Kettleman Hills Waste Landfill Slope Failure. II: Stability Analyses

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

Abstract: Analyses were made to determine the cause of a stability failure of a 90 ft high, 15 acre hazardous waste landfill in which lateral displacements of up to 35 ft and vertical settlements of up to 14 ft were measured. The failure developed by sliding along interfaces within the composite geosynthetic–compacted-clay liner system beneath the waste fill. The shear resistances of the different interfaces in the liner system were determined by direct shear and pullout tests as described in a companion paper (Mitchell et al. 1990). Conventional two-dimensional (2D) stability analyses of representative cross sections and three-dimensional (3D) analyses of the overall waste fill and liner configuration are described. Each type of analysis was applied to two cases: (1) The "Probable Minimum Clay/Liner Wetting Case," in which shear along a wetted HDPE liner/compacted clay interface was assumed to occur only over a small area of the base; and (2) the "Full-Base-Wetting Case," in which the HDPE liner/compacted clay liner interface was assumed to have become "wetted" over the full central base of the fill basin. The 2D stability analyses gave factors of safety of 1.2–1.25 and 1.1–1.15 for the minimum wetting case and the full base wetting case, respectively, while the 3D analyses yielded values of 1.08 and 1.01 for these two cases. Uncertainties in the strength parameters and analysis methods lead to a best estimate of the computed factor of safety at the time of failure of 0.85–1.25. This provides good agreement with the observed field performance, and suggests that the techniques used to evaluate liner-interface shear strengths and to perform overall stability analyses may be appropriate for future evaluation of other, similar lined waste-repository fills.

Introduction

Landfill Unit B-19, covering an area of about 36 acres, forms p
Class I hazardous-waste treatment-and-storage facility at Kettlema
California. The waste repository essentially consists of a very larg
shaped bowl excavated in the ground to a depth of about 100 ft, int
the waste fill is placed. The "bowl" has a nearly horizontal base, a
slopes of 1 on 2 or 1 on 3. To prevent the escape of hazardous n
into the underlying and surrounding ground, the base and sides of
cavation are lined with a multilayer system of impervious geomem
clay layers and drainage layers. An overall view of the facility as it appeared on March 15, 1988 after about one year of fill placement is presented in Fig. 1 of the companion paper by Mitchell et al. (1990).

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

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

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

[3]Deceased, formerly Cahill Professor 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 0733-9410/90/0004-0669/$1.00 + $.15 per page. Paper No. 24578.

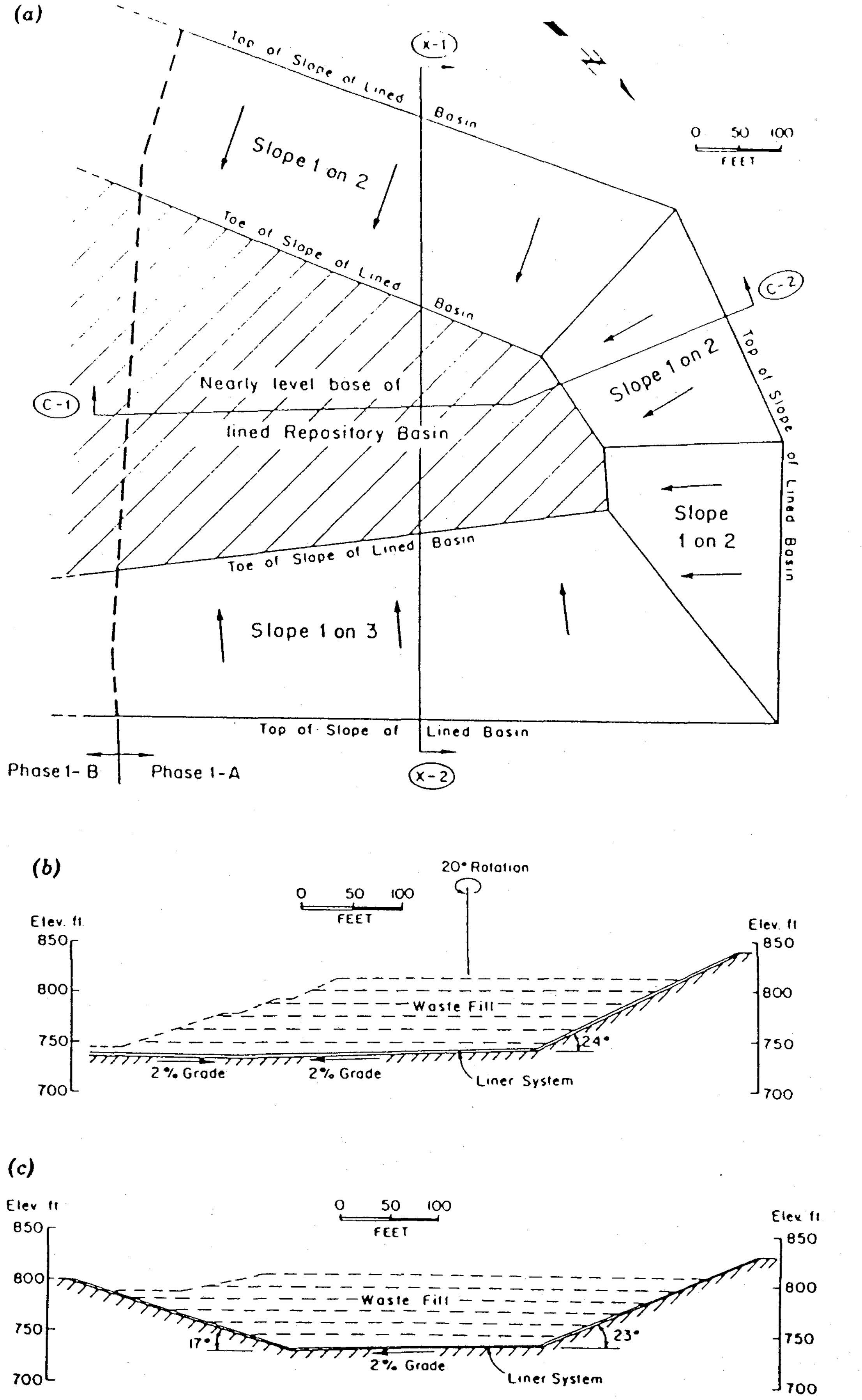

FIG. 1. Landfill B-19, Phase 1-A Cross Sections C-1/C-2 and X-1/X-2: (*a*) Plan View of Lined Repository Basin; (*b*) Cross Section C-1/C-2 (Schematic); (*c*) Cross Section X-1/X-2 (Schematic)

the facility in early 1987. At the same time, the liner systems for other phases of the project were being completed. A plan view of the Phase 1-A section of the "bowl" and representative sections showing the configuration of the liner and fill as it existed on March 19, 1988, are shown in Fig. 1.

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 were tears and displacement on the exposed portions of the liner system (shown in Fig. 2). The prefailure surface topography, as it existed on March 15, 1988 (four days before the failure) is shown in Fig. 3(*a*) and the postfailure topography is shown in Fig. 3(*b*). Vectors showing the directions and magnitudes of the horizontal movements are shown in Fig. 4. The maximum fill height at the time of failure was about 90 ft.

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 made to determine the cause of the failure are the subject of this and a companion paper (Mitchell et al. 1990). The companion paper contains a more complete description of the failure and a description of the testing program conducted to evaluate the shear resistances along interfaces in the com-

