various interface combinations.

Two types of tests were performed to evaluate interface shear-strength characteristics: (1) Direct shear tests; and (2) pullout-box tests. The relatively simple direct shear tests were performed to develop sufficient data to evaluate representative shear-strength characteristics and ranges of likely shear strengths for all interface combinations and conditions considered (e.g. dry, submerged, etc.). The more time-consuming pullout-box tests were able to test samples with considerably larger total interface-contact areas. In addition, the pullout-box tests could be performed to large relative shear displacements, in excess of 3 in., while the direct shear tests were limited to a relative shear displacement of less than 0.3 in. The pullout-box tests therefore served two purposes: (1) They provided a check on the results of the direct shear tests by evaluating the potential effects of sample size and sample interface-contact area; and (2) they established the shear-strength versus shear-displacement characteristics for each interface combination at relative shear displacements of more than approximately 0.25 in.

Testing was also performed to evaluate the shear-strength and consolidation characteristics of the compacted clay material that comprised the 1.5 ft thick primary clay liner and the 3.5 ft thick secondary clay liner shown in Fig. 3. Three types of tests were performed on samples of this material compacted to initial densities and water contents considered representative of field conditions: (1) Unconsolidated-undrained (UU) triaxial tests; (2) consolidated-undrained (CU) triaxial tests; and (3) incremental, one-dimensional consolidation tests.

Direct Shear Interface Resistance Tests

General

Direct shear tests of various interface combinations were performed using a modified Karol-Warner direct shear testing apparatus. The modifications of the apparatus consisted of installing strain-gaged load cells to facilitate precise electronic monitoring of normal and shear forces applied to test specimens.

Fig. 4(*a*) shows a schematic cross section of a typical interface-sample configuration for interface combinations that do not include compacted clay. These interface combinations include (1) HDPE liner/geotextile; (2) HDPE liner/geonet; (3) geotextile/geonet; and (4) HDPE liner/HDPE liner. All these interface combinations were tested using 2.8 in. × 2.8 in. square samples mounted with epoxy cement on 4 in. diameter round steel and/or aluminum platens. Normal stresses acting on the sample interfaces were corrected to account for the weights of the overlying top platen, loading block and loading ball, as well as for the change in contact area that developed with increasing shear displacement.

Fig. 4(*b*) is a schematic cross section showing a typical sample configuration for interface combinations consisting of either HDPE liner/compacted clay liner or geotextile/compacted clay liner. For these interface combinations, the HDPE liner or geotextile specimen was again a 2.8 in. × 2.8 in.

