

MUNICIPAL SOLID WASTE SLOPE FAILURE. II: STABILITY ANALYSES

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ABSTRACT: Analyses are presented to investigate the case of a large slope failure in a municipal solid waste (MSW) landfill that developed through the underlying native soil. The engineering properties of the waste and native soil are described in a companion paper by Eid et al. (2000). Some of the conclusions from this case history include (1) native colluvial/residual soils in the Cincinnati area underlying MSW can mobilize a drained shear strength less than the fully softened value without recent evidence of previous sliding; (2) strain incompatibility and progressive failure can occur between MSW and underlying materials and cause a reduction in the mobilized shear strength; (3) a stability evaluation of interim slopes, especially when the slope toe will be excavated, blasting will be occurring, and waste placement continues at the top of slope, should be conducted, even though it may not be required by regulations; and (4) the reappearance of cracking at the top of an MSW landfill slope is probably an indication of slope instability and not settlement.

INTRODUCTION

On March 9, 1996, the largest slope failure at a municipal solid waste (MSW) landfill in the United States, based on volume of waste involved (approximately 1.2 million m³), occurred, and it provides the industry with some lessons for the operation, expansion, and stability of landfill slopes. This paper describes the wasteslide event, failure mechanism, cause of the wasteslide, slope reconstruction, and lessons learned. A companion paper, Eid et al. (2000), describes the site, subsurface investigation, and engineering properties of the MSW and brown native soil involved in the slide. The landfill is located approximately 15.3 km northwest of Cincinnati, Ohio. At the time of the failure, it was the largest MSW facility in the State of Ohio, based on annual waste receipts, and it accepted an average of 4.5×10^6 kg of waste per day.

LATERAL EXPANSION AND WASTE PLACEMENT ACTIVITIES

In February 1994, the landfill owner/operator was granted a permit for a 486,000 m² lateral expansion. The north slope lateral expansion involved creating a large excavation adjacent to the toe of the existing landfill slope, with a maximum depth of 45 m, and installing a composite liner system in the expansion (Rumpke 1992). The areal extent and geometry of the lateral expansion on February 6, 1996, 32 days prior to the failure are shown in Fig. 1. The excavation for the access road along the toe (see dotted line) of the existing landfill is also shown. A closeup photograph of the toe excavation is shown in Plate 1 of Eid et al. (2000). To the north of the access road is the deep excavation. The outline of the existing landfill involved in the wasteslide is superimposed on Fig. 1 (see dashed line), which is south of the access road. In addition, the dashed line depicts the farthest extent of the waste after the slide. It can be seen that the slide mass essentially filled the deep ex-

cavation, except for a small area at the northeastern edge of the excavation.

Another important feature in Fig. 1 is the vehicle turnaround at the top of the landfill. Approximately one-half of the daily 4.5×10^6 kg of waste was being placed at the top of the slope in the months preceding the slide (King 1998). Placement activities are evident in Fig. 1 and were evident from the presence of three compactors at the top of the landfill immediately after the wasteslide. The other two compactors that were being used at the time of the slide were located at the west ridge placement area and are shown in Fig. 1, with the presence of four unloading trucks.

The large and deep excavation was being continued at the time of the failure after being started in 1993, or approximately 40 months prior to the wasteslide (Strachan 1998). Placement of the compacted clay for the composite liner system had started on a portion of the 3 Horizontal:1 Vertical (3H:1V) slope adjacent to the toe of the existing landfill in December 1995, or approximately three months prior to the slide. The continual collection of leachate and surface water contaminated by leachate in the deep excavation caused many construction delays. Site personnel had to activate a leachate pump (Plate 1 in Eid et al. 2000) approximately once every hour (Wells 1998) to remove liquid from a manhole. This pumping was required year round. In addition, it usually required several days for the leachate pools in the excavation to dry before construction could proceed.