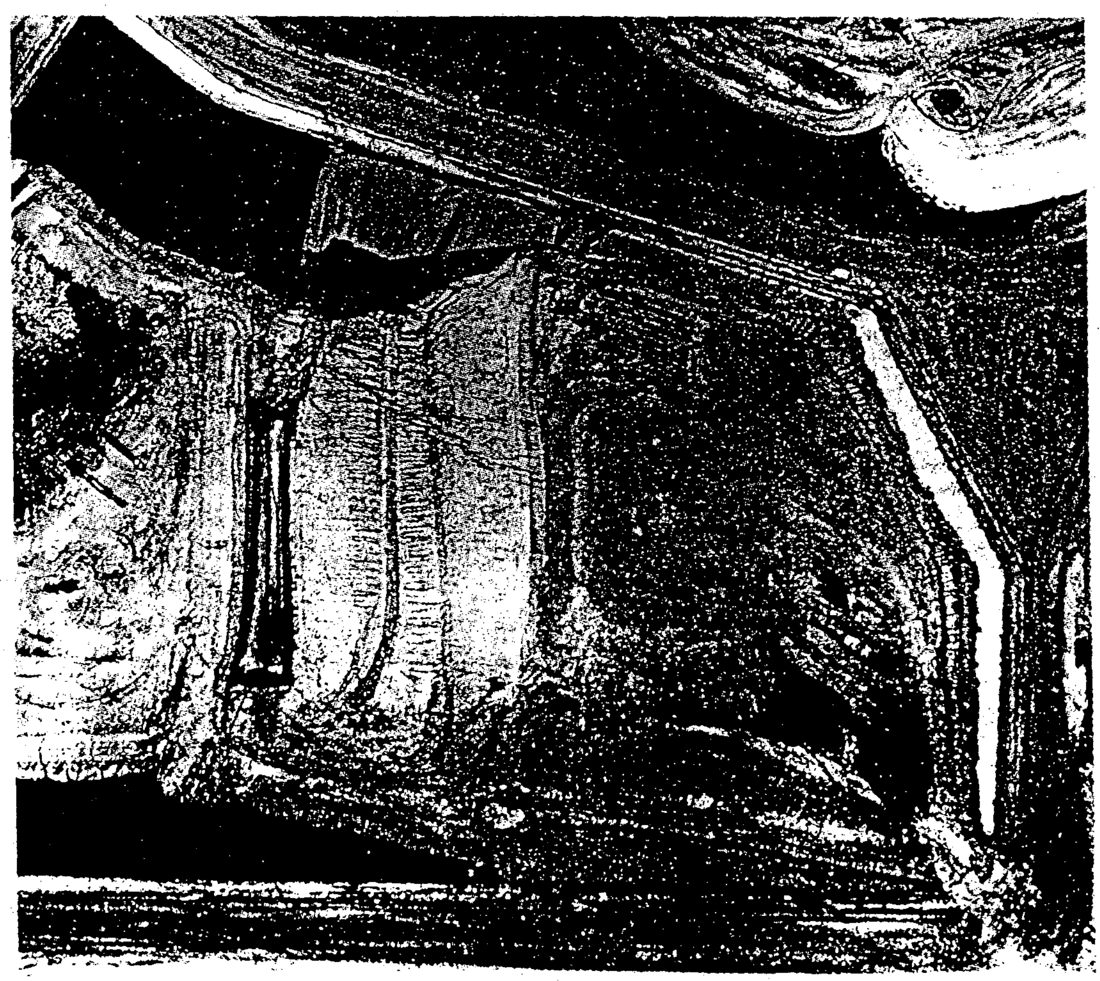

FIG. 2. Postfailure Air Photo of Landfill

TJFA 442
PAGE 003

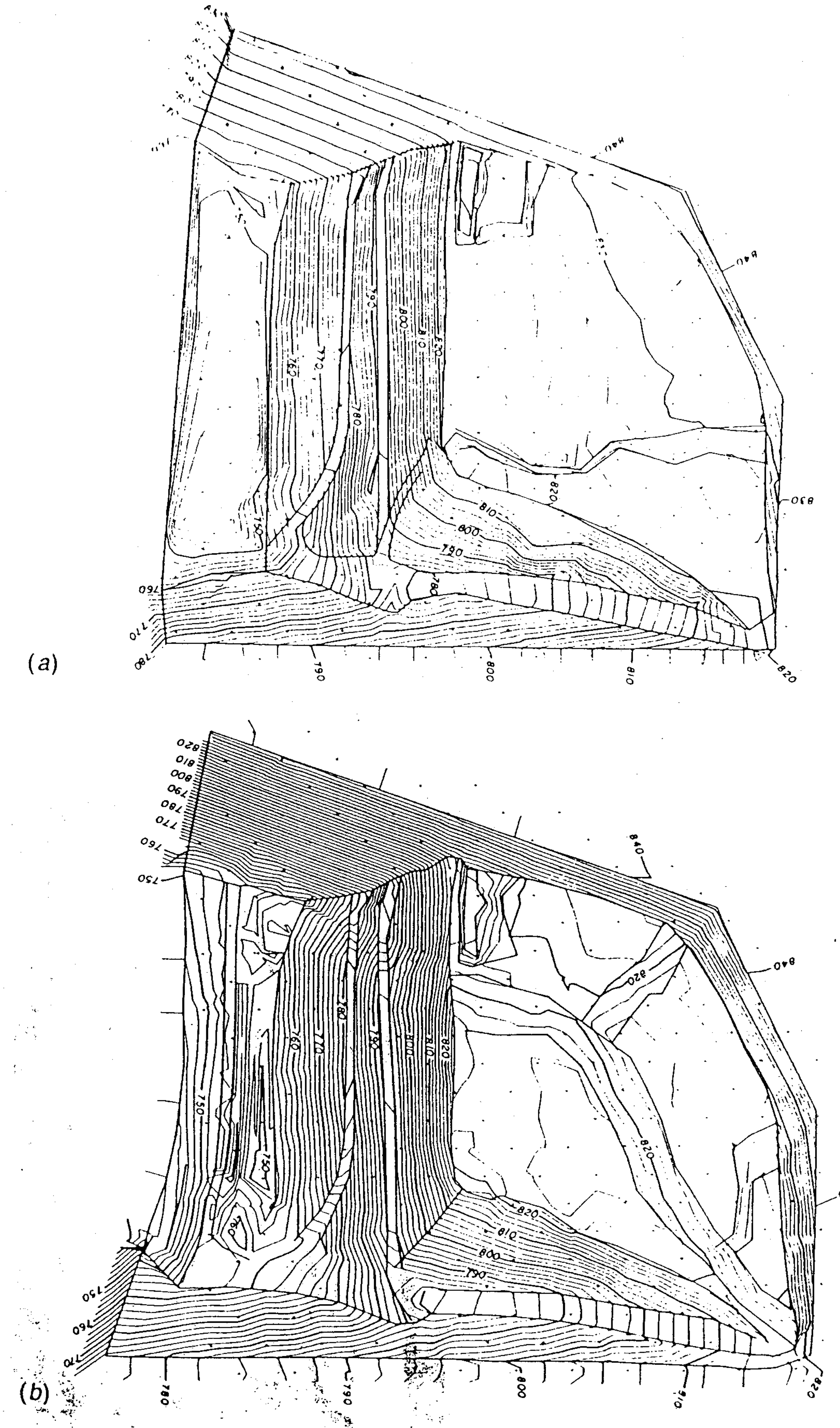

FIG. 3.—Surface Topography of Unit B-19, Phase 1-A Landfill: (*a*) Preslide Topography on March 15, 1988; (*b*) Postslide Topography

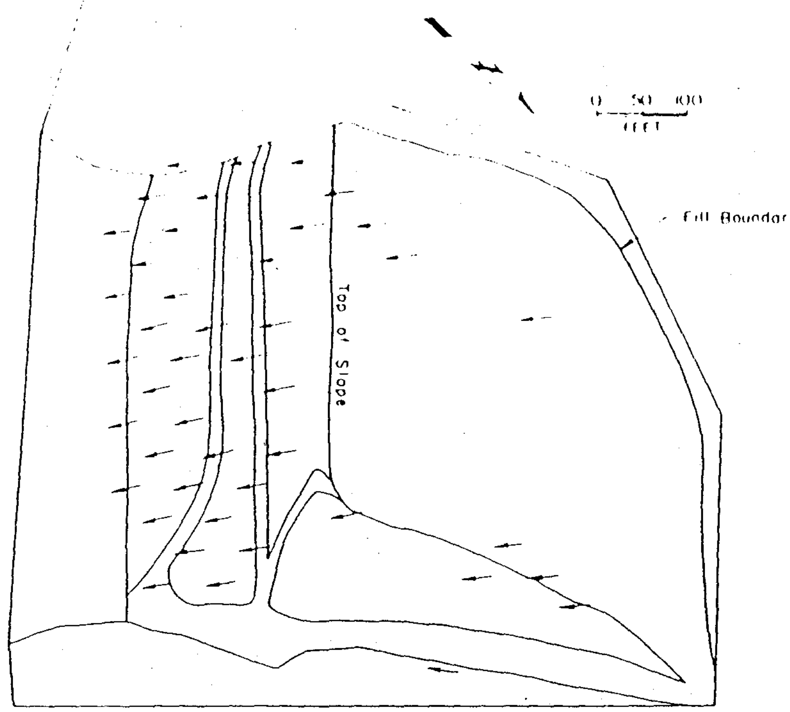

FIG. 4. Plan View Showing Displacement Vectors Representing Measured Lateral Movements Due to Slide

posite liner system. Conclusions are drawn concerning the properties appropriate for evaluation of the waste-landfill stability. In the present paper, these properties are used in stability analyses to provide a probable explanation for the cause of the failure and to investigate the applicability of the test results and analysis procedures for predicting the stability of other waste fills placed in this type of liner-protected facility.

Interface Strengths in Liner System

The composite flexible-membane–compacted-clay double-liner system under the Phase 1-A portion of the B-19 landfill is described and illustrated in the companion paper (Mitchell et al. 1990). The system included geotextile filter fabric, granular leachate-collection layers, plastic geonet-drainage layers, HDPE (high-density polyethylene) geomembrane liners, and compacted clay layers.

There was some wetting of the compacted clay by rainfall during construction and fill placement, and there was some consolidation of the clay during the period of waste-fill placement. Since the waste fill was relatively dry, the regular pumping of water from the leachate-collection system can be interpreted as indicating that the placement conditions, combined with the environmental conditions and the pressure of the waste deposit, had led to a condition of essentially full saturation of the compacted base-liner clay, at least over some portion of the base of the fill, sometime after fill placement had started. As a result, it was necessary to investigate a large number of interface conditions existing within the liner system to determine the most critical surfaces from a stability point of view.