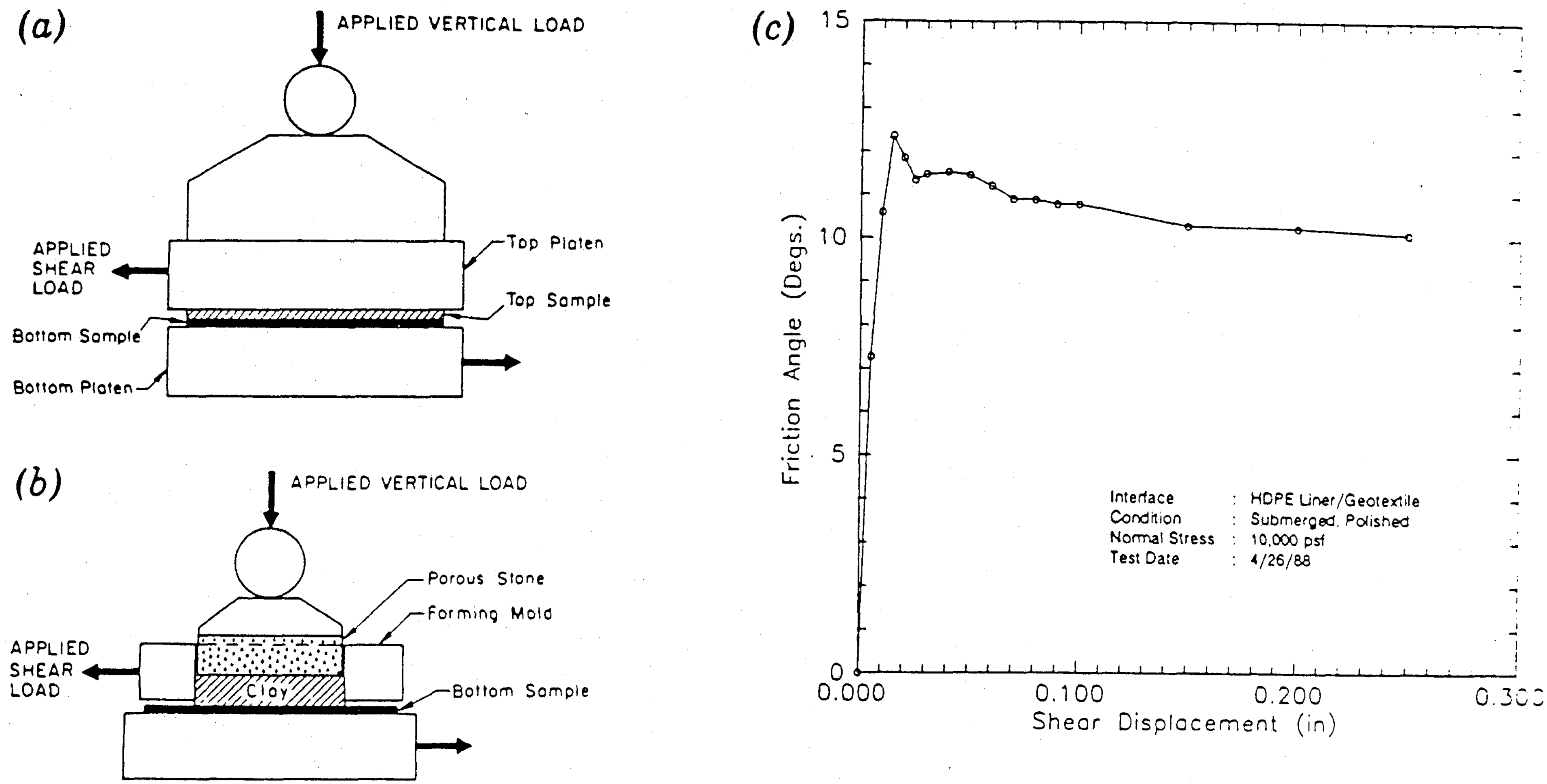

FIG. 4. Direct Shear Test Interface Sample Configurations and Typical Test Results: (*a*) Interface Sample Combinations Not Including Compacted Liner Clay; (*b*) Interface Sample Combinations Including Compacted Liner Clay; and (*c*) Interface Direct Shear Test Results: Test Number NN-10 (HDPE Liner/Geotextile)

specimen mounted with epoxy cement on a 4 in. diameter round-base platen. The clay liner material was then compacted above this base-interface specimen within a 2 in. × 2 in. steel forming mold. Clay specimens were compacted to water contents and densities considered representative of field conditions using a Harvard Miniature Compaction Test pneumatic tamping piston. Compacted clay specimens were typically 0.3–0.4 in. thick. All clay samples were compacted to initial dry densities of γ_d = 94 to 98 lb/cu ft, at initial water contents of 27–31%.

Most interface combinations were tested under a variety of conditions. This included testing most interfaces both "dry" and "submerged." In addition, HDPE liner/geonet interface combinations were found to be directionally dependent and so were sheared with varying orientations of the geonet relative to the direction of shear. Minor differences were visually observed between the two sides of HDPE liner samples, and also between the two sides of geonet samples, and the effects of these differences were investigated. The various geonet samples delivered for testing at different times were observed to have slightly different mesh spacings, and HDPE liner samples were observed to vary slightly in color and tone, so the effects of these differences on the interface shear strengths of various interface combinations were also investigated. Compacted clay specimens were tested under two conditions: "as compacted," and after soaking for 24 hr under light surcharge. Complete "wetting" was determined to require only several hours, but this is not necessarily full saturation. However, the observation that the shear strengths measured for these "presoaked" samples was essentially independent of the vertical stress applied under undrained conditions suggests that at least a very high degree of saturation was achieved during soaking under light surcharge.

Interface-shear tests were performed by shearing the interface samples under strain-controlled loading at rates of shear displacement of between 0.005 in./min and 0.05 in./min. Variations in shear rate in this range had no apparent influence on the measured shear-resistance characteristics of any of the liner-interface combinations tested. Normal stresses on the sample interfaces during testing were σ_n = 3,300 psf, 6,600 psf, and 10,000 psf; these were selected as representing the range of field values of interest.

HDPE Liner/Geotextile Interface Direct Shear Tests

HDPE liner/geotextile interface samples were tested, both dry and submerged, at normal stresses of σ_n = 3,300 psf, 6,600 psf, and 10,000 psf. One interesting characteristic of this interface combination is the tendency for the geotextile to "polish" the HDPE liner so that interface-shear strength is reduced with increased shear displacement. For this reason, samples were sheared repeatedly (releasing the applied vertical load or normal stress and then repositioning the samples before retesting them) in order to evaluate interface shear strengths under conditions ranging from "unpolished" (virgin samples) through "partially polished," to "fully polished." It was observed that both dry and submerged HDPE liner/geotextile interfaces exhibited progressively reduced shear strength with increased polishing, but that there was a limit to this strength reduction as designated by the description "fully polished."

Table 4 summarizes the results of direct shear tests performed on HDPE liner/geotextile interface samples under "dry" and "submerged" conditions.

TABLE 4. HDPE Liner/Geotextile Interface Direct Shear-Resistance-Test Results

Condition (1)	Number of tests (2)	Peak friction angle ϕ_p (3)	Shear displacement[a] at ϕ_p (in.) (4)	Residual friction angle ϕ_r (5)
Unpolished, dry	6	12.5° ± 0.7°	0.047 ± 0.010	10.6° ± 1.2°
Partly polished, dry	13	10.6° ± 0.7°	0.042 ± 0.020	9.8° ± 0.7°
Fully polished, dry	4	10.3° ± 0.9°	0.048 ± 0.010	9.6° ± 0.9°
Unpolished, submerged	4	10.4° ± 1.0°	0.022 ± 0.005	8.4° ± 1.2°
Fully polished, submerged	9	9.3° ± 1.0°	0.034 ± 0.013	8.4° ± 0.9°

[a]Mean values ± standard deviation.

Note: A solitary high value of ϕ_r = 10.0° is omitted.

Typical shear-strength versus shear-displacement behavior consisted of a minor peak in shear strength (ϕ_p) occurring at small shear displacement (typically less than 0.05 in.), followed by a slight decrease to a residual shear strength (ϕ_r). An example of this shear-strength versus shear-displacement behavior is shown in Fig. 4(*c*). Residual friction angles were typically about 0.5° to 2° less than the peak friction angles.

Both peak and residual friction angles were found to decrease with increased polishing for both "dry" and "submerged" conditions. Submergence was also found to result in a reduction in interface friction, with the residual friction angles of both unpolished and fully polished submerged samples being typically about 0.5°–2° lower than corresponding dry samples with similar degrees of polishing.

Interface-shear resistance was not found to vary with changes in interface normal stress. Similarly, the use of the two different sides of the HDPE liner, and the use of different "batches" of liner material (some with visible minor discoloration) had no significant effect on the interface-shear strengths observed.