In summary, the composite liner system and subsequent regulatory certification procedure were not going to be completed for a number of months. This delay in construction, and thus new disposal capacity, resulted in a shortage of disposal capacity at the site. Thirty-two days prior to the failure, that is, on February 6, 1996, the site was overbuilt/overfilled by at least 944,300 m³ (Civil 1996b). The north slope was overbuilt by 731,000 m³ on February 6, 1996 (Civil 1996b). Based on a conservative estimate (9.0×10^5 to 1.8×10^6 kg/day), of the waste being placed at the top of the north slope from February 6 through March 9, 1996 (King 1998), the overbuild and maximum height of the landfill on March 9, 1996, were estimated to be at least 776,940 m³ and +338 to +340 m, respectively.

As part of the 1994 expansion permit, the maximum allowable elevation of waste placement was increased from +317.2 to +324.8 m. As a result, the top of the landfill exceeded the permitted elevation by 13 to 15 m at the time of the failure. Fig. 2 shows the waste grades from the owner/operator's annual aerial surveys, dated February 1 and December 21, 1994, and the estimated waste grade on March 9, 1996. (The location of cross section B-B' is shown in Fig. 2 of Eid et al. 2000.) It can be seen that the slope inclination increased from February 1 to December 21, 1994, but the maximum elevation did

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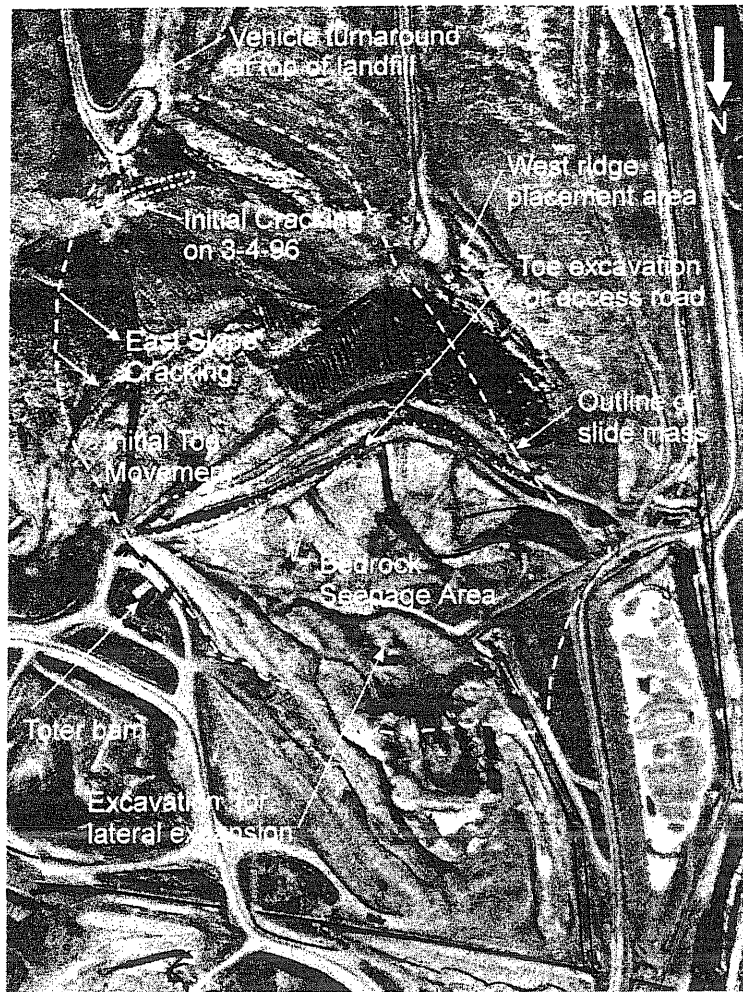


FIG. 1. Aerial View of Landfill on February 6, 1996 (32 Days Prior to Wasteslide)