These studies (Mitchell et al. 1990) led to the conclusion that the critical

TABLE 1. Liner Interface Shear Strengths Used for Analyses of Slope Failure

Interface components (1)	Dry interface conditions (2)	Submerged interface conditions (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 (presoaked, UU)	—[a]	$\tau_r \approx 900$ psf $\pm$ 250 psf
Values used for analysis	$\phi_r = 8.5°$	$\phi_r = 8.0°$ or $\tau_r = 900$ psf

[a]Not available.

interfaces within the liner system, as indicated by the testing program, were as follows:

1. Between HDPE liner and geotextile layer.
2. Between HDPE liner and geonet layer.
3. Between HDPE liner and compacted clay layer.

The relevant angles of friction or interface-shear strengths determined by the testing program for these three interface combinations are summarized in Table 1. On this basis, values of interface-shear strengths considered appropriate for analyses of the slope failure of March 19, 1988 were:

- For "dry" liner-interface conditions, the assumed representative value of the sloping sides of the waste fill basin was $\phi_r \approx 8.5°$.
- For "submerged" or at least moist liner-interface conditions, the assumed representative value for most of the nearly level base of the waste-fill basin (grade $\approx$ 2%) was $\phi_r \approx 8°$ where 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 $\tau_r = 900$ psf represents a more critical failure mechanism than $\phi_r \approx 8°$) was assumed.

Slope-Stability Analyses

Fig. 5 shows a plan view of the Kettleman Hills Unit B-19, Phase I-A landfill for the conditions existing on March 15, 1988, shortly before the observed slope failure of March 19, 1988. This topography represents a simplification of the more detailed topographical contour map presented in Fig. 3(*a*).

To understand the mechanisms involved in the observed sliding of March 19, 1988, it is necessary to examine the configuration of the underlying multilayer liner system as well as the surface landfill contours and configuration. Fig. 5 shows the principal preslide surface topography, and also (with dashed lines) the principal grade breaks in the underlying liner system. The Phase I-A landfill represented a partial infilling of a bowl-shaped, lined basin (see Fig. 1). The sides and base of the basin were lined with the multilayer liner system and were thus subject to potential shear slippage in view of the low liner interface-shear strengths.

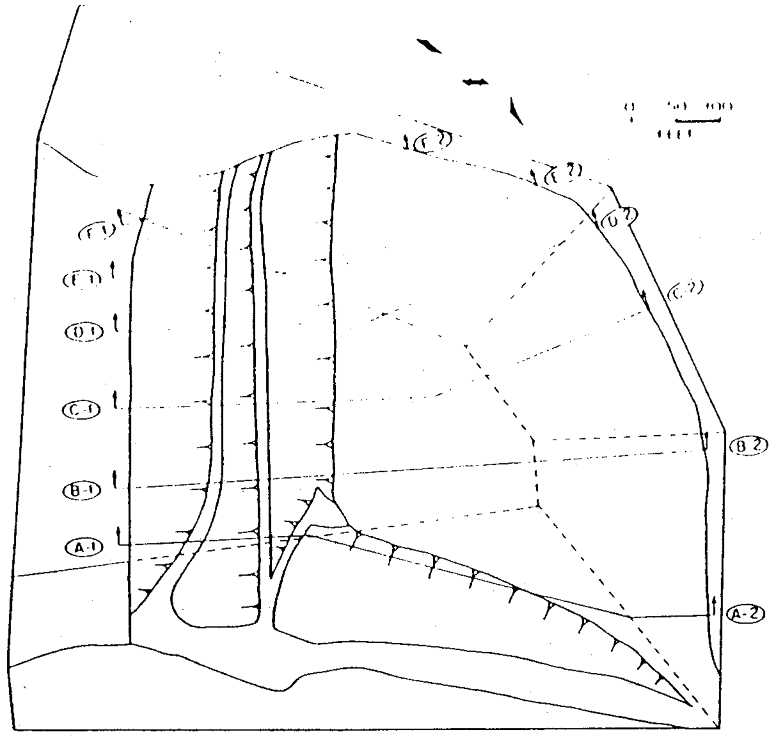

FIG. 5. Plan View of Landfill Showing Locations of Cross Sections A-1/A-2 through F-1/F-2

The nearly level base of the fill basin sloped at a slight grade of approximately 2% toward a leachate sump and collection system. The faces of the liner system at the sides of the fill basin were inclined at slopes of approximately 2:1 (horizontal:vertical) on the west, northwest and north sides of the fill basin, and at approximately 3:1 on the east side of the fill basin. The surface of the waste fill itself was highest at the northwest end of the fill basin, with a nearly level surface that extended to approximately the middle of the Phase I-A basin and then descended to the base of the fill basin with a sloping face traversed by two roads or bench cuts, as shown in Fig. 5. The average slope of this southeastern fill face is approximately 3:1. The landfill surface also descended with a sloping face toward the east face of the lined basin in order to accommodate an access road that entered the landfill area at the north corner of the lined fill basin and descended to the southeastern toe of the landfill along the contact between the eastern fill toe and the east face of the lined fill basin. The maximum depth of the waste fill occurred near the center of the base of the lined basin, at the top of the southeast-sloping fill face. The fill depth was on the order of 90 ft at this location.

Properties Used in Stability Analyses

All stability analyses performed as part of these studies were based on an assumed unit weight of γ = 110 lb/cu ft for the waste landfill, which had been indicated as a reasonable average for the wastes and soil cover placed in the repository. All principal sliding surfaces were considered to occur within the multilayer liner system underlying the waste fill, since the foundation soil at the site was much stronger than the critical liner-system in-

TABLE 2. Liner-Interface Combinations and Shear-Strength Parameters

Interface location (1)	Interface shear-strength parameter (2)
Sloping sides of lined basin	$\phi_r \approx 8.5°$
Nearly level base of lined basin	$\phi_r \approx 8°$ or $\tau_r \approx 900$ psf

terfaces. Accordingly, it was not necessary to develop accurate estimates of shear-strength parameters for either the waste-fill deposit or the underlying natural ground.

Shear-strength parameters for the failure planes within the multilayer liner system underlying the sides and base of the waste fill were selected based on the laboratory investigations described by Mitchell et al. (1990) and summarized in Table 1. Based on direct shear and pullout-box tests of various liner-interface combinations, the shear-strength parameters considered appropriate for stability analyses are given in Table 2.

The residual friction angle of $\phi_r \approx 8.5°$ used for the side slopes represents interface shear under nonsaturated-interface conditions, along either an HDPE liner/geotextile interface or an HDPE liner/geonet interface.

At the base of the lined basin, two different interface sliding mechanisms are hypothesized, and either may be more critical than the other depending on fill overburden and assumptions regarding "wetting" of the HDPE liner/compacted clay liner interface. One possible critical base-interface sliding mechanism consists of sliding on either an HDPE liner/geonet interface or an HDPE liner/geotextile interface. For either of these two interface combinations, a representative residual friction angle is $\phi_r \approx 8°$. This represents the most critical shear-failure mechanism at the base of the lined fill basin in all areas where the HDPE liner/compacted clay liner interface did not become "wetted" during liner construction or subsequent waste fill placement. This frictional base-shear mechanism will also represent the most critical shear-failure mechanism, even in zones where HDPE liner/compacted clay liner interface "wetting" did occur, so long as the waste-fill overburden stress is less than approximately 6,400 psf, at which point the interface-shear resistance due to a residual friction angle of 8° is equal to the residual strength of a "wetted" HDPE liner/compacted clay liner interface ($\tau_r \approx 900$ psf) as determined by

$$\sigma_N \cdot \tan \phi_r = \tau_r \quad \text{(1a)}$$

i.e.

$$(6{,}400 \text{ psf}) \cdot \tan 8° = 900 \text{ psf} \quad \text{(1b)}$$

Based on an assumed unit weight of 110 lb/cu ft, an overlying waste-fill height of approximately 58 ft is necessary to produce an overburden stress of 6,400 psf. Accordingly, a residual friction angle of $\phi_r \approx 8°$ represents the most critical potential shear-failure mechanism at the base of the lined fill basin: (1) In zones where no "wetting" of the HDPE liner/compacted clay liner interface occurred; and (2) in zones where the thickness of the overlying waste fill was less than 58 ft. Based on the test results reported previously, a residual shear strength of $\tau_r \approx 900$ psf, representing shear along a "wetted" HDPE liner/compacted clay liner interface, was considered to represent the most critical potential shear-failure mechanism at the base of the lined fill basin in zones where two conditions were both met: (1) "wetting" of the HDPE liner/compacted clay liner interface occurred; and (2) the thickness of the overlying waste fill was greater than 58 ft.

