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SYNOPSIS The sliding stability failure of the Kettleman Hills waste landfill focused attention on several issues related to the safe design and filling of waste repositories, including low strengths between geosynthetic material interfaces in composite liner systems and low interface strength between compacted clay and smooth geomembranes. Waste placement plans must be carefully developed to insure an adequate factor of safety against sliding at all stages of filling.

Because of assumptions and uncertainties that remained following the initial failure investigation, model tests, at a scale of 1:150, were done. These tests reproduced the field failure very well and provided insights into the failure mechanisms. A three-dimensional method for stability analysis gave results in close agreement with field observations and the results of a subsequent detailed failure investigation done by others (Byrne et al., 1992). Those special cases of landfill geometry and liner properties for which the 3D stability may be more critical than that computed using usual 2D methods of analysis could then be determined.

INTRODUCTION

The stability failure of the Kettleman Hills Unit B-19, Phase I-A hazardous waste landfill on March 19, 1988, has been extensively studied and reported; e.g., Seed et al. (1988), Mitchell et al. (1990a, 1990b), Seed et al. (1990), Byrne et al. (1992). The cause and mechanism of this failure are now well understood, and this precedent-setting case has focused attention on several important issues relating to the safe design and filling of waste landfills. It has also provided us with an opportunity to evaluate some alternative means for determination of failure mechanisms and for the analysis of stability. In particular, both model tests and analytical studies were made by Chang (1992) independently of the work previously reported.

In this paper we first summarize the earlier studies and the important lessons learned. Then the model tests and analytical investigations are summarized, the results are compared with the known behavior, and the roles of such tests and analyses in geotechnical research and practice are illustrated.

THE WASTE LANDFILL FAILURE

Landfill Unit B-19 covers an area of about 120,000 m^2 and forms part of a waste treatment and storage facility at Kettleman Hills, California. It is a large, straight-sided but oval-shaped bowl with a base that is nearly level. The liner in the northern end of the bowl, which covers an area of about 50,000 m^2, was completed first and is designated Phase I-A. A schematic illustration of the lined basin and the surface topography of the waste just four days prior to the failure are shown in Fig. 1 (a).

Placement of solid waste and soil cover began in early 1987 and proceeded at essentially a constant rate until March 19, 1988, when a stability failure occurred, resulting in lateral displacement of the fill of up to 11 m (35 ft) towards the southeast and surface settlements of up to 4.3 m (14 ft) along the back of the sliding mass. Surface cracking of the fill was clearly visible, as were also tears and displacements of the exposed portions of the liner system. The failure occurred over a period of a few hours, after which no subsequent movements were measured. There had been no earthquakes, rain, or other events that could have triggered the slide. Surface contours of the waste after failure are shown in Fig. 1 (b).

INITIAL FAILURE INVESTIGATIONS

From field observations, photographic and survey records, and preliminary stability analyses, it seemed clear that the failure had developed by sliding along interfaces within the multilayer liner system, within the compacted clay layers that formed parts of the composite double liner system, or along combinations of liner interfaces and through the clay. The slide appeared to have been initiated simply because the waste pile reached a height

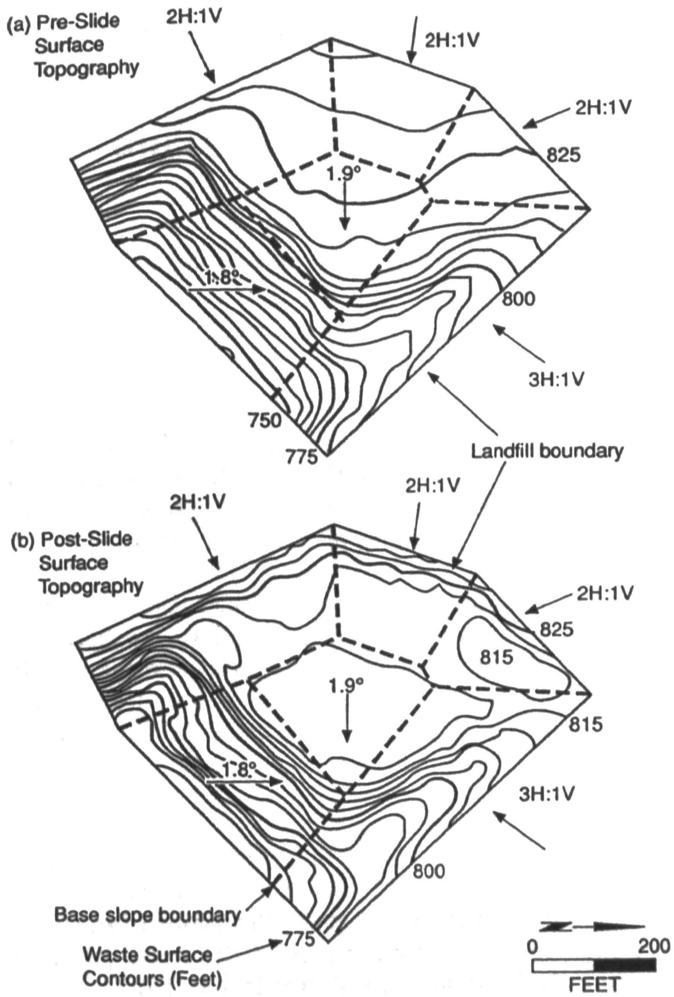


Fig. 1. Schematic Illustration of Lined Repository Basin and Waste Fill Surface Topography of Landfill Unit B-19, Phase I-A. (From Byrne et al., 1992)

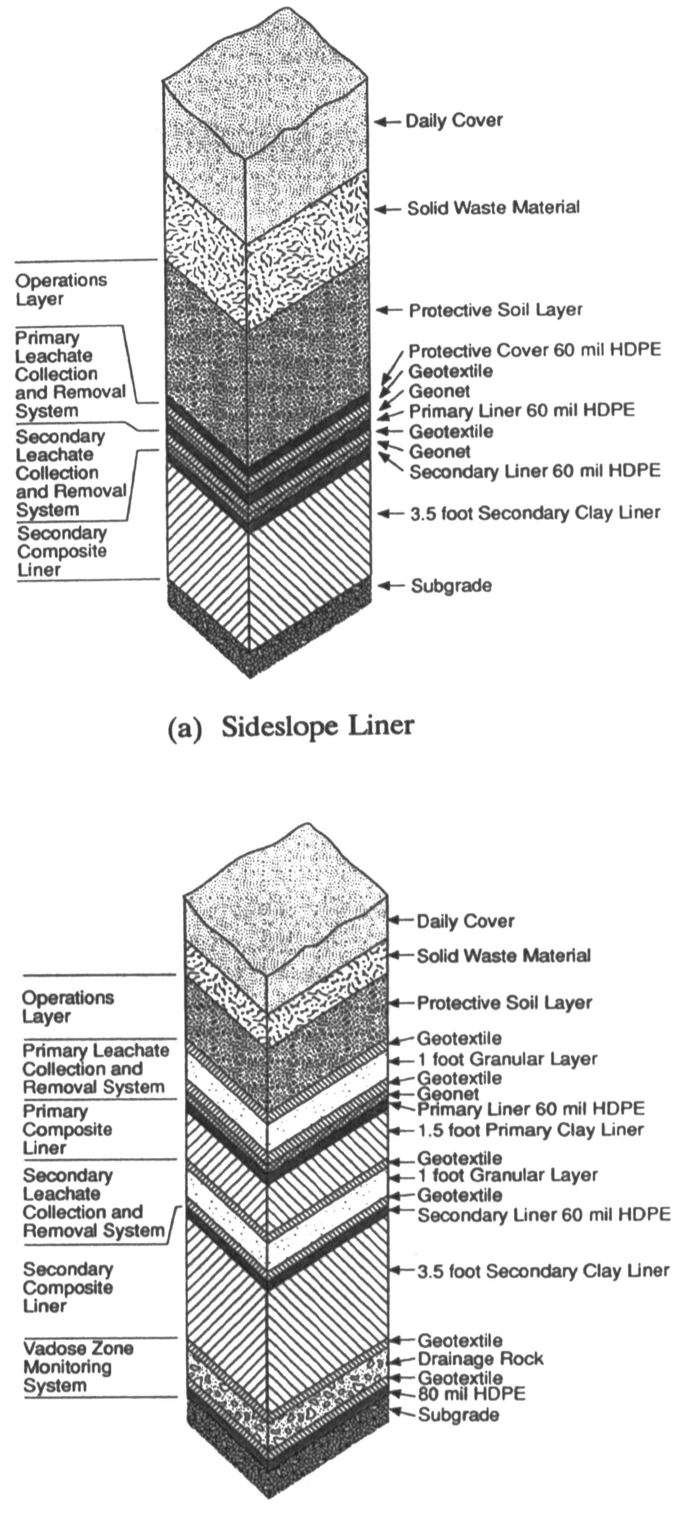


Fig. 2. Schematic Diagrams of the Kettleman Hills Landfill Double Liner System. (From Byrne et al., 1992)

that was excessive for the landfill geometry and liner system interface strengths, and the subsequent stability analyses (Seed et al., 1990) supported this hypothesis.