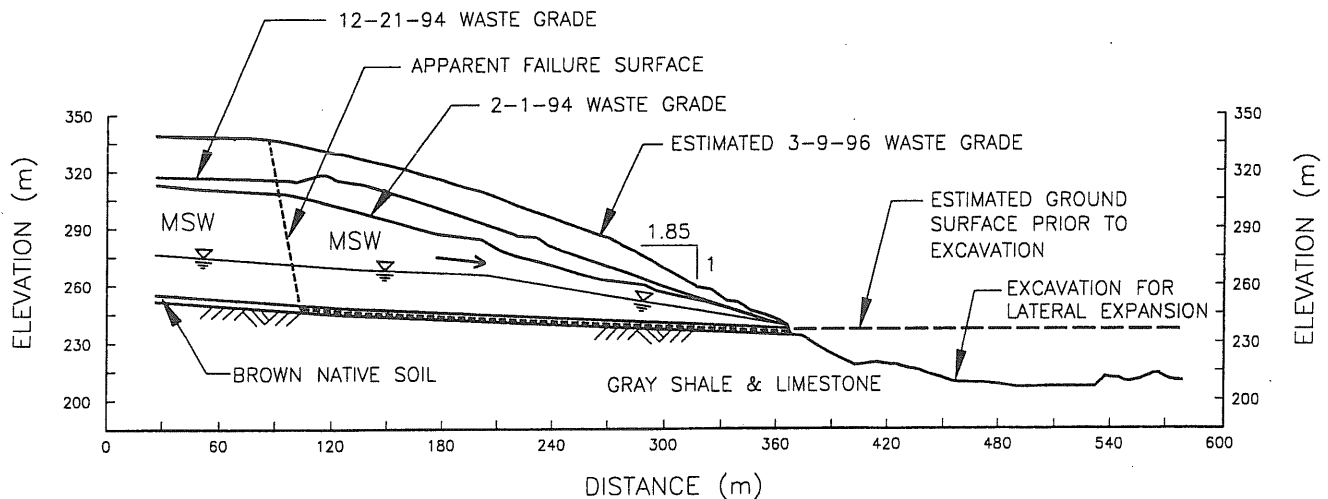
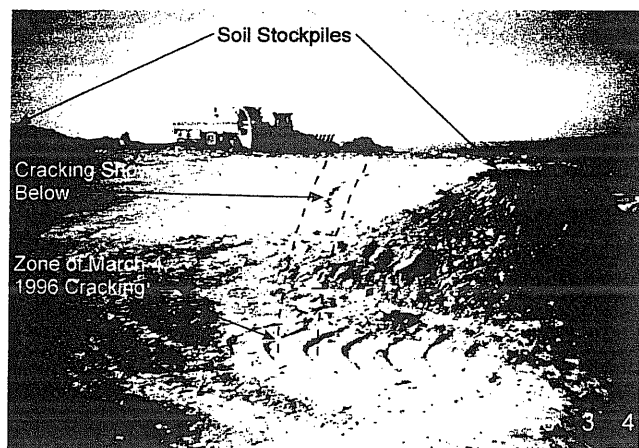


FIG. 2. Slope Cross Section B-B' Showing Recent Waste Placement

not increase significantly. However, from December 21, 1994, to March 9, 1996, both the slope inclination and maximum elevation increased significantly. In fact, a 9 to 20 m thick layer of waste was placed over the majority of the slope between December 21, 1994, and March 9, 1996, which resulted in a significant increase in slope height and inclination. In addition to waste placement, the owner/operator stockpiled soil at the top of the landfill [Fig. 3(a)] for daily cover operations.

The secant slope inclination (inclination of a straight line from the toe to the crest) was 2.6H:1V. However, the slope inclination near the toe of the slope was 1.85H:1V (Fig. 2). A 4.5 to 6.0 m high nearly vertical excavation was also constructed at the toe of the slope to allow construction of an anchor trench for the composite liner system and access road. Fig. 2 also shows the ground surface at the toe of the landfill prior to construction of the deep excavation estimated from a 1955 Quadrangle sheet (USGS 1955).