It should be noted that HDPE liner/compacted clay liner interface tests under presoaked, unconsolidated-undrained conditions have been interpreted as a measure of the shear resistance developed at the interface. It is possible

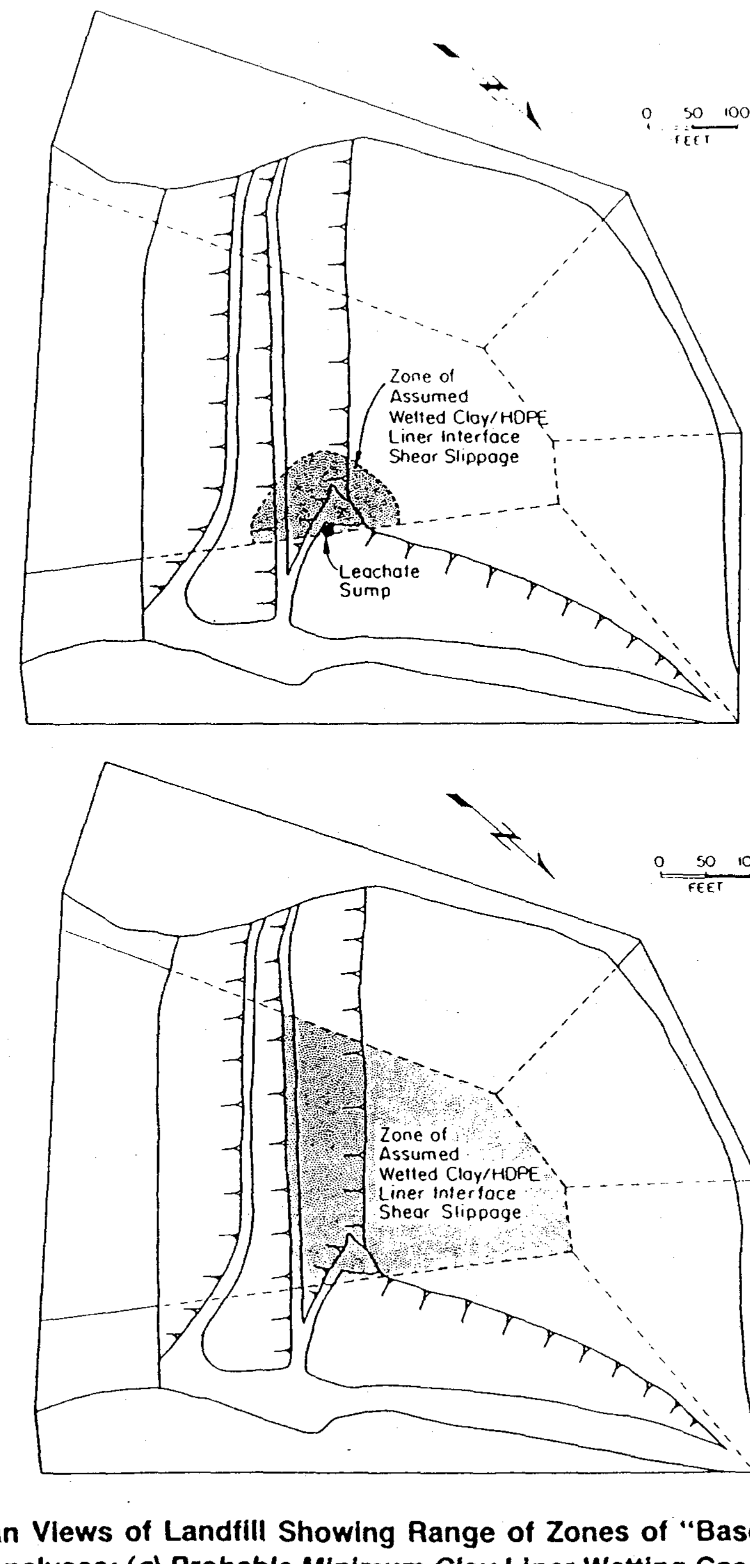

FIG. 6. Plan Views of Landfill Showing Range of Zones of "Base Wetting" Assumed for Analyses: (*a*) Probable Minimum Clay Liner Wetting Case (*b*) Full-Base-Wetting Case

that the resistance to sliding on a plane totally within the presoaked clay was less than the interface value. If this were the case, failure would have developed in these tests along a plane adjacent to the interface rather than at the interface, and the measured strength of about 900 psf would be representative of failure through the soaked clay adjacent to the interface. Thus the measured strength of about 900 psf is representative of either of these possible failure mechanisms under these test conditions.

It should also be noted that a number of mechanisms can be hypothesized as possibly resulting in "wetting" of the HDPE liner/compacted clay liner interface. These include rainfall during liner placement, squeezing of water from the liner clay itself during consolidation under the fill overburden pressures, and/or wetting associated with water ponding in the vicinity of the leachate collection system sump. As it is not possible to reliably predict the actual extent of HDPE liner/compacted clay liner interface wetting that occurred, all stability analyses were performed for two possible cases. The first of these was a "Probable Minimum Clay Liner Wetting Case" involving wetting of the HDPE liner/compacted clay liner interface only in the vicinity of the leachate-collection sump, as illustrated in Fig. 6(*a*). The "wetted" zone shown in this figure represents the probable minimum zone of HDPE liner/compacted clay liner interface wetting based on records of leachate-ponding levels in the sump. The "wetted" HDPE liner/compacted clay liner interface-shear strength of τ_r = 900 psf would represent the most critical shear-failure mechanism within this wetted zone only to the north of the second (upper) access road across the southeast face of the waste fill, as the fill depth to the south of this road is less than 58 ft.

The second case analyzed was the "Full-Base-Wetting Case," involving wetting over the full base of the lined waste basin, but not the sloping sides of the basin. This corresponds to the "worst case" or most critical probable conditions. For the Full-Base-Wetting Case, the "wetted" HDPE liner/compacted clay liner interface shear strength of τ_r = 900 psf would again represent the most critical potential shear-failure mechanism only to the north of the second (upper) access road across the southeast face of the waste fill [as shown in Fig. 6(*b*)], as the fill depth to the south of this road is less than 58 ft.

Two-Dimensional Stability Analyses

Probable Minimum Clay/Liner Wetting Case

Ten representative cross sections were selected for stability analysis. Six of these sections were located as shown in plan view in Fig. 5. These six cross sections, Sections A-1/A-2 through F-1/F-2, are shown in Fig. 7. The cross sections through the central portions of the landfill (Sections B-1/B-2, C-1/C-2, and D-1/D-2) are characterized by an active wedge or driving block at the northwest end of each cross section [see for example Block #1 in Fig. 7(*b*)], and a passive resisting block at the southeast end [see for example Block #2 in Fig. 7(*b*)]. It may be noted that the section of the passive block between sections XX and YY in Figs. 7(*b*), (*c*), and (*d*), becomes significantly smaller in moving from Section B-1/B-2 through C-1/C-2 and D-1/D-2, resulting in progressively decreasing passive resistance to sliding. At the northeast and southwest sides of the landfill, the cross