HDPE Liner/Compacted Clay Interface Direct Shear Tests

Two series of five direct shear tests were performed on HDPE liner/compacted clay liner interface samples, under two sets of conditions. The first series included unconsolidated undrained tests on samples in which the clay liner soil was compacted to the as-compacted field density and water content and then tested in this nonsaturated state. After compaction, a vertical (normal) stress of σ_n = 3,300 psf, 6,600 psf, or 10,000 psf was applied and the samples were sheared. The results of these tests on samples of nonsaturated "as compacted" clay/HDPE liner interface samples are summarized in the first line of Table 5. These samples showed a tendency for nominal peaking of shear strength at small shear displacements of approximately 0.05 in. or less, followed by a slight shear-strength reduction to a residual friction angle between 11° and 14°.

The second series of five tests was performed on samples in which the clay was initially compacted to field conditions. The samples were then submerged and soaked and allowed to swell for 24 hr under light surcharges of 50–200 psf. After this period of soaking, the final testing vertical load (final

Smoo

TABLE 5. HDPE Liner/Compacted Clay Interface Direct Shear-Resistance-Test Results

Condition (1)	Number of tests (2)	Peak friction angle[a] or strength[a] (3)	Shear displacement at peak strength[a] (in.) (4)	Residual friction angle[a] (ϕ_r) or residual strength[a] (τ_r) (5)
As compacted	3	13.6° ± 2.4°	0.110 ± 0.10	12.4° ± 1.1°
After soaking under 50–200 psf surcharge	5	1,004 ± 128 psf	0.15 ± 0.09	910 ± 86 psf

[a]Mean values ± standard deviation.

testing interface total normal stress) was applied, and the samples were immediately sheared without allowing time for drainage or pore-pressure dissipation. These rapid direct shear tests thus represented essentially unconsolidated-undrained (UU) testing conditions. Total normal stresses applied to these samples were $\sigma_{n,\text{total}}$ = 3,300 psf, 6,600 psf, and 10,000 psf. The results of the tests are shown in the second line of Table 5. The resulting interface-shear strengths under these presoaked UU conditions were independent of the normal stress applied, which is further indication that drainage under the increased normal stress was negligible during the tests. The samples showed a slight tendency for moderate peaking of shear strength at relatively small shear displacement, and all five samples tested exhibited residual interface-shear strengths of between 820 and 1,020 psf.

HDPE Liner/Geonet Interface Direct Shear Tests

The HDPE liner/geonet interface was one of the most complex interface combinations tested because of the following factors.

- Interface shear resistance was found to be directionally dependent. Shear resistance was significantly lower when the direction of shear slippage was aligned parallel to the geonet strands in contact with the HDPE liner (referred to as "aligned shear") than when shear slippage was not in this direction (referred to as "transverse shear").
- Geonet samples were delivered for testing on three different dates. The mesh, or grid spacings, of the samples in each of these three "batches" were slightly different.
- All geonet samples had one side (or "face") on which the strands comprising the grid or net were more curved or "wavy" than on the other face. There was, therefore, a "top" and a "bottom" to the geonet samples.
- Some of the HDPE liner samples received for testing were slightly, but visibly, discolored. This discoloration manifested itself as a slightly lighter tone (and/or slightly tan tint) to the otherwise deep, black HDPE liner color.

Among these factors, which were all considered to represent potential sources of variance in HDPE liner/geonet interface-shear-strength behavior, the most important was the difference in behavior between samples sheared in the

TABLE 6. HDPE Liner/Geonet Interface Direct Shear-Resistance-Test Results

Condition (1)	Number of tests (2)	Peak friction angle[a]: ϕ_p (3)	Shear displacement[a] at ϕ_p (in.) (4)	Residual friction angle[a]: ϕ_r (5)
Transverse shear, dry	3	9.0° ± 0.25°	0.03 ± 0.008	7.6° ± 0.3°
Transverse shear, submerged[b]	9	8.8° ± 1.2°	0.149 ± 0.113	8.3° ± 1.2°
Aligned shear, submerged[c]	20	7.6° ± 1.3°	0.032 ± 0.052	6.3° ± 0.9°

[a]Mean values ± standard deviation.

[b]ϕ_r was still increasing slightly in three tests when test was stopped at 0.25 in. shear displacement. Results of these tests are not included in the tabulated averages.

[c]ϕ_r was still increasing slightly in one test when test was stopped at 0.25 in. shear displacement. Results of this test are not included in the tabulated averages.

"aligned shear" mode and the "transverse shear" mode. Accordingly, a number of tests were performed for each of these two shearing modes, and the results are presented in Table 6.

A variety of degrees of alignment between the direction of interface-shear slippage and the orientation of the geonet strands in contact with the HDPE liner was investigated, and it was concluded that only very close parallelism between the contact strands and the direction of sheear slippage represented "aligned shear" with very low interface friction. When the contact-strand orientation and shear-slippage direction were more than approximately 10°–15° out of parallel, the shear strength behavior was found to be representative of the "transverse shear" mode, with higher interface-shear strength.

Samples from each of the three geonet "batches," with their slightly different mesh, or grid, spacings, exhibited no significant differences with respect to HDPE liner/geonet interface-shear-strength behavior. Similarly, the two different "sides" of the geonet samples appeared to result in similar HDPE liner/geonet interface-shear-strength behavior. The use of HDPE liner samples with and without slight discoloration also had no significant effect on HDPE liner/geonet interface-shear-strength behavior: it was surmised that this slight discoloration represented a shallow, surficial effect, and that the geonet gouged more deeply into the HDPE liner during interface shearing than the shallow depth to which this discoloration effect extended.