Schematic illustrations of the sideslope and base liner system cross sections are shown in Figs. 2(a) and 2(b), respectively. Details of the liner configurations and properties of the liner system materials are given by Mitchell et al. (1990) and Byrne et al. (1992). Direct shear and pullout tests were used to evaluate geosynthetic interface strengths, and direct shear tests were used to determine the strength along the HDPE geomembrane-compacted clay interface for as-compacted conditions and after soaking under light surcharge. The results for the most critical interfaces are given in Table 1; the detailed results for all interfaces are summarized in Mitchell et al (1990a). Residual strengths were used for stability analyses, because the peak strengths were reached at very small displacements which were very likely to have occurred during liner construction or waste fill placement.

Table 1. Friction Angles or Shear Strengths on Critical Interfaces in the Kettleman Hills Landfill Liner System from Initial Failure Investigation

Interface	Residual Friction Angle (ϕ_r) or Residual Undrained Shear Strength (τ_r) Along Saturated Base	Residual Friction Angle (ϕ_r) Along Dry Slopes
HDPE Liner/Geotextile	8° ± 1°	9° ± 1°
HDPE Liner/Geonet	8.5 ± 1°	8.5 ± 1°
HDPE Liner/Saturated Clay	45 ± 12 kPa	N/A

Two-dimensional factors of safety for the mass shown in Fig. 1(a) were made assuming plane cross sections of the type shown in Fig. 3, giving the results shown in Fig. 4. A difficulty with two-dimensional stability analyses for problems such as this is that there is uncertainty as to which cross section is representative of the overall mass. By weighting the plane section factors of safety in Fig.4 in proportion to the mass of fill tributary to the plane section, overall factors of safety were computed as about 1.15 to 1.25 if the wetted area of the repository at the time of failure was assumed to be the minimum possible, and 1.10 to 1.15 for the case where the wetted area was assumed to be the maximum possible (Seed et al., 1990). These factors of safety, while low, did not suggest sufficient instability that there would be a displacement of more than 10 m as a result of failure.

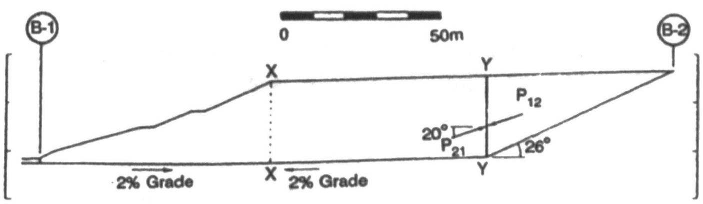


Fig. 3. Cross Section B-1/B-2 at the Time of the Failure of the Kettleman Hills Waste Landfill

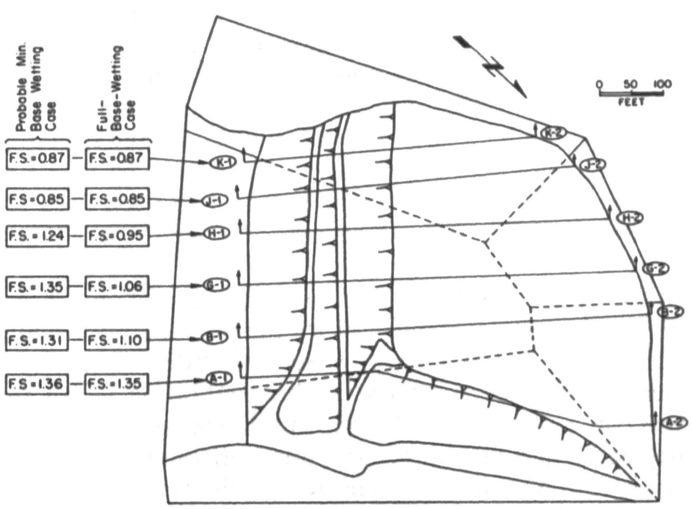


Fig. 4. Two-Dimensional Factors of Safety at the Time of Failure of the Kettleman Hills Landfill Failure

Accordingly, it was considered that three dimensional effects, might have been important. The physical reasoning for this can be explained by reference to Fig. 5, which shows a four-block simulation of a sliding mass. The driving forces for sliding are caused by the active force from block 1 plus the components of the active forces from blocks 2 that act in the sliding direction. The latter forces may be significant for a system in which the interface friction angle is low compared to the slope angle. Their magnitudes, for a given set of properties, depend on the side slope divergence angle θ_s and dip angle θ_{d1}. Whether or not the factor of safety is larger or smaller for divergent side slopes ($\theta_s > 0°$), as was the case for Unit B-19, Phase I-A, than for parallel side slopes ($\theta_s = 0°$) depends on whether the added driving forces from blocks 2 are greater that the added sliding resistance provided by the increased size and surface area of the passive block (block 3) as the divergence angle increases. Specific conditions for which the three-dimensional condition is more critical than the two-dimensional are indicated in a later section of this paper.

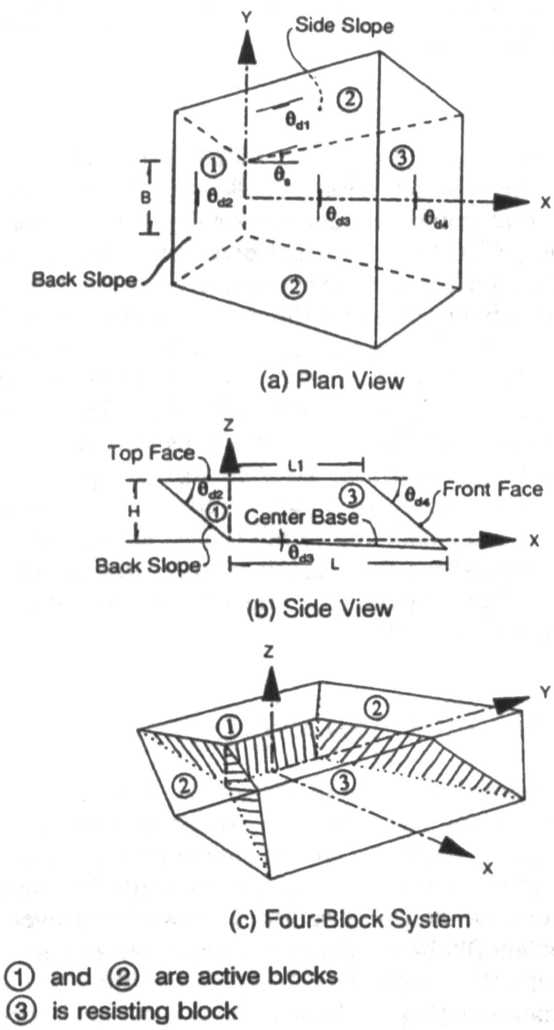


Fig. 5. Four Block Schematic of Lined Waste Repository for Analysis of Three-Dimensional Effects

No generally applicable three-dimensional stability analysis methods for systems such as the Kettleman Hills Landfill had been developed and validated at the time of the failure. Accordingly, two approximate approaches were used, as described by Seed et al. (1990). The results of these analyses are compared with those for the two-dimensional analyses in Table 2. Also indicated in Table 2 are overall best estimates of factor of safety that take into account uncertainties in interface shear strengths and the methods of analysis that were used.

Table 2. Summary of the Results of Stability Analyses of the Kettleman Hills Unit B-19, Phase I-A Landfill at Failure

Base Wetting Conditions	Factor of Safety		
	2-D Analyses	3-D Analyses	Overall Best Estimate*
Wetting only in the vicinity of the leachate collection sump	1.2 to 1.25 (est.)	1.08	0.95 - 1.25
Full saturation of clay along repository base	1.1 to 1.15 (est.)	1.01	0.85 - 1.15

* Authors' estimates taking into account uncertainties in liner system friction angles of ± 10%, uncertainty in the 3-D analysis methods of ± 15%, and uncertainty in the HDPE/compacted clay interface shear resistance of ± 25%.

It was concluded that these initial investigations, done during the first few months following the failure, gave results that could account reasonably for the observed behavior. Nonetheless, it had been necessary to assume that the interface properties measured in the laboratory corresponded to those for the materials and conditions in the field, that failure was indeed along one or more of the interfaces listed in Table 1, that a correct assumption has been made for the unit weight of the landfill waste, and that the stability analysis methods were accurate for the conditions analyzed. To shed further light on these issues, a separate study was initiated to develop an improved understanding of the failure mechanism and a better method for the analysis of the three-dimensional stability. The nature and results of this study are summarized subsequently.