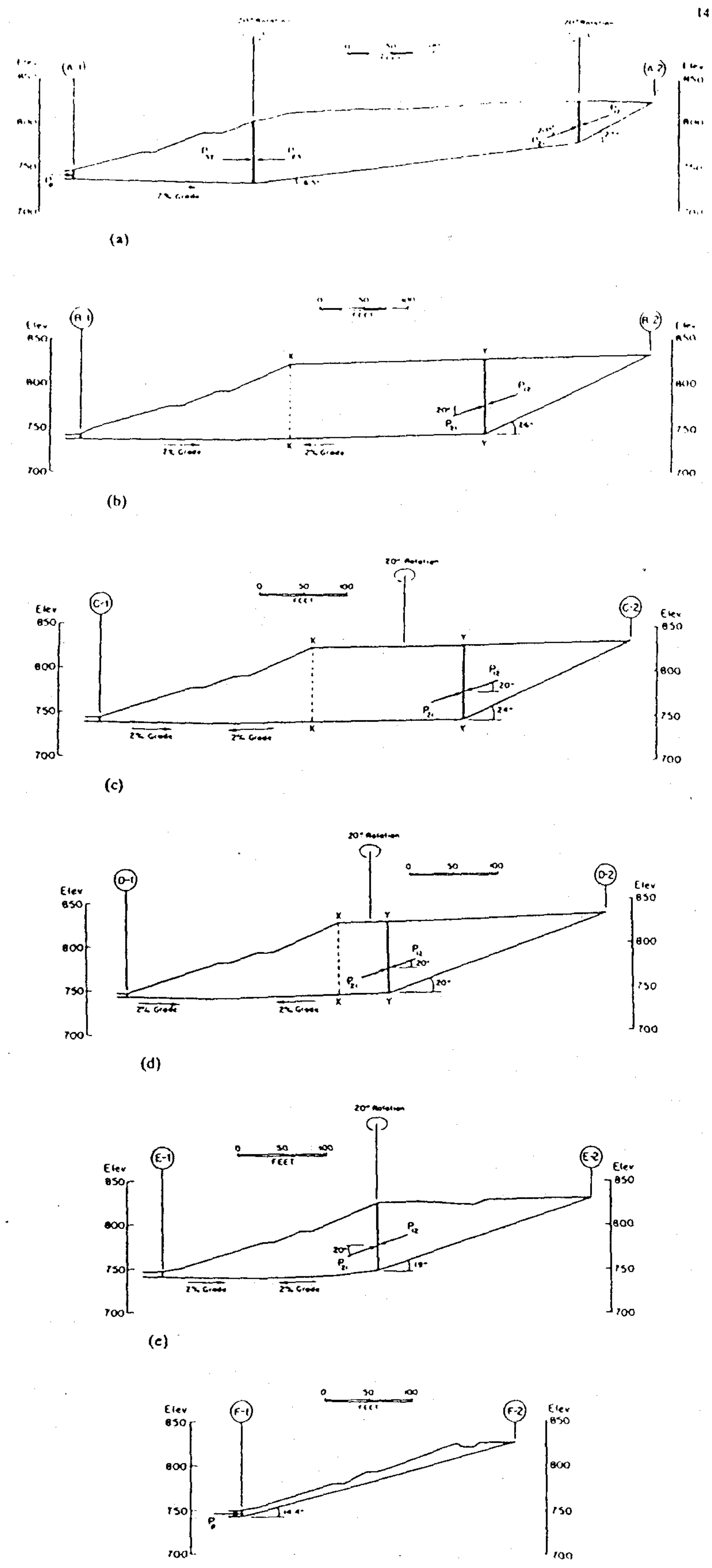

FIG. 7. Kettleman Hills Unit B-19, Phase 1-A Landfill: (*a*) Cross Section A-1/A-2; (*b*) Cross Section B-1/B-2; (*c*) Cross Section C-1/C-2; (*d*) Cross Section D-1/D-2; (*e*) Cross Section E-1/E-2; (*f*) Cross Section F-1/F-2

sections (Sections A-1/A-2 and E-1/E-2) have significantly smaller passive resisting blocks at the toe of the driving wedges. Section F-1/F-2, as shown in Figs. 5 and 7(*f*), has essentially no passive block.

The analyses were based on conventional force-equilibrium methods. The use of a vertical boundary between the active wedges and passive blocks was based on observations of the actual field failure conditions; the observed surface cracks (shown clearly in Fig. 2) formed almost directly above the toes of the active driving masses and thus represent near-vertical shear planes at this contact between the active wedges and passive blocks. In all analyses, the inclination of the resultant force between the active, or driving, wedges and the resisting passive block was assumed to be inclined at 20° to the horizontal, as shown in Figs. 7(*a*)–(*f*). This assumption is justified on the basis that the assumption of horizontal side forces gives safety factors that may be up to 15% too low and that the failure topography shows clearly that the active block dropped relative to the resisting block. Assumption of an obliquity of the resultant of 20° to the normal is not unreasonable for such a case.

For the Probable Minimum Clay/Liner Wetting Case, in which "wetting" of the HDPE liner/compacted clay liner interface was assumed to occur only near the leachate sump [as shown in Fig. 6(*a*)], the shear resistance within the multilayer liner system was taken as $\phi_r = 8.5°$ on the sloping sides of the lined fill basin, and as $\phi_r = 8°$ at the nearly level base of the basin, except in the "wetted" zone near the leachate sump and north of the upper access road on the southeast face of the waste fill. In this localized area, the liner-interface-shear resistance was taken as $\tau_r = 900$ psf.

Fig. 8 shows the results of these stability analyses for six cross sections under Probable Minimum Clay/Liner Wetting Case conditions. As shown in this figure, Sections A-1/A-2, B-1/B-2, and C-1/C-2, which together represent more than half of the overall waste-fill mass, all have calculated factors of safety of F.S. = 1.33 to 1.36. Conditions become somewhat less stable toward the southwest side of the landfill, and Section D-1/D-2 has a calculated factor of safety of F.S. = 1.07. It is only at the extreme southwest side of the landfill that factors of safety of less than 1.0 are calculated; the computed factor of safety for Sections E-1/E-2 and F-1/F-2, are F.S. = 0.85 and 0.81, respectively. As the factors of safety for Sections E-1/E-2 and F-1/F-2 are only slightly less than 1.0, and as these two sections represent only a very small portion of the overall waste-fill mass, it appears unlikely that these two sections could trigger a progressive slope failure beginning at the southwest end of the landfill. This is particularly so in view of the fact that analyses of all six cross sections were based on residual strengths, so that consideration of the effects of strain-softening along the failure surfaces, necessary for progressive failures of this type, is already included in the computed factors of safety. By weighting each plane-section factor of safety shown in Fig. 8 in proportion to the mass of the fill near the plane that might be considered "tributary" to the plane section, the factor of safety of the overall fill mass can then be estimated to be on the order of F.S. ≈ 1.15 to 1.25.

The results of similar analyses for six planar sections, including two of the sections previously discussed, Sections A-1/A-2 and B-1/B-2, are shown in Fig. 9. The overall factor of safety is about the same as that determined for the nonplanar sections. It would appear from these results for the Prob-

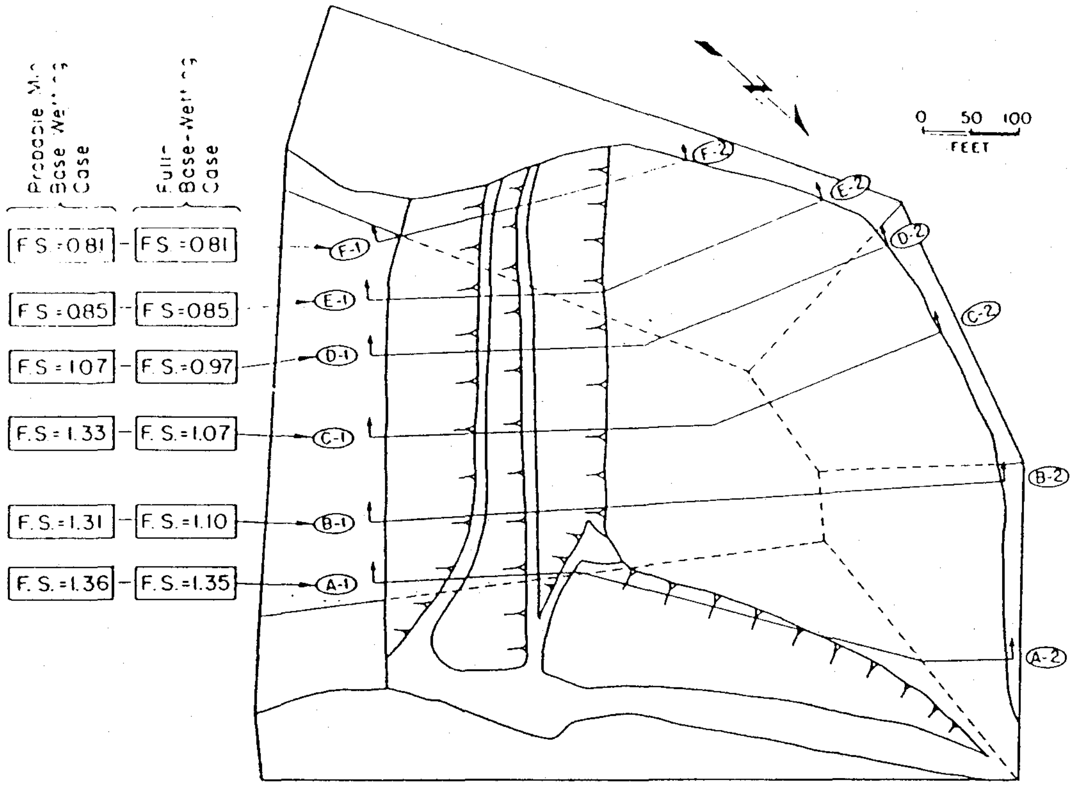

FIG. 8. Results of Two-Dimensional Slope-Stability Analyses of Sections A-1/A-2 through F-1/F-2 for Probable Minimum Base Wetting Case Conditions and Full-Base-Wetting Case Conditions

able Minimum Clay/Liner Wetting Case conditions, based on the use of interface-shear strengths developed by the laboratory investigations performed as part of these studies and the analysis approach described above, that the slope is stable.

Full-Base-Wetting Case

For the Full-Base-Wetting Case, shear failure was assumed to occur on a "wetted" HDPE liner/compacted clay liner interface with $\tau_r \approx 900$ psf over the full central base of the Unit B-19, Phase I-A landfill, as shown in Fig. 6(*b*). The shear resistance within the liner system over the remainder of the nearly level base of the fill basin was taken as $\sigma_N \tan \phi_r$ where $\phi_r = 8°$. The shear resistance within the liner system on the sloping sides of the lined fill basin was again taken as $\phi_r = 8.5°$.