The results of HDPE liner/geonet interface direct shear tests on interface samples sheared in the nonaligned or "transverse shear" mode for both "dry" and "submerged" conditions showed a moderate tendency for slight peaking in strength at small relative slip displacements, and then a fairly steady residual interface strength. Interface-friction angles did not vary significantly over the range of interface normal stresses applied (σ_n = 3,300 psf, 6,600 psf, and 10,000 psf). Although only three tests were performed under "dry" interface conditions, there does not appear to be a significant difference in the residual interface strengths (residual friction angles: ϕ_r) between samples tested "dry" and "submerged." Indeed, based on these tests, the "dry" HDPE liner/geonet interfaces appear to have a slightly lower residual strength ($\phi_r \approx$ 7° to 8°) than do the "submerged" interfaces ($\phi_r \approx$ 7° to 10°). This may represent an actual behavior characteristic of this interface combination, or may be due to the limited number of "dry" tests performed.

The "aligned shear" slippage mode was initially considered to be of significant potential interest with respect to the investigation of the slippage of March 19, 1988. However, investigations have shown that the near-perfect degree of alignment between the geonet contact strands and the direction of shear slippage necessary to mobilize the very low-strength "aligned shear" mode did not occur at any point beneath the Kettleman Hills Unit B-19, Phase I-A landfill. Thus the test results in Table 6 for this condition are not of significant interest for investigations of the slippage of March 19. However, the results in Table 6 should serve as a warning of potentially very low shear strengths for this "aligned shear" slippage mode for other liner systems, as it should be noted that this slip-displacement mode can result in unusually low interface-residual-friction angles on the order of $\phi_r \approx 5°$ to $8°$.

Geotextile/Compacted Clay Interface Direct Shear Tests

Three tests were performed to study the shear resistance developed between geotextile/compacted clay liner interfaces. In these tests, the clay liner material was first compacted to field conditions (density and water content), and then soaked for 24 hr under a light surcharge ($\sigma_v \approx 50$–200 psf). After soaking, a larger normal stress was applied, and the interface samples were then sheared without allowing time for consolidation or pore-pressure dissipation. These direct shear tests were intended to represent unconsolidated-undrained (UU) testing conditions, though in fact the geotextile probably facilitated rapid pore-pressure dissipation at the geotextile/clay interface contact. In these tests, the interface shear strengths ($\tau_{f,r}$) increased with increased normal stress and the residual friction angles for all three tests were in excess of 24°.

No attempts were made to test "as compacted" (nonsoaked) clay/geotextile interface samples as it was considered unlikely, based on the results discussed, that shear slippage on a geotextile/compacted clay interface under "as compacted" conditions could have been instrumental in the shear slippage that occurred in the liner system underlying the Kettleman Hills landfill.

Geotextile/Geonet Interface Direct Shear Tests

Six direct shear tests were performed on samples of geotextile/geonet interfaces with the direction of shear slippage aligned parallel to the orientation of the geonet strands in contact with the geotextile (the "aligned shear" mode). The single test performed on a "dry" interface sample (Test Number DD) was halted at small shear displacement to avoid damaging the load cell used to monitor shear force. The shear strength of this interface at the time the test was stopped was observed to be relatively high, certainly in excess of $\phi_p = 20°$.

The five tests on submerged samples showed a slight tendency for peaking of shear strength at small shear displacement, followed by a slight decrease in shear strength to a lower residual friction angle. In all cases, residual friction angles were greater than $\phi_r = 10°$, and most were greater than 12°.

No direct shear tests were performed on geotextile/geonet interface samples sheared in the "transverse shear" mode because it was apparent that this would result in higher interface strengths than those already described.

TABLE 7. HDPE Liner/HDPE Liner Interface Direct Shear-Resistance-Test Results

Condition (1)	Number of tests (2)	Peak friction angle*: ϕ_p (3)	Shear displacement* at ϕ_p (in.) (4)	Residual friction angle*: ϕ_r (5)
Dry	9	9.9° ± 2.2°	0.019 ± 0.014	8.8° ± 2.4°
Submerged	6	9.9° ± 1.8°	0.100 ± 0.119	9.2° ± 1.9°

*Mean values ± standard deviation.

HDPE Liner/HDPE Liner Interface Direct Shear Tests

A number of direct shear tests of HDPE liner/HDPE liner interface samples were performed under both "dry" and "submerged" conditions, and the results of these tests are summarized in Table 7. There was considerable variation in the residual interface-friction angles for samples tested under both "dry" conditions ($\phi_r \approx 6°$ to 13°) and "submerged" conditions ($\phi_r \approx 6°$ to 11°). This 5°–7° variation in residual friction angle was significantly greater than the variations in shear strength observed for any other interface combination tested during these studies. There did not appear to be a significant systematic difference between HDPE liner/HDPE liner interface-shear strengths for interface samples tested under "dry," as opposed to "submerged," conditions.

As discussed previously, the surfaces of some of the HDPE liner samples received for testing were found, upon close scrutiny, to be slightly but visibly discolored. This discoloration manifested itself as a slightly lighter tone (and/or slightly tan tint) to the otherwise deep black HDPE color. No consistent trend in interface-shear strength versus discoloration was observed. It is possible, however, that this slight discoloration was due to abrasion, dust, dirt, or chemical weathering. These and/or other surficial effects may have led to minor variations in surface-roughness characteristics. However, any such variations were not manifested in testing of HDPE liner/geonet interface samples, where the geonet abrasion gouged sufficiently deeply into the HDPE liner samples to overcome any such HDPE liner-surface effects. These variations also did not affect HDPE liner/geotextile or HDPE liner/compacted liner clay interface samples, probably due to a lack of sensitivity to subtle HDPE liner-surface characteristics.