ACTUAL CAUSE AND MECHANISM OF FAILURE

After removal of waste in Phase I-A, Unit B-19, it was possible, during late 1990, to examine and test the liner system along which the landfill sliding had occurred. The results of these investigations are presented in detail by Byrne et al. (1992). In brief, they showed that over most of the landfill base sliding occurred on the interface between the secondary clay and the secondary geomembrane (Fig. 2). Some movements also occurred above this interface owing to kinematic constraints at limited locations; e.g., in the vicinity of the intersections between the base and the 2:1 side slopes on the northwest and southwest. There was also sliding between the primary geomembrane and secondary geotextile in the upper parts of the southwest and northwest side slopes. The displacement vectors for the different sliding waste blocks deduced from both surface surveys immediately following the failure and striations in the uncovered HDPE geomembrane liner are shown in Fig. 6.

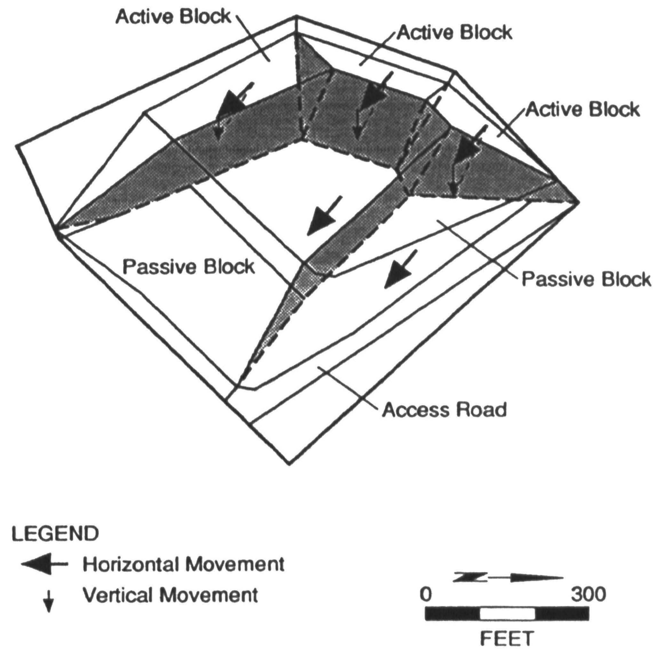


Fig. 6. Displacement Vectors of the Waste Blocks During Failure. (From Byrne et al., 1992)

A detailed program of sampling and testing enabled Byrne et al. (1992) to establish peak and residual interface strengths for geosynthetic-geosynthetic interfaces and for the compacted clay-geomembrane interface. With respect to the latter, a strong dependency of the residual undrained strength on compaction water content and a smaller, but still significant dependency on the normal stress were noted. The geosynthetic-geosynthetic interface friction angle of 8° was comparable to that found by Mitchell et al. (1990a); whereas, the actual clay strengths for the in-situ secondary clay liner obtained by Byrne et al. (1992) were somewhat lower than had been used for the initial failure analyses reported by Seed et al. (1988).

Byrne et al. (1992) did both two-dimensional and three-dimensional stability analyses for the pre- and post-failure geometries of the landfill mass. The three-dimensional analysis was based on the Janbu method, in which the vertical component of the interblock side forces is neglected and overall vertical and horizontal equilibrium conditions are satisfied. The results of their analyses for the pre-failure geometry, and assuming residual strengths, gave factors of safety of 0.85 for the three-dimensional ase and 0.81 for a selected two-dimensional section. For esidual strengths and the post-failure geometry the orresponding factors of safety were 1.08 and 1.02. As did eed et al. (1990), Byrne et al. (1992) concluded that the esidual strength values are the most useful values for ssessment of stability.

hus, the results of the cause of failure investigation eported by Byrne et al. (1992) resolved the uncertainties ι the preliminary failure investigations and confirmed ιat the failure can be explained by application of testing ιethods that are representative of the actual conditions ι the field and suitable stability analysis methods. Unfortunately, however, the three-dimensional nature of ιost waste repositories leads to uncertainties when two-imensional stability analysis are used, owing to the ifficulty in selection of a truly representative cross ection. In addition, three-dimensional analyses are onsiderably more difficult. Fortunately, however, those tuations where the three-dimensional stability may be ore critical than would be predicted using two-mensional analysis methods can often be readily etermined, and more is said about this later.

OME LESSONS LEARNED

he unexpected stability failure of the Kettleman Hills Vaste Landfill, Unit B-19, Phase I-A, on March 19, 1988, ocused attention on several important issues relating to ιe safe design and filling of such facilities:

1. The geosynthetic materials; e.g., geomembranes, geotextiles, geonets, that are used in landfill liner systems may have very low interface shear strengths; i.e., friction angle as low as 8 degrees.

2. The interface between a HDPE geomembrane and compacted clay immediately beneath it may have a very low shearing resistance. Unfortunately, those compaction conditions that favor low hydraulic conductivity - compaction to a high degree of saturation wet of optimum moisture content, an essential characteristic of compacted clay liners - also produce the lowest values of interface strength. This situation is shown clearly by Fig. 7, where Zone I represents both the region where compaction would most likely be specified to obtain a suitable hydraulic conductivity and the region where interface strength values are the lowest. It is important to note also, that for compacted clay layers of significant thickness, such as the secondary clay at Kettleman Hills, which had a thickness of 3 ft, consolidation and strength increase at the compacted clay-HDPE geomembrane interface will require a time that may be long compared to the rate of filling of the landfill. For example, the cause of failure investigation by Byrne et al. (1992) showed that the compacted clay-geomembrane interface remained essentially undrained during the one year of waste fill loading prior to the failure.

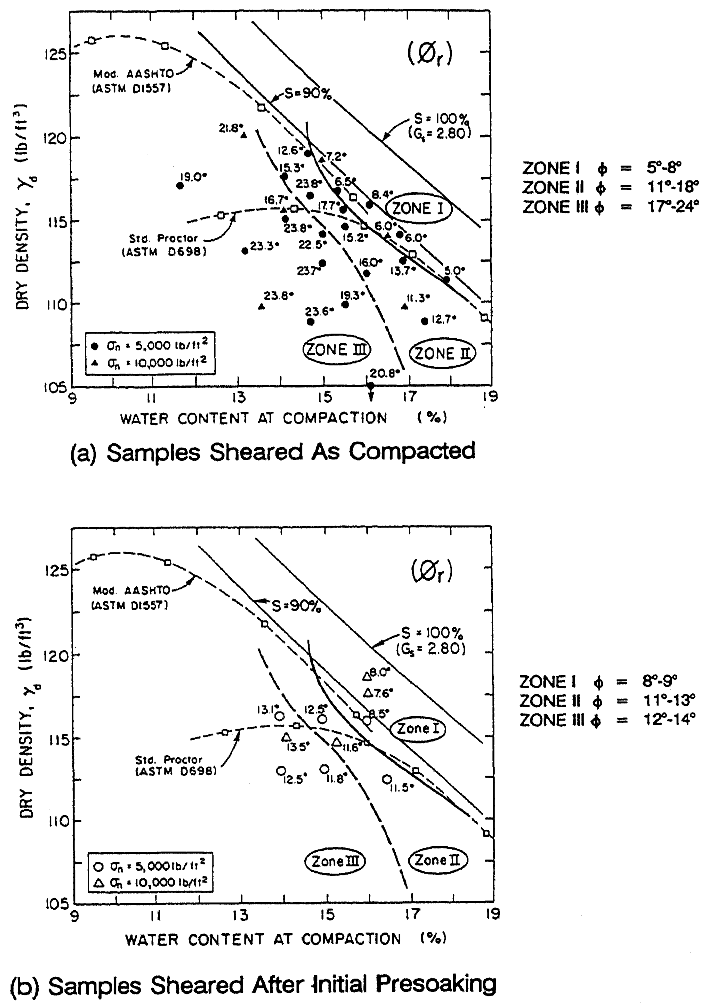


Fig. 7. Interface Shear Strengths for Smooth HDPE/Compacted Clayey Till (with 5% Bentonite) Interfaces as a Function of Compaction and Soaking Conditions. (From Seed and Boulanger, 1991)

3. The unique geometries of some waste repositories and the very low strengths in liner systems lead in some cases to situations in which the three-dimensional stability is more critical than would be predicted from the results of two-dimensional analyses. Fortunately, situations where this may be the case can often be identified without detailed analysis.

4. Testing programs should cover the full range of anticipated field conditions. As illustrated in Fig. 7, the interface shear strength can be significantly altered by minor changes in water content. For example, the figure shows that the interface shear strength may change by as much as a factor of two from a shift in as - compacted water content of as little as three percent. It is important also to model correctly the overburden stress, post-compaction wetting, and degree of consolidation.

5. Landfill filling plans; i.e., the sequences of fill stages, should be developed in such a way that an adequate factor of safety can be maintained at all times for all fill heights and geometries.