Fig. 8 also shows the results of stability analyses for Full-Base-Wetting Case conditions for the same six cross sections considered previously. The calculated factors of safety for Sections A-1/A-2, E-1/E-2, and F-1/F-2 are essentially unchanged from those calculated previously for the Probable Minimum Clay/Liner Wetting Case conditions, because they are negligibly affected by the assumed larger zone of "wetted" HDPE liner/compacted clay liner failure surface. The three centrally located cross sections, which traverse the enlarged zone of "wetted" HDPE liner/compacted clay liner interface-shear failure assumed for this Full-Base-Wetting Case, show reductions in their calculated factors of safety. The resulting factors of safety for Sections B-1/B-2, C-1/C-2, and D-1/D-2 are F.S. = 1.10, 1.07, and 0.97, respectively.

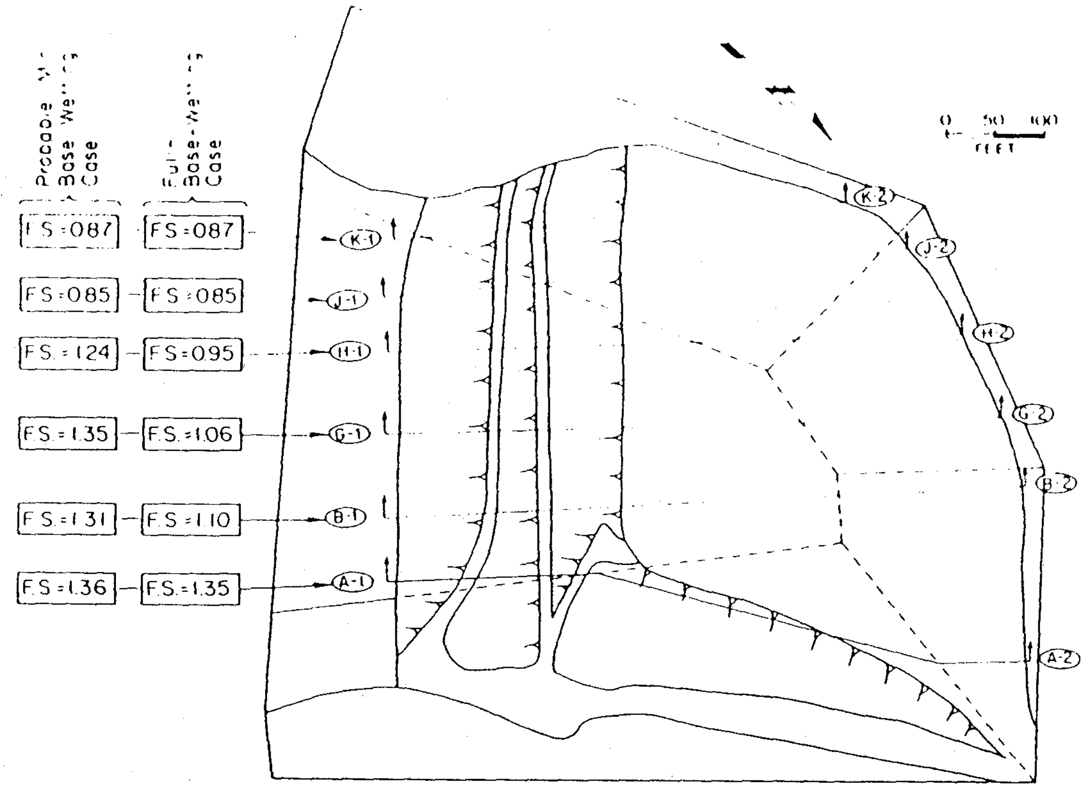

FIG. 9. Results of Two-Dimensional Slope-Stability Analyses of Sections A-1/A-2, B-1/B-2, and G-1/G-2 through K-1/K-2 for Probable Minimum Base Wetting Case Conditions and Full-Base-Wetting Case Conditions

As shown in Fig. 8, for the six cross sections considered, the calculated factors of safety generally decrease from northeast to southwest across the fill for Full-Base-Wetting Case conditions. The calculated factors of safety are greater than 1.0 for the four sections (Sections A-1/A-2 through D-1/D-2) representing the northeast and central portions of the waste fill, though for the centrally located Sections B-1/B-2 through D-1/D-2, the calculated factors of safety are significantly less than for the Probable Minimum Clay/Liner Wetting Case.

As the factors of safety for Sections E-1/E-2 and F-1/F-2 are only slightly less than 1.0, and as these two sections represent only a very small portion of the overall waste-fill mass, it again appears unlikely that these two sections could trigger a progressive failure, particularly because (1) These analyses are once again based on residual liner-interface-shear strengths; and (2) the more stable cross sections A-1/A-2, B-1/B-2 and C-1/C-2 represent significantly more than half of the overall waste-fill mass. Although these two-dimensional analyses of six representative cross sections for Full-Base-Wetting Case conditions result in the calculation of somewhat lower factors of safety than did the analyses described for the Probable Minimum Base Wetting Case conditions, they do not appear to represent a clear condition of overall slope instability. Based on the calculated factors of safety for the six cross sections shown in Fig. 8, the factor of safety of the overall fill mass might be estimated to be on the order of F.S. ≈ 1.1. This factor of safety, though low, is based on residual interface-shear strength for "worst case" possible wetting conditions, and is not clearly indicative of a level of slope instability that could result in the observed slope displacements of up to 35 ft that occurred.

Similar results are shown in Fig. 9 for six planar cross sections through the fill. Once again, the results of the analyses of the planar sections are very similar to the results of the analyses of the nonplanar sections.

In addition to the computed factors of safety, other aspects of the results of the two-dimensional stability analyses are as follows:

1. Near the southwest side of the landfill, computed factors of safety are significantly less than those near the northeast side, suggesting that failure may have been initiated along the southwest side and propagated by a progressive tearing or dragging effect to the northeast side of the fill; these movements could also have led to some tendency for horizontal rotation of the slide mass, reducing lateral pressure within the fill near the northeast side.
2. The factors of safety for nonplanar sections (in plan) near the southwest side are in some cases lower than those for plane sections through the fill parallel to the main direction of sliding, suggesting that movements may not have occurred in a single direction within the slide mass.
3. The fanning out of the zones of the driving mass behind the passive resisting zone, together with the clearly weak condition at the southwest boundary suggests that three-dimensional analytical studies may provide a better indication of the actual factor of safety against sliding in this case.

Three-Dimensional Stability Analyses

The fact that the side slopes of 1 on 2, or 26.6°, on the southwest and northwest sides of the basin and 1 on 3, or 18.4°, on the northeast side are considerably greater than the critical underlying liner-interface-friction angle of 8.5° is significant. It means that fill on the sides must rely on the resistance provided along the base for support. Any component of this downslope force that acts in the direction of potential sliding of the mass on the base will contribute to instability. Similarly, it can be envisioned that "squeezing" forces applied by the fill masses overlying the inclined sides of the lined fill basin might, through Poisson-type effects, act to promote instability at the central toe of the waste-fill mass. Such conditions could lead to a situation wherein the three-dimensional factor of safety is less than that computed for the two-dimensional case, a situation that has not been reported heretofore for geotechnical stability problems. No generally applicable methods for the three-dimensional analysis of the stability of systems such as that at Kettleman Hills have been developed and verified. Two approaches were used in the present study to investigate potential three-dimensional effects.

Multiple Block Analysis—Force-Equilibrium Approach

A convenient three-dimensional-analysis approach for the type of configuration of slip surfaces and fill geometry involved in the Kettleman Hills Phase I-A waste-fill facility is to divide the overall fill into a series of five blocks as shown in Fig. 10, to consider the equilibrium of each block and the boundary stresses developed between blocks, and finally to resolve all forces in the anticipated directed of sliding to evaluate the potential for sliding of the entire system. In this method of analysis, forces can be balanced for each individual block and for the overall system to establish the factor