Pullout-Box Interface-Resistance Tests

General

Fig. 5(*a*) shows a schematic illustration of the apparatus used to perform pullout-box tests on an interface sample composed of two materials (A and B). The internal dimensions of the box are 17 in. long by 10 in. wide by 10 in. high. The steps involved in performing these tests were:

1. A sheet of interface material A was affixed to the base of the testing bay, as shown in Fig. 5(*a*).
2. Two strips of interface material B were then cut, each to a width of 1.5 in. and a length of 11 in. These were epoxied back-to-back with the faces to be

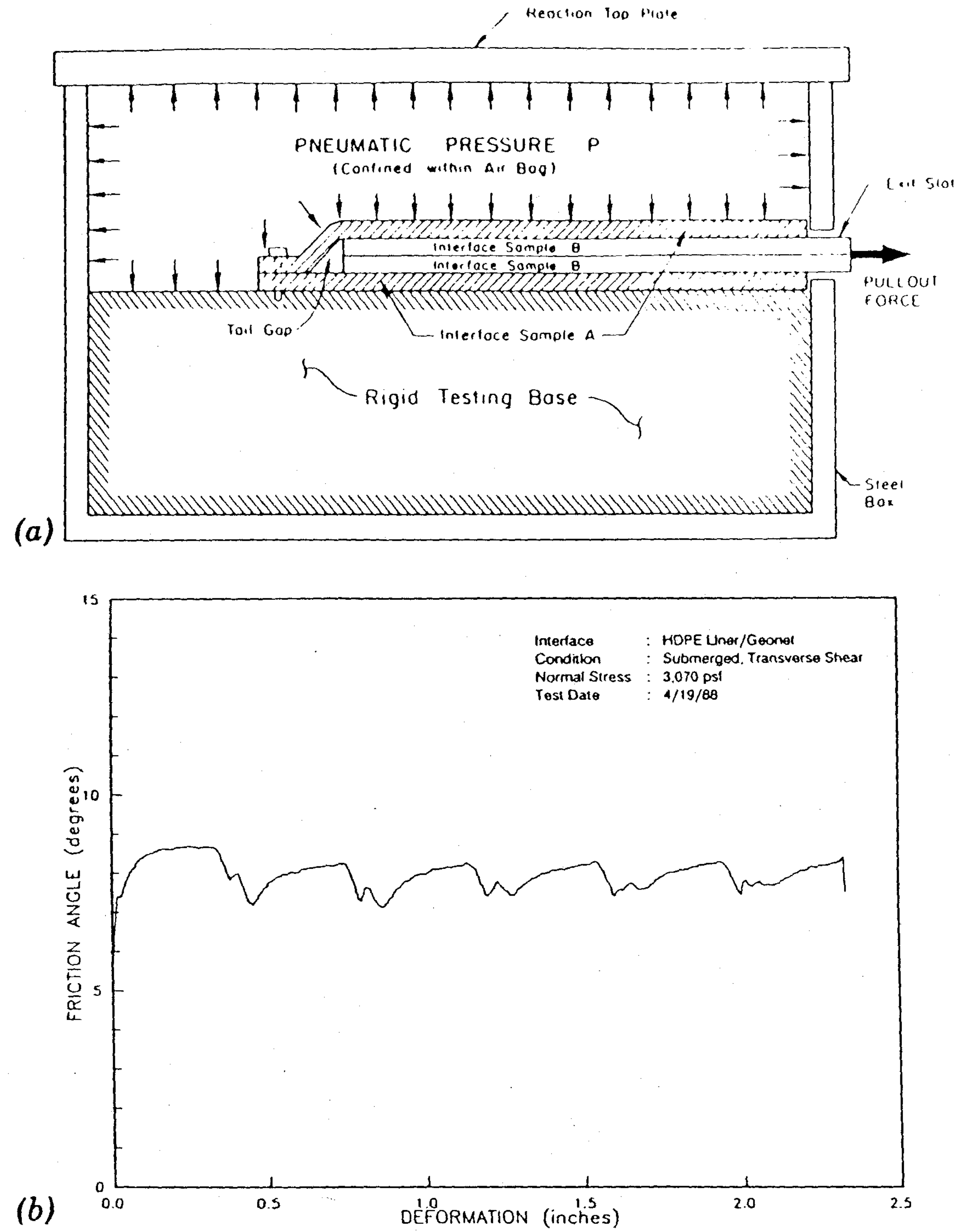

FIG. 5. Pullout-Box Testing-Apparatus Configuration and Example Test Results: (*a*) Schematic Illustration of Pullout-Box Testing Apparatus and Interface-Sample Configuration; and (*b*) Interface Pullout-Box Test Results: Test Number P-16 (HDPE Liner/Geonet)

tested facing outward, and the resulting "pullout strip" was then placed in the testing bay atop the sheet of material A.

3. "Spacing" sheets of material B were placed in pairs (double thickness) closely alongside both sides of the testing pullout strip so that the tops of the upper spacing sheets and the top of the double-thickness pullout strip were at approximately the same elevation.

4. A second sheet of interface material A was then placed on top of the spac-

ing sheets and pullout strips and fixed in place at one end.

5. The top of the pullout box was then bolted into position, and a rubber diaphragm was inflated at the top of the box to apply vertical pressure to the "sandwich" system formed by the underlying two sheets of material B and the double-sided pullout strip.

6. The pullout strip was then pulled out through a slot in the side of the box, at a constant rate, using an electrical winch. Pullout rates used were on the order of 0.15 in. per minute, and this represents the rate of interface-shear displacement. The total pullout force applied was continuously monitored electronically using a strain-gaged load cell, and the pullout displacement was continuously monitored electronically using an LVDT (linear variable differential transformer). Data were automatically collected during testing using a IBM-PC AT microcomputer with A/D and D/A (analog to digital and digital to analog) capacity.