MODEL TESTS

At the completion of the preliminary failure investigations in the summer of 1988, there remained questions of the actual mode of failure, the true in-situ properties, and the suitability of different methods of stability analysis that were not to be answered definitively until late 1990 when the waste fill had been removed and the cause of failure investigation reported by Byrne et al. (1992) could be completed. Nonetheless, it was considered that useful information could be obtained concerning the general mechanisms of failure by means of model tests. Furthermore, if the results of these tests could be reasonably documented and quantified, then there would be a basis for development and evaluation of suitable stability analysis methods. Accordingly, such an investigation was undertaken at the University of California, Berkeley. This investigation and its results are described in detail by Chang (1992). In this paper we present only a brief overview of these studies, which will be described in more detail elsewhere, and indicate the most significant conclusions for use in geotechnical practice.

The model tests had three purposes:

1. To try to reproduce the in-situ failure conditions.

2. To develop an understanding of the actual failure mechanism for use as a basis for the development of a suitable three-dimensional method for stability analyses of waste landfills.

3. To obtain data for quantitative evaluation of three-dimensional stability analysis methods.

Model Design and Construction

A model:prototype scale of 1:150 was chosen. This resulted in a model with dimensions of 1.4 x 1.4 x 0.2 m (4.5 x 4.5 x 0.6 ft). The sliding mass had a weight of 5400 N (1200 lb) and a maximum overburden stress on the liner system of 4 kPa (80 psf). The low vertical stress on the liner interface and within the simulated waste fill required special low pressure tests for determination of relevant strength properties.

Plywood panels were used to fabricate a "waste repository basin" that conformed, to reduced scale, very closely to that in the actual Kettleman Hills facility. This basin was lined with smooth, 60 mil thick HDPE. Several plastic sheet materials were investigated, using specially designed direct shear tests and inclined plane sliding tests (Chang, 1992), to provide a suitable geosynthetic interface between the fill and the base HDPE. A 4 mil thick polyethylene sheet was chosen as representative of the desired interface conditions. The actual values of peak and residual interface friction angles were influenced by several factors, including interface cleanliness, characteristics of joints between adjacent sheets, polishing caused by prior sliding, time under normal stress prior to shear, and temperature. A summary listing of these factors and their effects on the measured friction angles is given in Table 3.

Table 3. Effects of Different Factors on the HDPE/PE Interface Friction. ϕ_p and ϕ_r are peak and residual friction angles, respectively.

Factor	$\Delta\phi_p$	$\Delta\phi_r$
HDPE Surface Cleaning		
C_0 - process	+2° ~ +7°	+2° ~ +6°
C_3 - process	0 ~ -1°	-1° ~ -2°
C_5 - process	+3° ~ +9°	+2° ~ +10°
Scotch Tape on HDPE Joints		
∥ dir. of sliding	+6° ~ +8°	+1° ~ +8°
⊥ dir. of sliding	+9° ~ +11°	+4° ~ +5°
Interface Polishing		
1-sided, (N/P)	0 ~ -2°	0 ~ -2°
2-sided, (P/P)	0 ~ -2.5°	0 ~ -3°
Time under Sustained Pressure		
Δt = 0.5 day	+4°	+1.5
1.0 day	+5.5°	+2°
1.5 days	+6°	+2°
2.0 days	+6°	+2°
Temperature		
T ∈ (5~15)°C or (41~59)°F	≈ +0.2°/°C	≈ +0.3°/°C
T ∈ (20~40)°C or (68~104)°F	≈ 0	≈ 0

Loose sand was used to simulate the waste fill. As failure in the actual landfill had occurred along the liner system, it was not necessary to duplicate the waste properties exactly. Ticino sand was used for model tests No. 1-3, and Monterey No. 0 sand was used for model tests No. 4-6. The sand was loosely placed by pouring. In some of the tests, thin layers of white diatomite were placed in critical zones so that cross-sections cut following failure would provide insight into the locations and patterns of slip planes. Surface markers were used to enable determination of failure displacement vectors. The shear resistances of sand/PE and sand/sand interfaces were determined, and the results are summarized in Table 4.

Table 4. Shear Resistances for Sand/PE and Sand/Sand Interfaces

Combination	ϕ_p	ϕ_r	$d_p \times 10^{-3}$ (in)	No. of Tests
mil PE / 60 mil HDPE	6.5°	5.6°	1 ~ 4	18
mil PE / 4 mil PE	10°	6.7°	2 ~ 3	2
lont. #0 Sand / 4 mil PE	13°	11°	5 ~ 10	3
lont. #0 Sand / Mont. #0 Sand	36° ~ 38°	36° ~ 38°	~ 140	3

Note: d_p = Shear displacement at peak strength

The models were built to an initially stable condition by ilting the rear of the model basin downwards slightly. Failure was then induced by lifting the rear of the model ısing a vibration-free hoist until sliding started. At this point further change in inclination was stopped, and observations of movement continued as a function of time ıntil further sliding ceased. Careful measurements were nade of the tilt angle at failure, displacement vectors at number of points, surface settlements, surface cracking, nd cross sections in the failure zone.

Model Test Results

The photographs in Figures 8 and 9 show pre- and post-ailure views of Model No. 4, respectively. Post-failure urface displacement vectors for Model No. 4 are shown ı Fig. 10. The horizontal displacement vectors in the ıodels agreed well with those for the actual landfill, as ıay be seen for Model No. 6 as shown in Fig. 11. There as also excellent agreement between the post-failure round surface profiles in the model tests and in the ctual landfill. Surface contours after failure in the models onformed well to those in the field, as shown, for xample, by Fig. 12 for Model No. 6. The surface xpression of internal cracking was also very much the me for the models and the actual landfill failures, as ıown in Fig. 13.

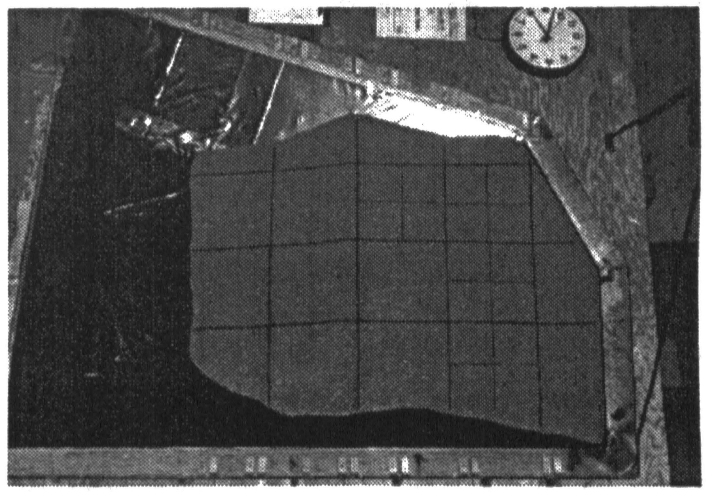


Fig. 8. Pre-failure View of Kettleman Hills Landfill Model No. 4.

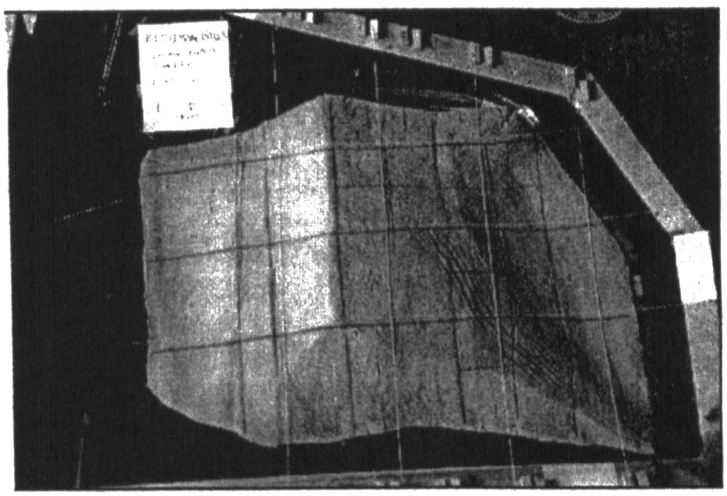


Fig. 9. Post-failure View of Kettleman Hills Landfill Model No. 4.

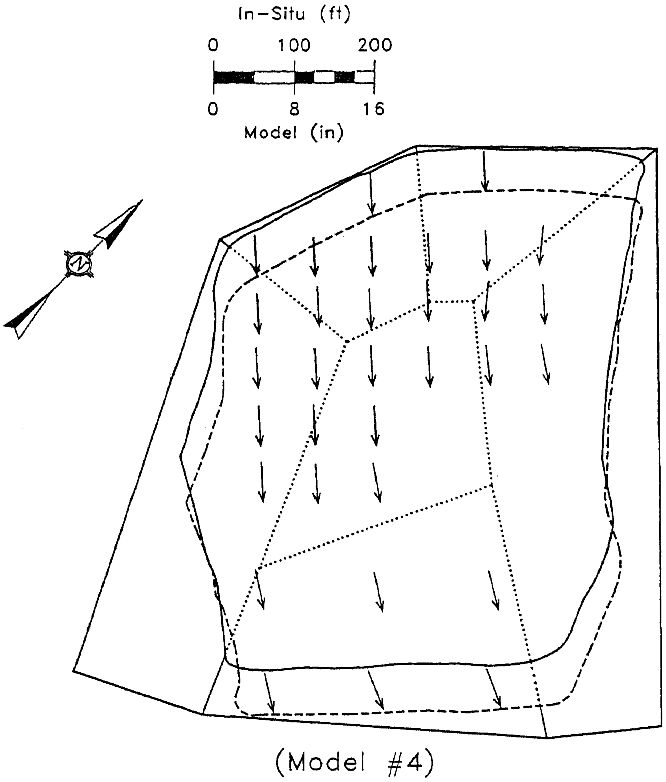


Fig. 10. Failure Displacement Vectors for Model Test No. 4.