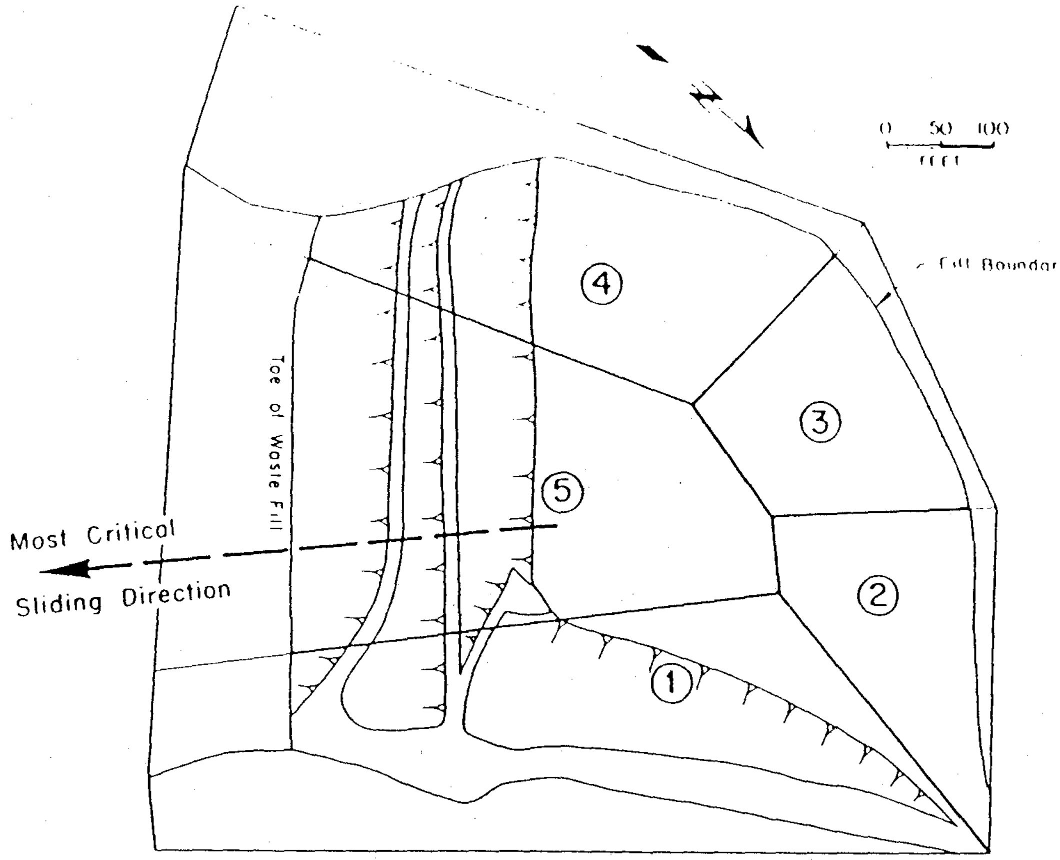

FIG. 10. Plan View of Landfill Showing Five Block Masses Used for Three-Dimensional Force-Equilibrium Stability Analyses, and Resulting Critical Sliding Direction

of safety against failure of the complete system of blocks acting as a unit.

Analyses were made by this approach, using the system of five blocks as shown in Fig. 10 and the same properties as those discussed previously for the two-dimensional analyses. Boundaries between blocks were again assumed to be vertical. Lateral forces acting on these boundaries were assumed to be horizontal, and no lateral shear forces were applied at the vertical interblock boundaries, only normal forces. These assumptions were made to render the analysis determinate.

The analysis was performed by first selecting an assumed direction of lateral translation of the overall system (all five "blocks" were assumed to translate in the same lateral direction). A "trial" factor of safety (FS) was then assumed, and all shear forces on the bases of the five blocks were then taken as

$$T_{base} = (N_{base})(\tan \phi_m) + \frac{(c_{base})(A_{base})}{FS} \quad (2)$$

where T = total base shear force mobilized; N_{base} = total base normal force; $\tan \phi_m = (\tan \phi_{base})/FS$, where $\phi_{base} = \phi_r$ for the critical liner interface; $c_{base} = c_r$ for the critical liner interface; A_{base} = total base surface area; and FS = factor of safety. As the vertical interblock-boundary forces had no vertical component, the system could then be analyzed by considering vertical equilibrium for each of the five individual "blocks" or masses, and overall lateral equilibrium of the total five-block system in the assumed direction of sliding.

This required iteration of the "trial" factor of safety to achieve convergence. The ("converged") factor of safety was determined by this procedure for a number of different assumed directions of sliding in order to find the most critical sliding direction and the associated most critical (lowest) factor of safety for both the Probable Minimum Base Wetting Case conditions and the Full-Base-Wetting Case conditions. The resulting computed factors of safety were for the Probable Minimum Base Wetting Case conditions, $FS = 1.14$; and for the Full-Base-Wetting Case conditions, $FS = 1.06$. These values are somewhat lower than the overall factors of safety estimated from the results of the two-dimensional stability analyses. It is interesting to note that these force-equilibrium analyses show the most critical potential sliding direction (the assumed direction of sliding that results in the lowest calculated overall factor of safety) to be in close agreement with the actual observed direction of sliding, as shown in Figs. 4 and 10.

However, while this method of analysis offers the advantage of being analytically "correct" and statically determinate from a force-equilibrium point of view and is, therefore, in principle readily reproducible, it is not clear that this analysis, which requires the entire fill to undergo the same lateral translation (while behaving as essentially rigid block masses), meets the requirements of kinematic compatibility associated with the actual mode of failure, which probably involved some out-of-plane movements and some degree of progressive failure. For this reason, an alternative approach was also considered.

Multiple Block Analyses Allowing for Differential Movements of Slide Mass

In an attempt to obtain some estimate of the possible magnitude of other kinematic conditions on the computed stability of the waste fill, analyses were made for multiblock systems giving some consideration to possible nonuniform directions of potential sliding of the blocks. All of these types of analyses involve some degree of judgment and they do not necessarily result in the full satisfaction of overall force-equilibrium conditions. However, since this situation is considered acceptable to some degree in some widely used methods of two-dimensional slope-stability analyses, exploratory analyses were made to determine the type of results that might be obtained using such an approach for the Kettleman Hills landslide. The type of effects involved in such an approach are illustrated schematically in Fig. 11(*a*). As illustrated in this figure, it might be visualized that driving forces promoting overall slope instability were generated by the "active" waste-fill masses overlying the sloping liner faces on the north and northwest sides of the landfill; however, additional "active" driving forces might also be generated along the east and west sides of the fill mass. All of these "active" driving forces could then be considered to have converged on the central waste-fill mass overlying the nearly level base of the fill basin, where they were resisted by "passive" resisting forces due to shear strength mobilized at the base of this central waste-fill mass.

For a three-dimensional analysis of this type, the Phase I-A landfill was subdivided by vertical planes into 11 "blocks" or masses, as shown in Fig. 11(*b*). Also shown on this figure are dashed lines representing the six cross sections analyzed previously using two-dimensional force-equilibrium meth-

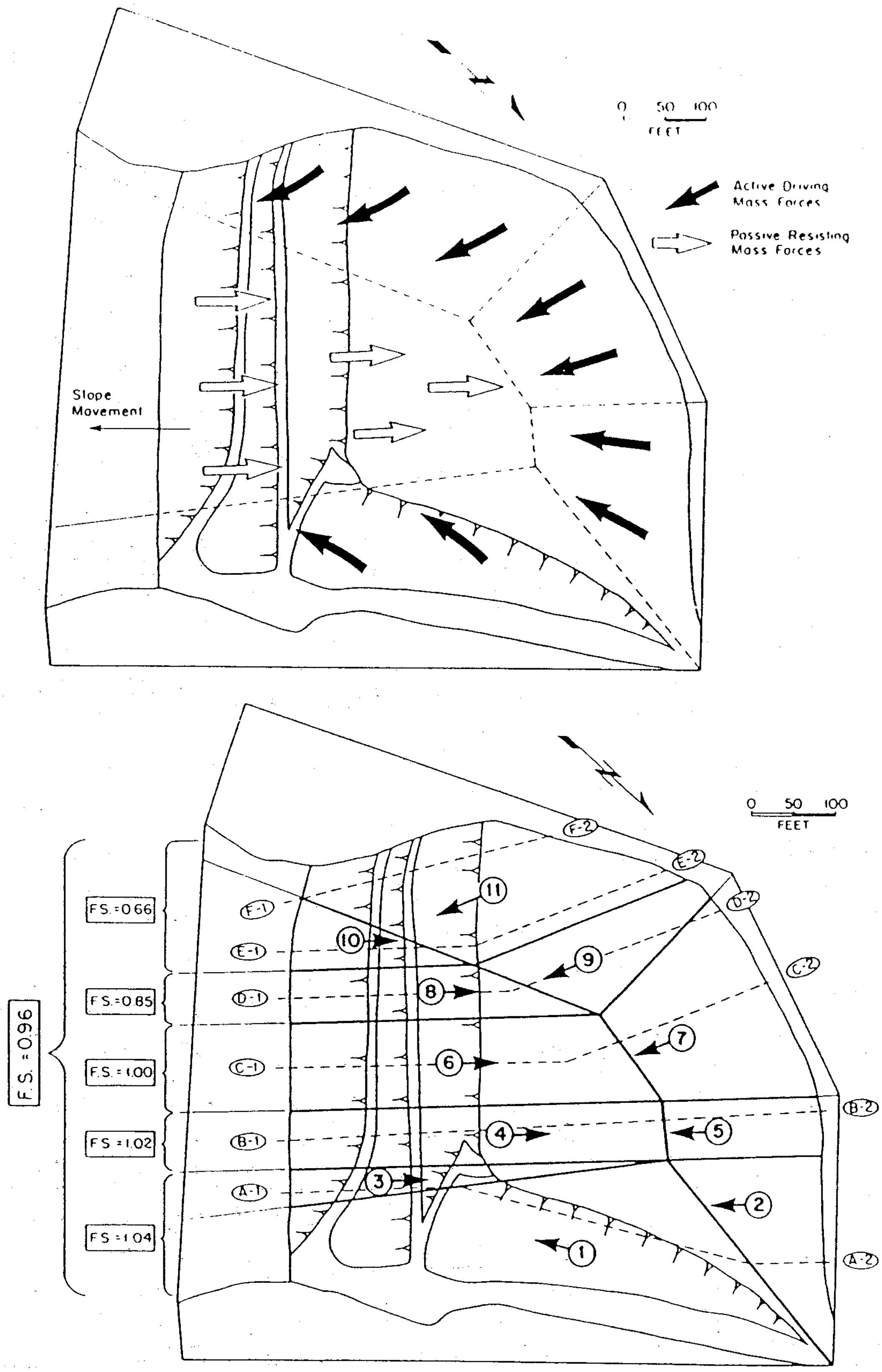

FIG. 11. (*a*) Schematic Illustration of Three-Dimensional Failure Mechanism; and (*b*) Results of Multiblock Analysis