(a)



(b)

FIG. 3. (a) Waste Placement and Cracking at Top of Landfill on March 4, 1996, and (b) Closeup of March 4, 1996, Cracking

WASTESLIDE EVENT

Field Observations

On the morning of Monday, March 4, 1996 (five days prior to the wasteslide), landfill operating personnel noticed some cracking in recently placed cover soil just north of the vehicle turnaround (Fig. 1) at the top of the slope. Fig. 3(a) shows some of the cracking extending across the top of the landfill with a vertical offset of 25 to 50 mm as waste placement and soil stockpiling activities continued on March 4, 1996. A closeup of the crack [Fig. 3(b)] shows that the longest crack had a width of 75 to 125 mm and extended 15 to 30 m across the crest of the slope. The location of this cracking essentially corresponds to the location of the scarp that resulted from the wasteslide (note truncated vehicle turnaround at top of landfill in Fig. 4) and thus delineates the extent of the slide mass. Owner/operator personnel inspected the entire slope, including the toe, to determine if the cracks were an indication of slope instability. No other cracks or other indications of slope movement were identified on the slope or at the toe on March 4 (Kenter et al. 1996, 1997a, 1997b). Consequently, the owner/operator concluded that the cracks were caused by landfill settlement, and ongoing visual monitoring and covering the area with soil to control infiltration were deemed appropriate. As a result, waste placement continued at the top of the landfill (until the failure occurred), and blasting resumed on March 5.

This cracking reappeared each day in the same location until

the wasteslide occurred on Saturday, March 9, 1996. This cracking was a manifestation of the translational movement that was occurring in the brown native soil underlying the waste. A major problem in MSW slopes is distinguishing settlement-induced cracks from slope instability cracks. The reappearance of significant cracking in the same location is more likely an indication of slope instability than settlement. In general, settlement cracks will not reappear in the same location in a short period of time because the biological and mechanical processes that cause the settlement require time.

Between the end of work on Friday, March 8, and approximately 7:00 a.m. on Saturday, March 9, the cracks at the top opened again and widened substantially, and 0.45 to 0.75 m of vertical displacement had occurred at the northern edge of the vehicle turnaround (King 1998). In addition, the cracking had extended down the east side of the slope towards the Toter Barn (Fig. 1), and steam was emanating from the cracks. The cracks were estimated to be at least 3 m deep and were widening and deepening. This vertical displacement was probably the start of a graben formation (Fig. 5), and the crack just forming approximately 30 m down the 450 m long north slope (King 1998) was probably formation of the downslope or northern edge of the graben.

Between 8:00 and 8:30 a.m., the toe of the landfill started moving slowly toward the access road. Photographs show the movement was occurring in the brown native soil between the waste and gray shale. The soil and waste were moving slowly onto the access road and toward the deep excavation. This initial toe movement was observed on the eastern flank of the ravine underlying the MSW (Fig. 1). The axis of the ravine corresponds to the western end of the access road, where it makes a sharp northeasterly turn. Cracking and the associated vertical displacement continued and extended across the top of the landfill and down the east and west sides of the north slope by 11:00 to 11:30 a.m. (J. Holm, personal communication, Civil and Environmental Consultants, Cincinnati, Ohio, 1997). The cracking and movement continued until between 11:30 a.m. and noon. At this time, a large slide block accelerated toward the deep excavation, and the entire slide was complete in 1 to 5 min (Holm 1997; King 1998; and Strachan 1998). After movement of the large slide block and formation of a graben and scarp, some sloughing occurred at the top of the landfill, which resulted in the final scarp truncating the vehicle turnaround.