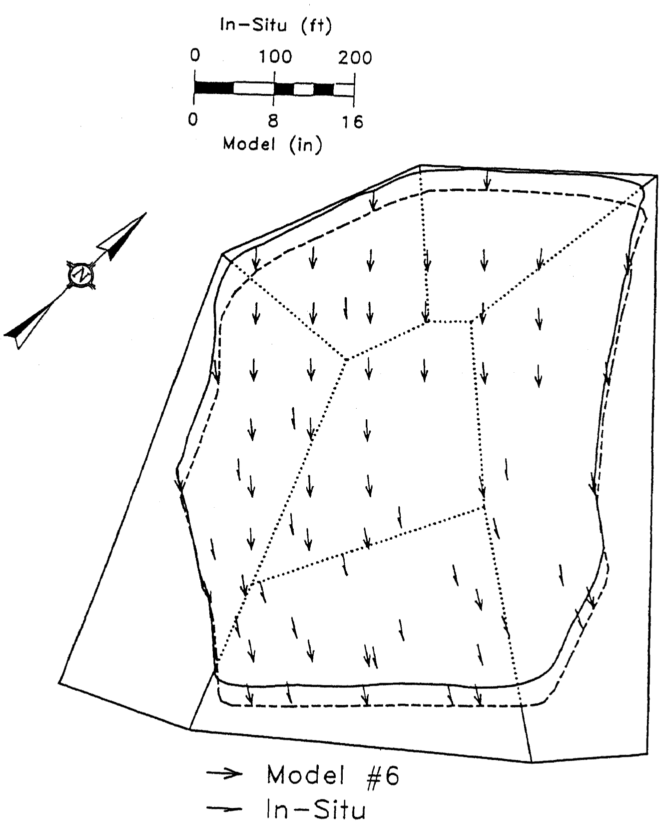


Fig. 11. Horizontal Displacement Vectors for Model No. 6 and the In-Situ Landfill Failure.

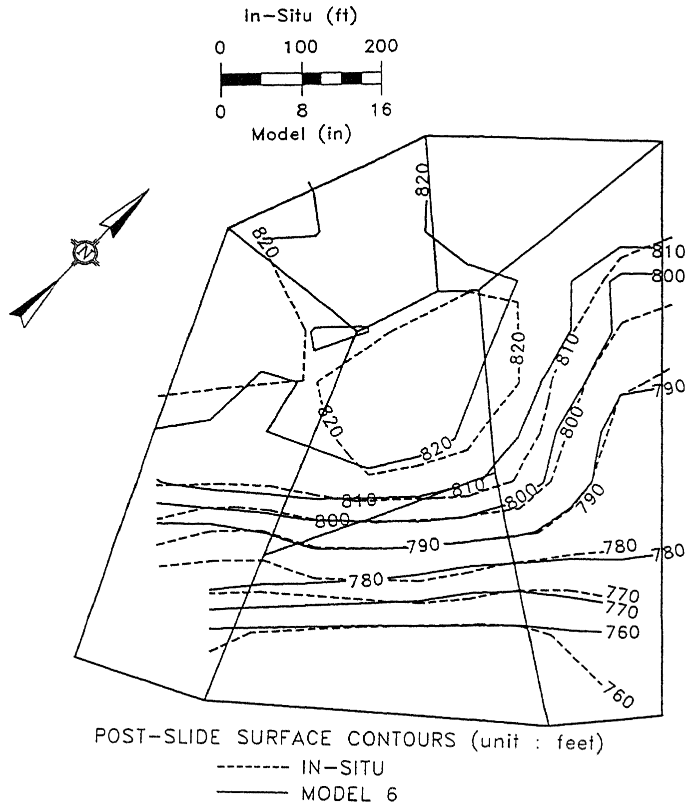


Fig. 12. Post-failure In-situ and Model Surface Contours for Model No. 6.

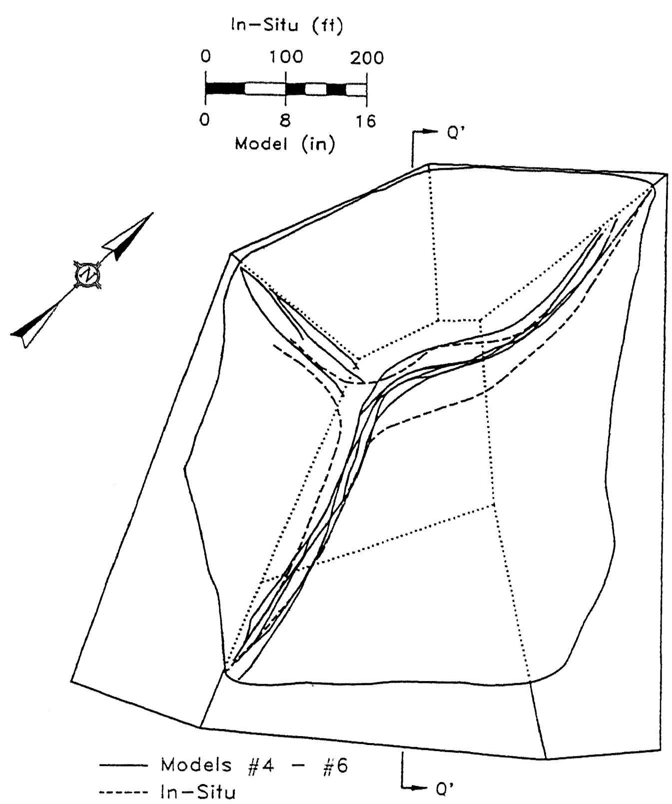


Fig. 13. Surface Cracking Patterns in Models and in Actual Landfill.

The results showed clearly that the landfill failure mass could be divided into two active blocks and one passive block, as indicated in Fig. 14. A series of interblock interfaces formed during displacement that tilted forward from the vertical at angle θ, as shown in detail by Chang (1992). The band defined by these shear interfaces had a width approximately equal to the total distance of sliding and was located as shown in Fig. 13.

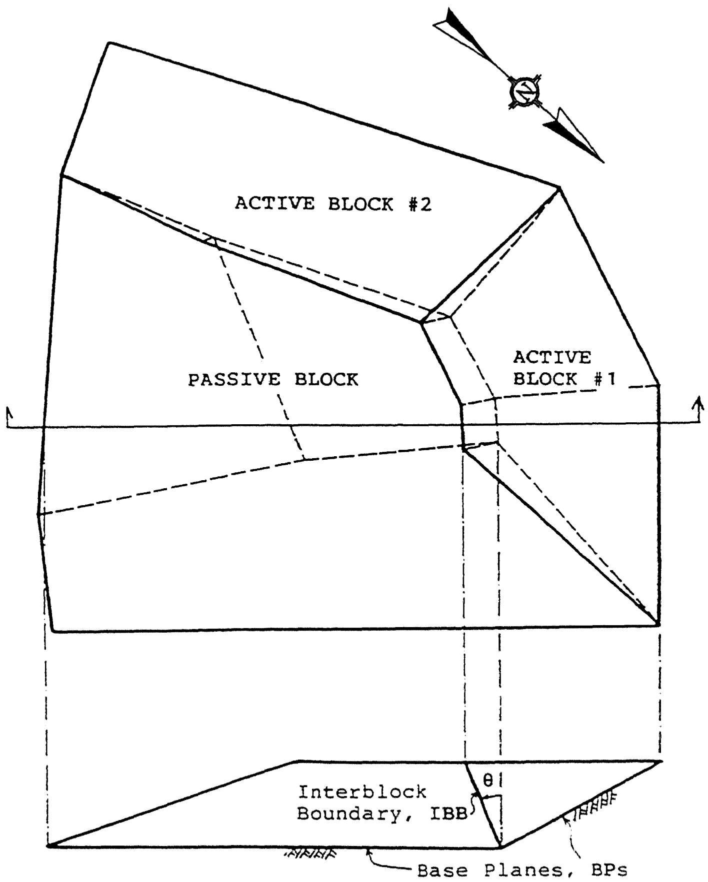


Fig. 14. Two Active and One Passive Failure Block Configuration of Landfill Mass

Not all models failed when the inclination of the model table was raised to exactly 0°; i.e., horizontal. The actual inclinations ranged from -1.9° to +5.0°, with Models 5 and 6 failing within a half a degree of horizontal. However, estimates could be made for the actual friction angle at failure using corrections for the influences of the different HDPE/PE interface conditions indicated in Table 3. When this was done, equivalent friction angles could be computed for each model that would have caused failure at 0°, and this gave equivalent peak friction angles in the range of 11.2 to 13.5 degrees and residual friction angles in the range of 6.9 to 7.6 degrees, with best estimates of 12.0 and 7.1 degrees, respectively. As discussed earlier, the residual friction angle is the appropriate one for stability analysis.