The normal force on the pullout-sample interfaces was monitored and controlled by controlling the pneumatic pressure in the rubber air bag at the top of the pullout-box apparatus. Two corrections were necessary in order to evaluate interface-normal or contact stress based on this applied pressure. There was necessarily a slight lateral gap along both sides of the pullout strip, between the strip and the adjacent spacing sheets. The top sheet of interface material A (and the rubber bladder) bridged this gap so that half of the gap width on each side of the pullout strip represented part of the contributory area for vertical loading of the interfaces. In addition, as shown schematically in Fig. 5(*a*), there was also some bridging (or gap formed) by the top sheet of interface material A immediately beyond the tail end of the pullout strip. Once again, the pneumatic vertical pressure applied to half of the bridging span was assigned to the interface-contact pressures. The significance of this second correction for bridging increased slightly as the test progressed and the pullout strip was pulled out of the box, progressively reducing the remaining interface-contact area.

This pullout box testing of "sandwiched," double-sided pullout strips resulted in a total tested initial interface-contact area of 33 sq in., which was more than four times the surface-contact area tested in the direct shear-interface tests described previously. In addition, the pullout tests could be continued to interface-shear displacements in excess of 3 in., compared with the smaller maximum continuous-shear displacements of less than 0.3 in. that could be achieved in the direct shear-interface tests. As the geosynthetic materials were stiff relative to the applied stresses, slip developed along the full length of the samples.

These pullout-box interface-shear-resistance tests thus served two purposes: First, they provided a check on the results of the direct shear-test results described previously evaluating the potential effects of sample size or sample-interface-contact area; and second, they established the shear-strength versus shear-displacement characteristics for each interface combination to a much larger relative shear displacement than was possible in the direct shear tests.

Tests of interface combinations that included compacted liner clay could not be performed in the pullout-box apparatus. Accordingly, pullout-box tests were performed for the following liner-interface combinations: HDPE liner/

geotextile interface, HDPE liner/geonet interface, and HDPE liner/HDPE liner interface.

HDPE Liner/Geotextile Interface Pullout Box Tests

Two pullout-box interface-shear-resistance tests were performed on HDPE liner/geotextile interface samples tested under "dry" conditions, and eight additional tests were performed on interface samples tested under "submerged" conditions. The peak and residual friction angles were found to be in good agreement with those observed in the direct shear tests, with a range of residual friction angles of $\phi_r \approx 6.5°$ to $10°$.

HDPE Liner/Geonet Interface Pullout Box Tests

All pullout-box tests on HDPE liner/geonet samples were performed under "submerged" conditions. Two tests were performed on interface samples sheared in the "transverse shear" mode, in which the orientation of the geonet strands in contact with the HDPE liner were not closely parallel to the direction of shear displacement; and six tests were performed in the aligned-shear mode, in which the orientation of the geonet strands were closely parallel to the direction of interface shear.

An interesting feature of the pullout-box tests was the ability of the tests to continue to relatively large shear displacements well in excess of the 0.25 in. displacement possible in the direct shear tests. This was significant for the HDPE liner/geonet samples as it permitted observation of the type of shear-strength versus shear-displacement behavior shown in Fig. 5(*b*). As shown in this figure, the interface-shear strength periodically increased and decreased slightly as the geonet/liner contact points tracked within grooves worn into the face of the HDPE liner. The periodic increase and decrease occurs with a constant period (or recurrence interval), which is equal to the spacing between points of liner/geonet contact. Progressive wearing or abrasion does not appear to influence this interface-shear-strength behavior, at least over the several inches of shear displacement that could be evaluated in the pullout-box tests.

Values of the residual angle of friction for the transverse-shear condition ranged from $\phi_r \approx 8°$ to $9°$, and values for the aligned-shear condition ranged from about $\phi_r \approx 6°$ to $8°$. Again, the results were in good agreement with the results of the direct shear tests on HDPE liner/geonet interface samples.

HDPE Liner/HDPE Liner Interface Pullout Box Tests

Six pullout-box tests were performed on HDPE liner/HDPE liner interface samples tested under "submerged" conditions. As in the direct shear tests of HDPE liner/HDPE liner interface samples discussed previously, a large range of variation in residual friction angles ($\phi_r \approx 7°$ to $13.5°$) was observed.

Summary of Liner Interface Shear Strength Test Results

Table 8 presents a summary of the liner-interface shear-strength-test results for tests performed using both the direct shear and pullout-box testing apparatus. As shown in this table, there is very good agreement between the residual interface-shear-strength characteristics measured in direct shear and pullout-box tests for all interface combinations subjected to pullout-box testing. This adds confidence to the use of the simpler direct shear box tests

TABLE 8. Summary of Interface-Shear-Strength Tests: Kettleman Hills Repository

Interface components (1)	Conditions (2)	Direct Shear Tests: Residual friction angle: ϕ_r (3)	Pullout-Box Tests: Residual friction angle: ϕ_r (4)	Values proposed for stability analyses (5)
HDPE liner/geotextile	Dry unpolished	9.5° to 12.5°	9.5° (1 Test)	$\phi = 9° \pm 1°$
	Dry, partly	9.0° to 11.0°	—	
	polished	8.5° to 10.5°	8.0° (1 Test)	
	Dry, polished			
	Wet, unpolished	8.0° to 10.0°	7.0° to 10.5°	$\phi = 8° \pm 1°$
	Wet, polished	7.0° to 9.5°	6.5° to 9.0°	
HDPE liner/clay	Dry (as compacted)	11.0° to 14.0°	—	$\phi = 12° \pm 1°$
	Saturated (UU)	(τ_r = 800 to 1,000 psf)	—	τ = 900 psf ± 250 psf
HDPE liner/geonet (transverse shear)	Dry	7.0° to 8.0° (+)	—	$\phi = 8.5° \pm 1°$
	Submerged	7.0° to 10.0°	8.0° to 9.0°	$\phi = 8.5° \pm 1°$
HDPE liner/geonet (aligned shear)	Submerged	5.0° to 8.0°	6.0° to 8.0°	$\phi = 7.0° \pm 1.5°$
Geotextile/clay	Saturated (UU)	~24°	—	$\phi \approx 24°$
Geotextile/geonet	Dry	>20°	—	$\phi > 20°$
	Submerged	10° to 14° (+)	—	$\phi > 12°$
HDPE liner/HDPE liner	Dry	6.0° to 13.0°	—	$\phi = 9.5° \pm 3°$
	Submerged	6.0° to 11.0°	7.0° to 13.5°	$\phi = 8.5° \pm 2.5°$

for determination of interface strength.