An area of approximately 81,000 m² of waste slid into the 44,500 m² excavation at the toe of the existing slope, resulting in a total of about 125,500 m² of exposed solid waste. The toe of the existing slope moved approximately 245 to 275 m to the northern edge of the deep excavation, where it was stopped by the northern wall of the excavation or came to rest in an open part of the excavation. As the waste moved into the deep excavation, it broke into blocks, or wastebergs, as the slide mass accelerated down the 3H:1V excavation slope at the toe of the existing landfill. Some of these wastebergs were 24 m high (Fig. 4). Fig. 6 compares the prefailure and postfailure geometry of the slope. Fortunately, no injuries or loss of life occurred, but some of the large mining and earth moving equipment being used in the excavation was damaged. In addition, the extensive gas recovery system was damaged and rendered inoperable. The slope was reconstructed by using two rock berms at the toe of the slide mass and rebuilding the slope from the toe of the slide mass up toward the scarp. This involved placing new solid waste at the toe of the slope and in the graben area and excavating the overbuilt waste from the top of the slope to achieve the final slope of approximately 5H:1V. The majority of the slope reconstruction was completed by January 1997.

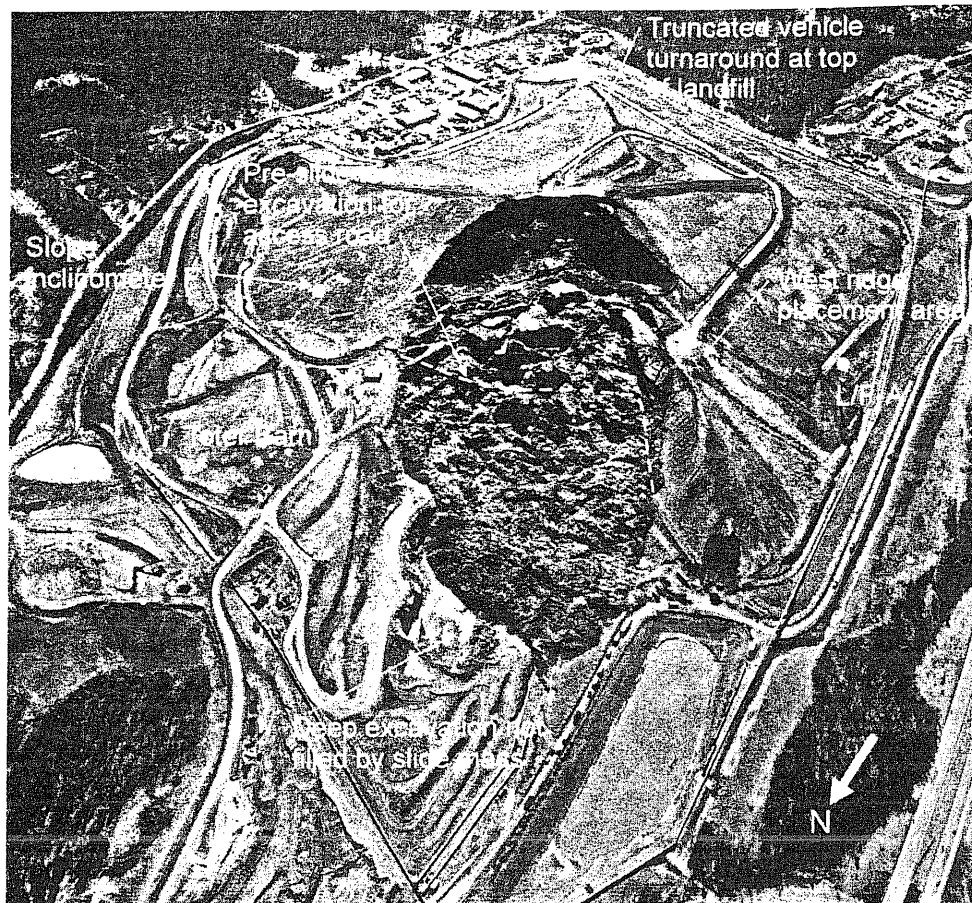


FIG. 4. Aerial View of Wasteslide on March 11, 1996 (Two Days after Wasteslide)