ods. The orientation or "sense" of the active driving-mass forces and of the passive resisting-mass forces were, in most places, considered to closely parallel to the orientations of the active and passive forces acting on the six nonplanar cross sections analyzed previously (as shown in Figs. 5 and 8). Interblock contact forces acting on the vertical boundaries between active driving masses and passive resisting masses were again considered to be inclined at 20° to the horizontal, as in the two-dimensional analyses described previously. As relatively small shear strengths (less than τ = 100 psf) were necessary to transmit side shear forces across these interblock boundaries, it was not necessary to make an accurate assessment of the shear strength of the waste fill.

Fig. 11(*b*) shows the results of such an analysis for the Phase I-A landfill under Probable Minimum Clay/Liner Wetting Case conditions. As shown in this figure, the estimated overall factor of safety for the whole waste fill mass is F.S. = 0.96, which is about 20% lower than the factor of safety estimated on the basis of the two-dimensional (2D) analyses of six representative cross sections using the same liner-interface-shear-strength parameters, and about 15% lower than the factor of safety computed by the three-dimensional force-equilibrium analyses. However, the analysis results do not perfectly satisfy overall translational equilibrium requirements. To perfectly satisfy translational equilibrium, the sum of the force vectors acting on the basal sliding planes underlying each of the 11 fill blocks should be a single force vector, perfectly vertical and equal in magnitude to the overall weight of the fill mass. The actual summed base forces produce a "resultant" force vector that is not quite vertical: the vertical resultant is approximately 2% less than the total fill weight, and there are horizontal resultant components equal to approximately 3.7% of the total fill weight in the direction of sliding and 1.5% in the direction orthogonal to the direction of sliding. While these unbalanced forces may appear small, the analytical results are clearly sensitive to the magnitude of unbalanced forces considered to be acceptable in any given case. Nevertheless the approach does lead to a varying factor of safety across the width of the fill mass, with the lowest stability occurring at the southwest side of the fill mass, which seems to be in better accord with the observed mode of failure than the development of a single factor of safety for the system as a whole.

The significance of these effects in engineering analysis is clearly a matter of engineering judgment. The situation is further complicated by the fact that the assumption of horizontal interblock side forces in the five-block "rigid block" force-equilibrium analyses described earlier is likely to have resulted in a somewhat lower factor of safety than would have been calculated based on assumed inclined-side forces. Unfortunately, as the inclination of these side forces makes the problem indeterminate, it cannot be definitively determined to what extent the horizontal-side-force assumption affects the factor of safety calculated. After consideration of these issues and the need to satisfy the "statics" of the problem and at the same time to consider more flexibility than is provided by three-dimensional force-equilibrium analyses of essentially rigid block masses, it was the judgment of the writers that a reasonable allowance for effects not considered in the rigorous force-equilibrium approach might be about 5%, and that the results determined by the (five block) force-equilibrium approach could appropriately be reduced by this amount for engineering-evaluation purposes. This leads to factors of safety for the two base-wetting conditions as follows: Probable Minimum Base Wetting Case condition, $FS \approx 1.08$; and Full-Base-Wetting Case condition, $FS \approx 1.01$. Clearly, other engineers may arrive at other values in making judgments of this type, but this aspect of the problem does not appear to be a critical matter in the present case.

Summary of Stability-Analysis Results

Two types of stability analyses were performed as part of this investigation: conventional two-dimensional stability analyses of nine representative cross sections and three-dimensional analyses of the overall waste-fill and liner configuration. Each type of analysis was applied to two cases: First, the Probable Minimum Clay/Liner Wetting Case, in which shear failure along a "wetted" HDPE liner/compacted clay liner interface was considered to occur only over a small area in the vicinity of the leachate sump [as shown in Fig. 6(*a*)]; and second, the Full-Base-Wetting Case, in which the HDPE liner/compacted clay liner was assumed to have become "wetted" over the full central base of the fill basin [as shown in Fig. 6(*b*)]. It is not presently known over what area "wetting" of the HDPE liner/compacted clay liner interface actually occurred, but these two cases might be considered to represent a reasonable range of likely effects of wetting on this interface. The results of these stability analyses are summarized in Table 3.

Liner-interface-shear-strength parameters used for all analyses were based on laboratory investigations described in the companion paper by Mitchell et al. (1989). For the Probable Minimum Clay/Liner Wetting Case, the shear resistance within the multilayer liner system was taken as $\phi_r = 8.5°$ on the sloping sides of the lined fill basin, and as $\phi_r = 8°$ over the nearly level base of the basin except in the "wetted" zone near the leachate sump and north of the upper access road on the southeast face of the waste fill [as shown in Fig. 6(*a*)]. In this localized area, the liner-interface-shear resistance was taken as $\tau_r = 900$ psf. For the Full-Base-Wetting Case, shear failure was assumed to occur on a "wetted" HDPE liner/compacted clay liner interface with $\tau_r \approx 900$ psf over the central base of the Unit B-19, Phase I-A landfill, as shown in Fig. 6(*b*). The shear resistance within the liner system over the remainder of the nearly level base of the fill basin was taken as $\phi_r = 8°$. The shear resistance within the liner system on the sloping sides of the lined fill basin was again taken as $\phi_r = 8.5°$.

As shown in Table 3, the results of these stability analyses for conditions

TABLE 3. Summary of the Results of Stability Analyses of the Unit B-19, Phase I-A Landfill

	Factor of Safety		
Base liner conditions (1)	(2D analyses) (2)	(3D analyses) (3)	Overall best estimate (4)
Probable minimum clay/liner wetting case	1.2 to 1.25[a]	1.08	≈0.95 to 1.25
Full-base-wetting-case	1.1 to 1.15[a]	1.01	≈0.85 to 1.15

[a]Estimated.

TJFA 442
PAGE 020

representing a reasonable range of HDPE liner/clay liner "wetting" conditions differ by only about 10%. This is true for both the 2D and 3D stability analyses. Thus, it may be concluded that although wetting of the HDPE liner/compacted clay liner interface may have contributed in some minor way to the observed slope failure of March 19, 1988, it is probable that a slope failure of this type would have occurred at about this same stage of fill placement regardless of the actual extent of wetting of this clay/liner interface.

As shown in Table 3, the factors of safety based on 3D stability analyses are approximately 10–15% lower than those based on 2D analyses of representative cross sections, indicating the apparent significance of three-dimensional effects in this case.

While the results of the analyses indicate a factor of safety in the range of about 1.01 to 1.08, it is important to note that there are a number of areas of uncertainty in the computed values, the primary ones being:

- The fact that the angles of friction of the liner-system components used in the analysis have a possible error of ±10%.
- The fact that three-dimensional analyses involve some degree of engineering judgment and probably have a level of uncertainty of about ±10%.
- The fact that the shear resistance developed at the HDPE liner/compacted clay interface in the liner system may have been different from that used in the analyses by ±25%.

Other minor areas of uncertainty with regard to the geometry of the landfill/liner system also exist. In the writers' judgment, consideration of these areas of uncertainty leads to the conclusion that engineering estimates of the factor of safety of the landfill/liner system at the time of failure are of the order of F.S. $\approx$ 0.85 to 1.25, as indicated in Table 3.

Conclusion

The use of liner-interface-shear strengths measured in the laboratory-test program (Mitchell et al. 1990) in conjunction with three-dimensional stability-analysis methods can provide results in good agreement with the failure that occurred at the Kettleman Hills facility. This evaluation of the cause of the slide movements suggests the following approaches for the design of other facilities:

- The variations in measured interface-shear-strength parameters for various liner-system interfaces indicate the desirability of performing similar test programs for proposed new facilities to establish design parameters until such time as more data and experience are available.
- Special consideration in the design process is needed in dealing with low strength liner systems to consider possible three-dimensional effects which may have a significant impact on the overall system stability during the placement of waste fill.

Appendix I. Reference

Mitchell, J. K., Seed, R. B., and Seed, H. B. (1990). "Kettleman Hills waste landfill slope failure. I: Liner system properties." *J. Geotech. Engrg.*, ASCE, 116(4), 647–668.