Most of the interface combinations tested showed a slight tendency for "peaking" in interface-shear strength at very small shear displacement, followed by a slight reduction in shear strength to a slightly lower residual interface-shear strength. The reduction from peak to residual shear strength was relatively small for all interface combinations of interest. Because the peak strengths occurred at very small relative interface-shear displacements (typically less than 0.1 in. shear displacement) that are likely to be exceeded by deformations occurring during construction and fill-placement operations, the residual interface-shear-strength behavior was taken as representative of the interface-shear-strength characteristics of each interface combination for purposes of slope-stability analyses. Values proposed for use in stability analyses are given in column 5 of Table 8.

This residual interface-shear strength was best characterized in terms of the residual friction angle (ϕ_r) for all interface combinations and conditions, with the exception of the HDPE liner/compacted clay interface tested under presoaked, unconsolidated-undrained (UU) conditions. As discussed previously, the shear strength of this HDPE liner/compacted clay under presoaked (saturated UU) conditions was found to be independent of applied total interface-normal stress (σ_n), and was therefore best characterized directly in terms of residual interface-shear strength (τ_r).

The HDPE liner/geonet interface-shear-displacement mode corresponding to "aligned shear" did not occur beneath the Kettleman Hills Unit B-19, Phase I-A landfill. Among the remaining interface combinations and conditions of potential interest, it can be seen in Table 8 that the geotextile/clay interface, the geotextile/geonet interface (both dry and submerged), and the HDPE liner/clay interface (dry only) all have significantly higher residual interface-shear strengths than do the remaining interface combinations and conditions. The remaining interface combinations and conditions, which are thus of primary interest for the studies of the Unit B-19, Phase I-A slope failure of March 19, 1988, are HDPE liner/geotextile, HDPE liner/geonet, and HDPE liner/clay under presoaked UU conditions.

As shown in Table 8, two of these critical interface combinations have approximately equal residual interface-friction angles under "dry" conditions. These are HDPE liner/geotextile ($\phi_r \approx 9° \pm 1°$) and HDPE liner/geonet ($\phi_r \approx 8.5° \pm 1°$). It is therefore not possible to ascertain with any reasonable degree of certainty which of these two interface combinations represents a more critical potential shear failure mechanism under "dry" conditions within the multilayer liner system of the Phase I-A landfill. Regardless of which of these two interface combinations is most critical, a representative value of $\phi_r \approx 8.5°$ appears reasonable for analysis of shearing within the multilayer liner system under "dry" conditions.

These same two interface combinations also exhibit approximately equal residual interface-friction angles when both are sheared under "submerged" conditions. These residual friction angles are $\phi_r \approx 8° \pm 1°$ for the HDPE liner/geotextile interface, and $\phi_r \approx 8.5° \pm 1°$ for the HDPE liner/geonet interface. Again, it is not possible to ascertain with any reasonable degree of certainty which of these two interface combinations represents a more critical potential failure mechanism within the multilayer liner system underlying the Phase I-A landfill. Regardless of which of these two interface combinations is most critical, a representative value of $\phi_r \approx 8°$ can be used for analysis of shearing within the liner under "submerged" interface conditions.

This evaluation of shear strengths within the multilayer liner system is further complicated by the possibility of shear slippage along HDPE liner/compacted liner clay interfaces in zones where the clay and the HDPE liner/clay interface is "wetted." This wetting may occur as a result of a number of events, including rainfall during construction, squeezing of water from the clay itself during consolidation under large fill overburden, thermal effects leading to collection of water on the underside of the HDPE liner during initial liner placement, and wetting associated with water ponding in the vicinity of the leachate-collection-system sump.

If the clay at the HDPE liner/clay interface became wetted at one or more points beneath the Unit B-19 landfill, then the subsequent dissipation of pore pressures during consolidation would be a very slow process, controlled by a coefficient of consolidation on the order of $c_v \approx 2$ sq ft/yr, which was determined by consolidation tests on compacted liner clay. Based on this low coefficient of consolidation, the rate of consolidation and pore-pressure dissipation within the compacted clay liner layers would be very slow, and for the 3.5 ft thick primary clay liner the pore-pressure dissipation at the HDPE liner/clay interface over a period of one year was estimated to be less than 5%. Accordingly, any zones in which the compacted liner clay became wetted either before or during fill placement can be assumed to be well represented by the interface-shear-strength tests performed on HDPE liner/compacted clay liner interface samples under presoaked, unconsolidated-undrained (UU) conditions, with an average interface shear strength of $\tau_r \approx 900$ psf.

The average strength for the five samples tested under these conditions was about 900 psf, with values ranging from about 800 to 1,000 psf. Thus, the average value of 900 psf was selected for use in the analyses. However, it should be recognized that the variability in soil properties is likely to be significantly higher than that determined for manufactured materials such as geotextiles, and thus actual values for the unconsolidated-undrained strength at the clay/geomembrane interface, allowing for possible variations in borrow-source material, soil composition and placement conditions are likely to exceed the ±100 psf range indicated by these limited data. In view of these factors, allowance for a possible variation of ±250 psf from the average measured value of 900 psf would seem to provide a more appropriate range of values to be considered in evaluating the effects of the resistance developed at this interface.