Overall, the results of the model tests established a pattern of failure that was consistent among models and with what was known at the time about the failure mode in the actual Kettleman Hills Landfill. A basis was available, therefore for development of a method of stability analysis that could be consistent with actual behavior.

STABILITY ANALYSES

Two-Dimensional Analyses of Models

Two-dimensional sliding block stability analyses of the models were made for two cases: (1) pre-failure geometry and (2) post-failure geometry. The effects of liner interface shear resistance and interblock boundary inclination were studied. Residual strengths were used to represent worst case conditions and because residual strength values are considered the most appropriate for representation of the actual conditions, as discussed earlier in this paper. Six representative cross sections, A-A′ to K-K′, located as shown in Fig. 15, were evaluated. These sections are similar to those studied by Seed et al. (1990). Spencer's method, which assumes the same side force inclination between all blocks and satisfies all conditions of equilibrium, was used. The analysis procedures and results are given in detail in Chang (1992).

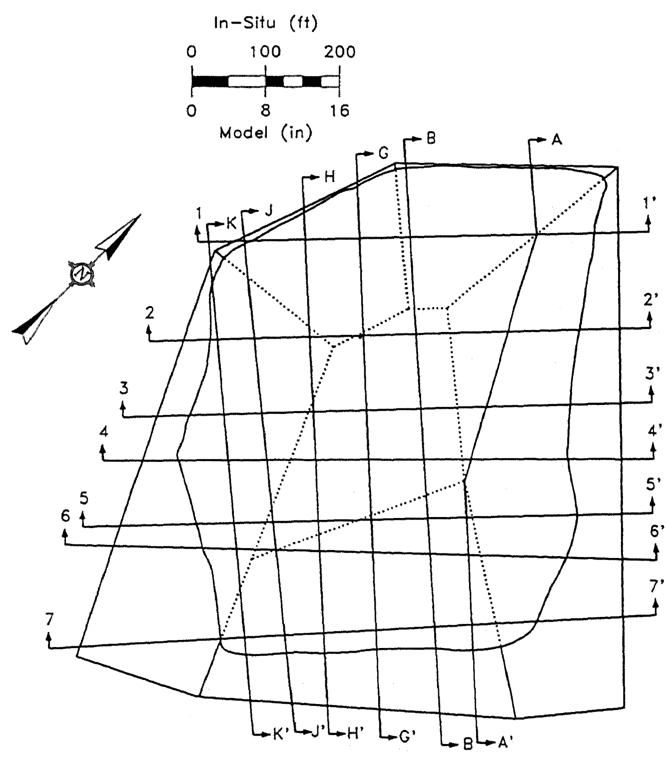


Fig. 15. Cross-Sections Used for Two-Dimensional Stability Analysis of Kettleman Hills Waste Landfill Models

The results for the pre-failure geometry showed that only the sections in the southwestern part of the model, which accounted for about 20 percent of the mass, gave factors of safety less than 1.0. The overall factors of safety for the fill mass, based on averaging of weighted factors of safety for each cross section, were in the range of 1.05 to 1.12, with an average of 1.08. The weighting was done by counting each section's value in proportion to the mass tributary to it. Side force inclinations for the different sections were in the range of 6 to 16 degrees.

For the post-failure geometry, the overall factors of safety should be very close to 1.0 if true values for the properties and a correct analysis method are used. In these analyses account was taken of the effect of the actual distance of sliding on the residual friction angle of the liner interface materials. The results were similar to those for the pre-failure geometry analyses. The post-failure factors of safety for the different models ranged from 1.10 to 1.18, with an average of 1.14. It was concluded therefore that the two-dimensional analyses, done using the best available stability analysis methods and the most reliable estimates of properties, gave unconservative results; i.e., stability was predicted when failure actually occurred. Furthermore, the factors of safety for maximum cross sections taken through the center of the fill mass, which might intuitively appear to be the most critical, gave overestimations of the factor of safety of 10 to 30 percent. Accordingly, it was necessary to develop a three-dimensional analysis method to provide a more correct representation of the actual stability conditions.

Three-Dimensional Stability Analysis Method

A new method for three-dimensional stability analysis of fill masses sliding on pre-determined failure surfaces was developed by Chang (1992) and will be reported in a separate paper. Only the basis of the method, assumptions, and some of the results are given here. The following assumptions are made:

1. The failure mass can be represented by a single layer of blocks that slide on a base. The blocks can distort at constant volume during sliding.

2. All interfaces between blocks and between the blocks and the base planes remain in contact during sliding.

3. Failure is by translation only. This is justified by the fact that the model tests showed that rotational movements were very small.

4. The principal stress directions in the fill ar vertical and horizontal.

5. The factor of safety is defined as the numbe by which the shear strength parameters c and φ must be divided to bring the system to a stat of limiting equilibrium.

6. Interface and waste fill deformation behavior i linear elastic-plastic.

7. The shear forces on base planes act parallel tc the block displacement vectors.

8. Interblock shear forces act parallel to the line of intersection of the interblock boundaries.

9. The normal forces on interblock boundaries are functions of the lateral earth pressure coefficient and the boundary orientation.

10. The interblock shear forces are deformation dependent and are proportional to the amount of shear stress mobilized.

11. Strength is governed by the Mohr-Coulomb shear strength criterion.

12. There is the same factor of safety on all base planes.

Computer code SSA-3D was developed to do the analyses (Chang, 1990).

The method was tested against four problems with known solutions. The results obtained using SSA-3D for analysis of simple sliding wedges agreed almost exactly with those for analytical solutions. The SSA-3D analysis of very long landfills with constant cross section, where two-dimensional conditions would apply, gave factors of safety that were within five percent of those obtained using Spencer's method for a range of cross sections and material properties. Values of factor of safety within five percent of those by Spencer's method were obtained for simple slopes with rotational failure along an assumed failure surface. Boundary stress distributions by the two methods were similar. For a three-dimensional rotational failure of a vertical cut in clay SSA-3D overestimated the factor of safety by up to 10 percent for large length to height ratios. It was concluded that overall SSA-3D is quite accurate for computation of factors of safety and directions of sliding for translational failure along predetermined failure surfaces.

Results of Three-Dimensional Analyses of Models and Actual Landfill

Both pre- and post-failure landfill geometries for a model and the actual Kettleman Hills landfill were analyzed using SSA-3D. Model No. 6 was studied, as it most closely simulated the actual field conditions. A six-block system, as shown in Figs. 16 and 17, was analyzed. The inclination of the interblock boundaries between blocks 2 and 5 and blocks 3 and 5 is defined by angle θ. Pre-failure factors of safety for Model No. 6 were determined for θ varied between -30° and +60°, with the results shown in Fig. 18. A comparison between the critical interblock boundary zone based on SSA-3D and the surface cracking patterns observed in models 4, 5, and 6 is shown in Fig. 19. Using the post-failure geometry (actual factor of safety = 1.0) and the friction angle for the Model No. 6 liner at the end of sliding a factor of safety of 1.016 was computed. Thus SSA-3D gave accurate prediction of the model stability.

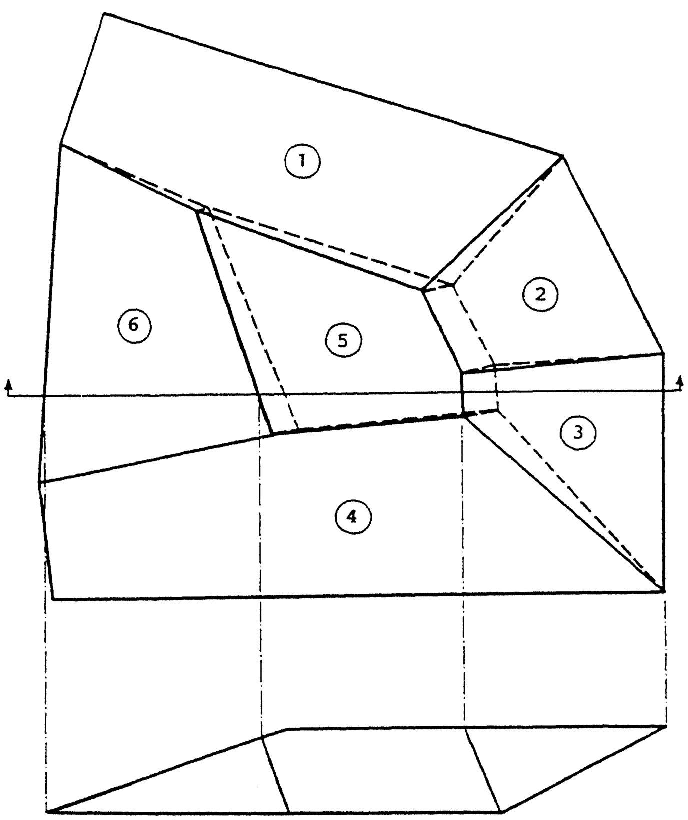


Fig. 16. Six Block Simulation of Unit B-19, Phase I-A Kettleman Hills Repository for Analysis by SSA-3D.