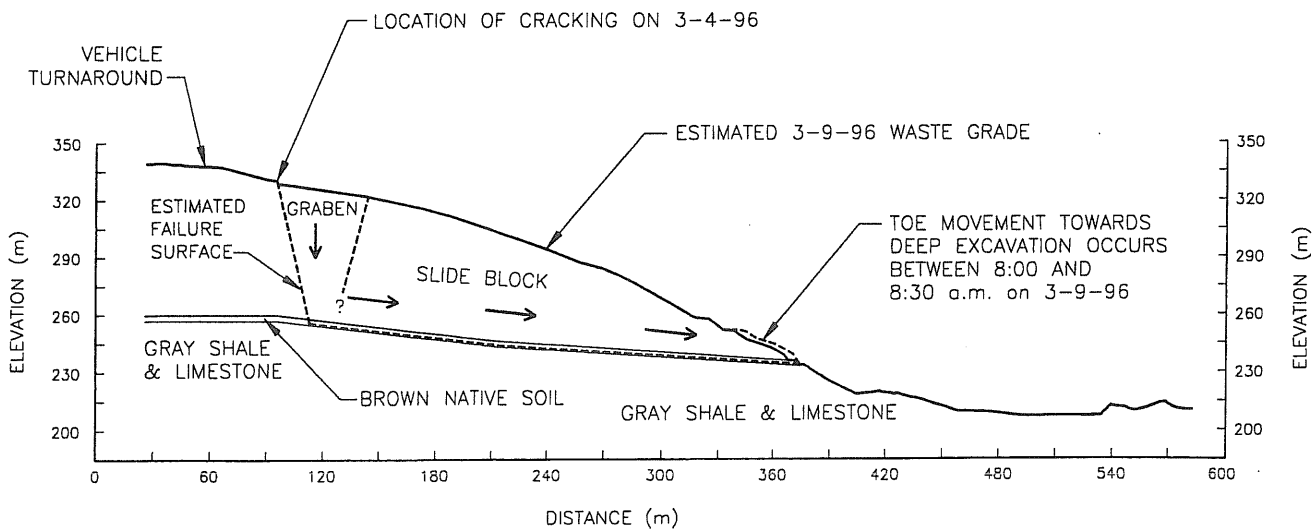


FIG. 5. Cracking and Translational Movement of Waste Slope along Cross Section A-A'

Translational Failure Mechanism

The appearance of substantial cracking initially at the top of the slope and formation of a large slide block, graben, and nearly vertical scarp (Figs. 4–6) are characteristic of a translational failure (Cruden and Varnes 1996). In addition, observations at the slope toe and slope inclinometer readings after the slide, discussed subsequently, suggest that the failure was translational and occurred through the weak brown native soil underlying the MSW. Based on field observations and the results of a subsurface investigation, the failure surface was estimated to have passed through the solid waste at a steep in-

clination to the underlying weak, saturated brown native soil (Fig. 5). The failure surface continued along the brown native soil until it daylighted at the vertical excavation at the slope toe that exposed the MSW and native soil.

Progressive Failure and Acceleration of Slide Mass

As noted previously, a large slide block accelerated toward the excavation around noon on March 9. It is anticipated that the primary cause of the acceleration was the progressive failure that was occurring in the brown native soil. The significant cracking (Fig. 3) observed at the top of the landfill on March