It should be noted that the residual interface strength of $\tau_r \approx 900$ psf could only have occurred in areas beneath the Unit B-19, Phase I-A landfill in which the compacted clay at the HDPE liner/compacted clay liner interface became "wetted." Unconsolidated-undrained compression tests on the clay in its "as compacted" state gave shear strength values greater than 2,000 psf, considerably greater than the critical strengths on other interfaces in the liner system, and the interface shear strength behavior for HDPE liner/compacted clay liner interface specimens sheared under "as compacted" conditions also showed higher strengths.

In summary, based on the interface shear strength testing program performed to evaluate conditions in the multilayer liner underlying the Unit B-19, Phase I-A landfill, the interface shear strengths that seem to be most appropriate for analyses of the slope failure of March 19, 1988 are:

- For "dry" liner-interface conditions, assumed representative of the sloping sides of the waste fill basin, a value of $\phi_r \approx 8.5°$.
- For "submerged" or at least moist liner-interface conditions, assumed representative of most of the nearly level base of the waste fill basin (grade ≈ 2%), a value of $\phi_r \approx 8°$ where the frictional resistance controls the location of the critical sliding surface.
- For "submerged" liner-interface conditions in zones in which apparent wetting of the clay liner occurred, a value of $\tau_r \approx 900$ psf for zones of high fill overburden where $\tau_r \approx 900$ psf represents a more critical failure mechanism than $\phi_r \approx 8°$.

These values are used in the stability analyses described in a second paper (Seed et al. 1990) that investigates the cause of the slope failure.

Conclusions

The following conclusions can be made regarding the liner systems associated with the slope-stability failure of the Kettleman Hills Unit B-19, Phase I-A landfill.

1. The materials used to construct the protective multilayer liner system at the Kettleman Hills facility involved contact surfaces between various geosynthetics including sheets of HDPE liner, geonet, and geotextile; and between these materials and compacted clay liner. The interfaces between these materials are characterized by low frictional resistance.
2. The frictional resistance is affected by various properties, including the

TABLE 9. Representative Measured Values of ϕ_r and Shear Strength

Interface (1)	Shear Strength τ_r or Residual Angle of Friction, ϕ_r — Dry condition (2)	Wet condition (3)
HDPE liner/geotextile	$\phi_r \approx 9° \pm 1°$	$\phi_r \approx 8° \pm 1°$
HDPE liner/geonet	$\phi_r \approx 8.5° \pm 1°$	$\phi_r \approx 8.5° \pm 1°$
HDPE liner/clay	—[a]	$\tau_r \approx 900$ psf $\pm$ 250 psf
Geotextile/geonet	$\phi_r > 20°$	$\phi_r > 12°$
Geotextile/clay	—[a]	$\phi_r \approx 24°$

[a]Not available.

degree of polishing, whether the interfaces are wet or dry, and in some cases the relative orientation of the layers to the direction of shear-stress application. There are also some small variations in properties between one batch and another of the HDPE liner and geonet materials. The values of interface friction for the materials from the Kettleman Hills liner system were not significantly influenced by the magnitude of normal stress.

3. It is likely that in some areas the compacted clay is saturated at its contact surface with the liner, and since some wetting may have occurred during construction, significant pore-water pressures could still exist at the compacted clay liner interface. Estimates based on the consolidation characteristics of the clay indicate that these pore pressures may be almost as large as the overburden pressure of the overlying materials. Tests indicate that under these conditions the shear resistance at the HDPE liner/clay interface would be on the order of $\tau_r \approx$ 900 psf.

4. In all cases, the minimum ultimate or residual frictional resistance is fully mobilized at very small deformation levels that are likely to be exceeded by deformations occurring during construction and fill-placement operations. Thus for stability-analysis purposes the frictional resistance can be expressed by the residual angle of friction, ϕ_r, or the residual shear strength τ_r. For the materials used at the Kettleman Hills facility, representative values of the measurements of ϕ_r and shear strength for the different interfaces involved in the liner system are given in Table 9. The values in Table 9 are those considered most appropriate for analyses of the cause of failure in this case and for the stability analyses. It should be noted that where less extensive test data are available, and to allow for uncertainties in material characteristics, somewhat lower values than those indicated above would probably be more appropriate for design purposes.

5. The most critical interface combinations controlling the stability of the fill and liner system at the facility are apparently those between HDPE liner/geotextile, HDPE liner/geonet, and HDPE liner/compacted clay (saturated).

6. Because of the very similar strength parameters of the HDPE liner/geotextile, HDPE liner/geonet, and HDPE liner/compacted clay interfaces within the multilayer liner system underlying the base and sloping sides of the waste deposit, it is not possible to determine with any high degree of assurance which of these surfaces was the one on which sliding actually occurred.

7. The variation in measured interface-shear-strength parameters for the various liner-system interfaces investigated in the testing program indicates the de-

sirability of performing similar test programs for proposed new facilities to establish design parameters, until such time as more data and experience are available.

Acknowledgments

The writers thank Clarence K. Chan, Richard C. Sisson, Makram Jaber, Mu Hsiung Chang, and Peter G. Nicholson of the University of California, Berkeley Geotechnical Group who assisted in the testing program.

Appendix I. References

Martin, J. P., Koerner, R. M. and Whitty, J. E. (1984). "Experimental friction evaluation of slippage between geomembranes, geotextiles and soils," *Proc. Int. Conf. Geomembranes*, Denver, Colo., Jun. 20–23, pp. 191–196.

Seed, R. B., Mitchell, J. K., and Seed, H. B. (1990). "Kettleman Hills waste landfill slope failure. II: Stability analyses," *J. Geotech. Engrg.*, ASCE, 116(4), 669–690.

Appendix II. Conversion to SI Units

To convert	To	Multiply by
acres	ha	0.405
cu ft	m^3	0.0283
ft	m	0.3048
gal	L	3.785
in.	cm	2.54
lb	kg	0.4536
psf	kPa	0.04788