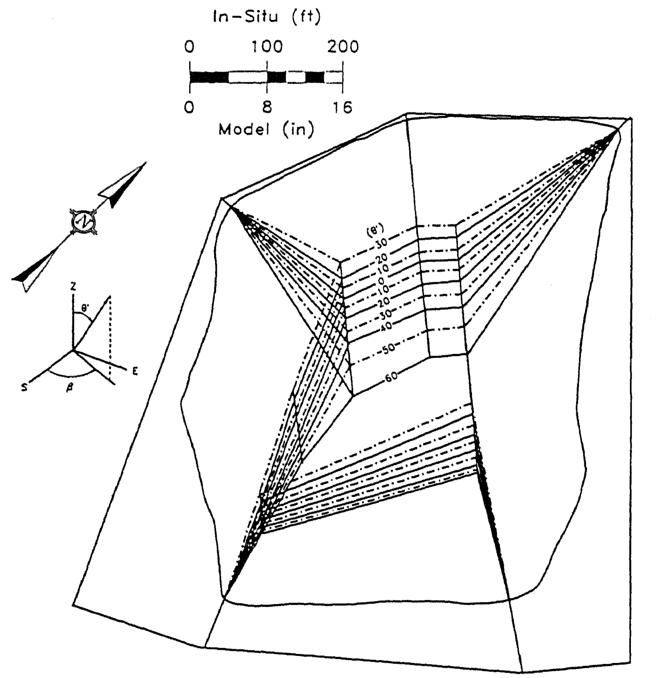


Fig. 17. Interblock Boundary Pattern for Analysis of Six-Block System.

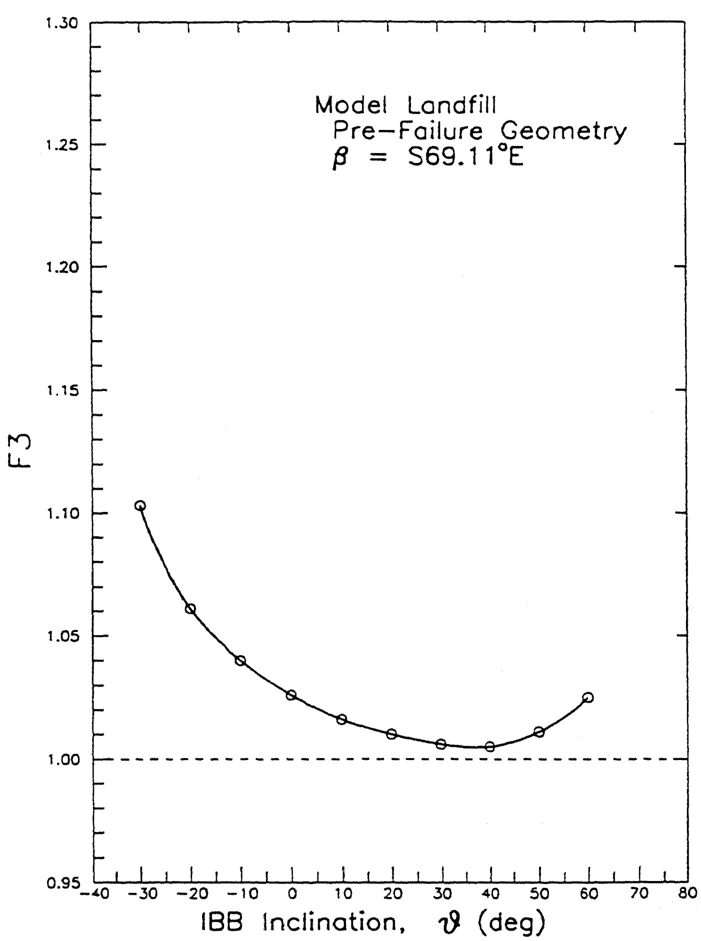


Fig. 18. Computed Factor of Safety for Model No. 6 as a Function of Interblock Boundary Inclination.

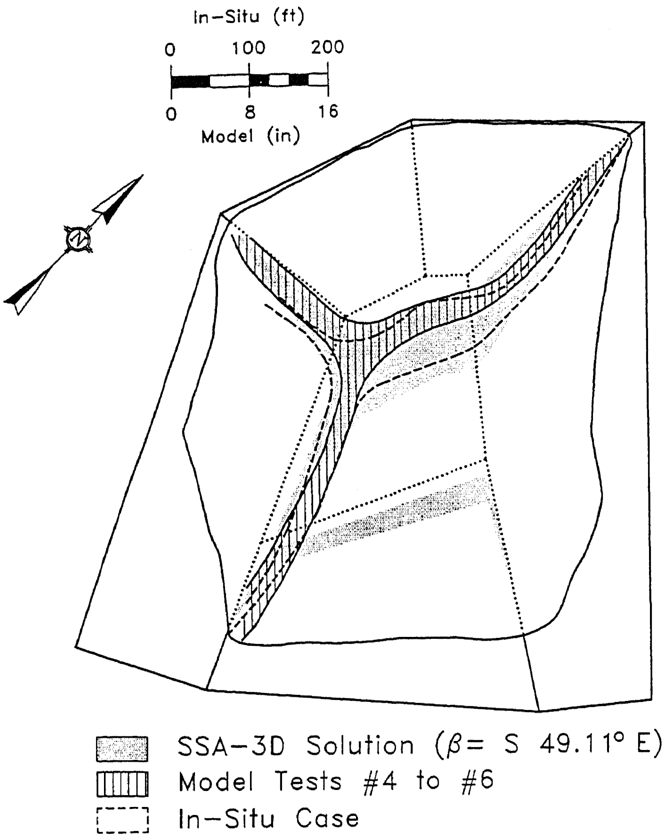


Fig. 19. Comparison Between Critical Interblock Boundary Region Based on SSA-3D and Observed Surface Cracking Pattern of Models 4-6. (Chang 9-9).

In the actual landfill the interface strengths for failure along geosynthetic interfaces were estimated by Mitchell et al. (1990) to be 8.5° for dry interfaces and 8.0° for wet interfaces. These values were confirmed during the subsequent failure investigation by tests on samples removed from the landfill structure (Byrne et al., 1992). In the initial investigations following the failure the HDPE/compacted clay liner interface strength *c* was estimated as 900 psf. Whether geosynthetic/geosynthetic interface friction or HDPE/compacted clay strength controls stability depends on the fill height above the liner. For H less than $H_{critical}$, $\gamma H \tan\phi_{liner}$ should be used, and for H greater than $H_{critical}$ the HDPE/compacted clay strength should be used, where $H_{critical}$ is defined by $c/(\gamma \tan\phi_{liner})$. The unit weight of the waste fill was about 120 pcf, and the actual strength of the landfill material is of little importance in the analysis.

SSA-3D analyses for the assumption of no base wetting and full base wetting conditions (Seed et al., 1990) for the pre-failure geometry gave factors of safety of 1.163 and 1.052, respectively, for the most critical inclination of interblock boundaries *θ*, which was about +40 degrees. Analyses of the post-failure geometry also gave values that were too high; i.e., greater than 1.0. However, the displacement vectors were correctly predicted, as may be seen in Fig. 20, which compares them for Model No. 6, in situ, and as predicted by SSA-3D.

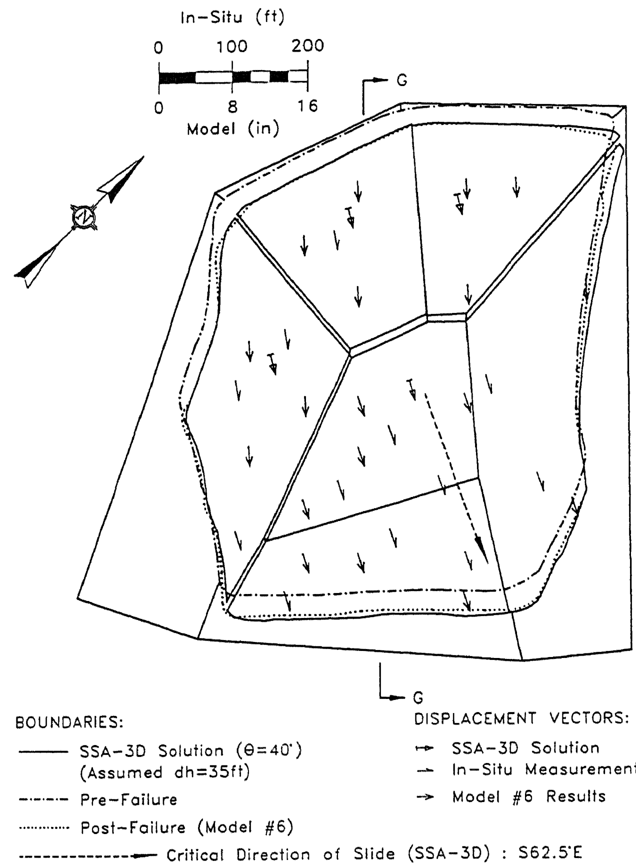
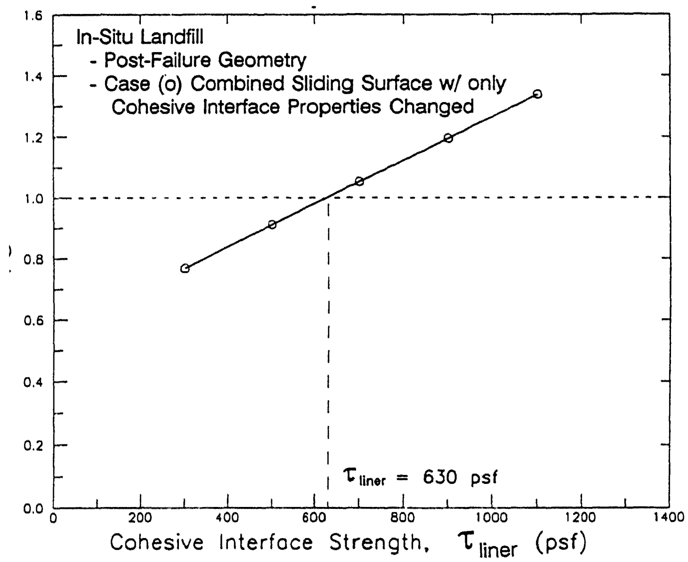


Fig. 20. Comparison of Displacement Vectors for SSA-3D Analysis, Model Test Failures, and Actual Landfill Failure.

Analyses were then done to evaluate the effect of variations in the HDPE/compacted clay interface strength on the factor of safety, and the results in Fig. 21 were obtained. They indicate that an interface strength of 630 psf would cause failure. The results of the failure investigation reported by Byrne et al. (1992) indicated that the actual strength of the secondary HDPE/compacted clay interface depended on both the water content and the normal stress. Over the ranges of these quantities appropriate for the in-situ conditions the strength variation was 570 to 700 psf, with an estimated overall average value of 640 ± 30 psf; i.e., about 30 percent less than had been assumed by Seed et al. (1990) and used in their analyses. Thus predictions of strength at failure using SSA-3D agree very closely with the updated best estimates of the actual field strength at failure.



'ig. 21. Three-Dimensional Factor of Safety of Kettleman Hills Waste Landfill as a Function of HDPE/Compacted Clay Liner Interface Strength.

'HE IMPORTANCE OF THREE-DIMENSIONAL :FFECTS IN LANDFILL STABILITY ASSESSMENT

'he Kettleman Hills waste landfill failure has focused ttention on whether the three dimensional analysis of .ope stability can give factors of safety less than those btained by two-dimensional analysis methods. After a areful review of studies of three-dimensional slope :ability done over the past 25 years, Duncan (1992) oncluded that "the factor of safety calculated using 3D nalyses will always be greater than, or equal to, the ictor of safety calculated using 2D analyses." 'onetheless, the results summarized in the previous :ction indicate that the factors of safety calculated using SA-3D were less than those calculated using Spencer's iethod for two-dimensional cross sections and that the :sults obtained using SSA-3D were consistent with the 1own properties and failure conditions.

he physical reasoning that would suggest that the ability can indeed be lower in three-dimensions than in vo dimensions was presented in connection with the scussion of Fig. 5. However, there are two major :oblems in extending this physical reasoning to a 1antitative comparison of two- and three-dimensional 1alysis methods for most cases; namely, (1) selection of two-dimensional cross section that is truly representative : the three-dimensional structure being analyzed, and (2) ie use of analysis methods that are based on exactly the ime assumptions concerning equilibrium conditions and aterial properties for both the 2D and 3D cases. Parametric studies by Chang (1992) using SSA-3D have been made which avoid these difficulties, and the major findings are summarized briefly here, with the detailed results to be presented elsewhere.

SSA-3D is adapted for two-dimensional plane strain analysis by assuming parallel and vertical side slopes; i.e., a divergence angle θ_s of 0, and a dip angle θ_{d1} of 90°, and an interface friction angle on the side slope interfaces of 0°. The results of the analyses showed that, for lined landfill type structures with low interface strengths, the three-dimensional factor of safety, for one set of typical values of landfill width, height, back slope angle, and front slope angle, could be lower for side slope divergence angles greater than about 10° and side slope dip angles between 5° and 80° for frictional liner interfaces. For cohesive liner interfaces, the 3D factor of safety was less than the 2D value for divergence angles greater than about 30° and dip angles between 15° and 70°.

The 3D factor of safety was up to 30 percent less than the 2D factor of safety for the most critical cases analyzed; i.e., side slope divergence angles greater than about 50°, side slope dip angles between 25° and 50°, and frictional liner interfaces. For cohesive slip surfaces, however, the 3D factor of safety was only up to about 10 percent less than the 2D value for the most critical cases. In practice, therefore, it would seem that the possibility for adverse three-dimensional effects can be determined in most cases by inspection. For regular geometries, such as that shown in Fig. 5, estimates of the potential magnitude of the reduction in 3D factor of safety compared to the 2D value can be made, and safe designs and filling plans can be developed. For more complex configurations, if the side slope divergence angles and dip angles suggest that the 3D configuration may be critical, then a 3D analysis may be required because of the difficulty in selection of a 2D cross section that is representative of the overall stability. Even in cases such as these, however, instability can be avoided by keeping unbalanced waste fill heights below specified levels.

SUMMARY AND CONCLUSIONS

The Kettleman Hills waste landfill failure of March 19, 1988, has focused attention on several issues related to the safe design and filling of waste repositories. Many of them have been reviewed in this paper. Initial investigations done during the first few months following the failure showed that sliding along the liner system could be explained by the waste fill reaching a height and mass that, for the given geometric conditions and probable in-situ properties, corresponded to a factor of safety of 1.0. However, because of the assumptions and uncertainties in the analyses, we undertook model tests that better defined the actual failure mechanisms, and analytical studies that yielded a new method for stability analysis. A failure investigation which involved complete removal of the waste from the landfill basin, observation of the direction of the sliding surface, and tests on the actual liner system materials was completed by early 1991 and reported by Byrne et al. (1992).

Among the most important lessons learned from the failure investigations are that (1) the interface strengths in liner systems may be very low, with friction angles as low as 8 degrees between layers of geosynthetic materials and strengths of only a few hundred psf along the interfaces between geomembranes and compacted clay; (2) compaction conditions that favor low hydraulic conductivity for clay liners are in the range of water contents and densities that give the lowest shearing strength at geomembrane/clay interfaces; (3) the unique geometries that may exist in some landfills can give conditions in which the stability in three dimensions can be somewhat lower than estimated using two-dimensional methods of analysis; (4) the testing programs for determination of liner interface strengths should allow for the full range of likely field conditions; and (5) the waste placement plans (sequence) for the filling of waste repositories should be developed with consideration of an adequate factor of safety against stability failure at all times.

Model tests of the Kettleman Hills landfill done at a scale of 1:150 gave failure patterns that conformed very closely to those that developed in-situ. The test results gave a more detailed understanding of the actual failure mechanism and provided a basis for development of a three-dimensional method for stability analysis. They further confirmed the usefulness of earth structure modeling both for development of understanding and for providing systems against which analysis methods may be tested.

A new three-dimensional method for stability analysis, SSA-3D, gave results in excellent agreement with the actual behavior, both in the models and in the actual landfill. This limit equilibrium method is applicable for translational failures along predetermined sliding surfaces. It has been shown with this analysis method that the factor of safety against sliding failure of a waste landfill mass can be lower for lined waste repositories having divergent side slopes than would be predicted using two-dimensional analysis methods. Although the 3D factor of safety may, in extreme cases be up to 30 percent less than the 2D value, it is not likely to be more than 10 to 15 percent less for most cases. In fact, for many cases the 3D factor of safety is comfortably higher than the 2D factor of safety. Those cases where the three-dimensional stability is likely to be less than that in two-dimensions can usually be determined by inspection, and, for most cases, acceptable estimates of the corresponding differences in factors of safety can be made. For those situations where a complete analysis is needed, SSA-3D can be used. As in any stability analysis, however, the most important information required for accurate and useful results is that pertaining to the shear strengths of the materials involved in the sliding.

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