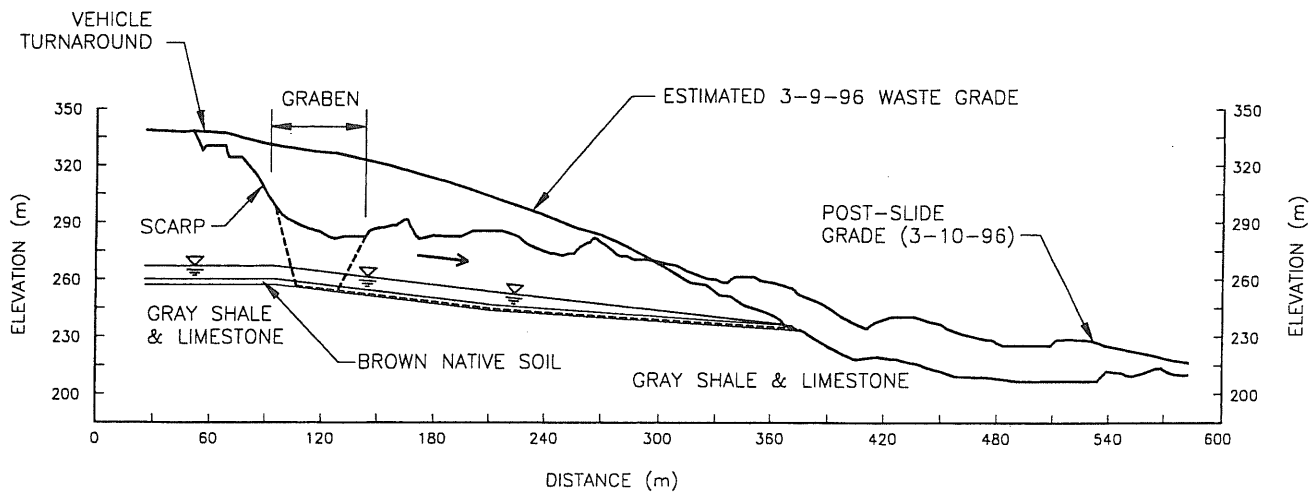


FIG. 6. Graben and Slope Geometry after Translational Movement of Waste Slope along Cross Section A-A'

4, 1996, was a manifestation of the shear failure that was occurring, and probably had been occurring, in the brown native soil at this location of the failure surface. As the fully softened shear resistance of the brown native soil was exceeded in the vicinity of this cracking, shear stresses were transferred to the native soil farther down the slope. Therefore, cracking reappeared the following day at the top of the slope because this MSW and native soil displaced downslope to mobilize additional shear resistance. On March 5, the largest blast in the past 33 days occurred, causing additional shear displacement and stress transfer. Therefore, this blast accelerated the timing or occurrence of the slope failure, as discussed subsequently. As the shear resistance of the downslope brown native soil was exceeded, shear stresses were transferred farther down the slope, and thus cracking reappeared on March 6, 1996. Shear stresses continued to be transferred down the slope toward the toe from March 4 to March 9, 1996, resulting in continued and substantial cracking at the top of the slope. Between the evening of March 8 and the morning of March 9, 1996, the shear stresses and progressive failure had been transferred to the vicinity of the slope toe at the eastern flank of the landfill. Since the toe had no buttress and the shear stresses exceeded the shear resistance of the brown native soil on the eastern flank of the landfill (Fig. 1), the toe started moving toward the access road. After the entire failure surface had been engaged in the brown native soil, the entire slide mass accelerated toward the access road. As the slope toe reached the northern edge of the access road, the materials descended into the deep excavation on the eastern flank of the ravine. As the materials descended, the buttressing effect that would have existed had the ground surface not been excavated (Fig. 2) was absent, and the slide mass accelerated, resulting in the global failure.

BACK-ANALYSIS OF WASTESLIDE

A 3D back-analysis of the wasteslide was conducted to investigate the shear behavior of the brown native soil. A 3D analysis was used to account for the complex geometry of the scarp (Fig. 4), slide mass (Fig. 1), original ground surface, including, for example, the presence of a ravine under the MSW, landfill geometry, and piezometric level, which varied due to the underlying topography. In addition, a 3D analysis was used to account for the end effects (i.e., shear forces along the sides of the slide mass) and thus yield a back-calculated shear strength that is in agreement with the field or mobilized strength. Two-dimensional (2D) analyses do not account for the 3D end effects and thus yield estimates of the mobilized shear strength that are too high, or unconservative (Stark and Eid 1998). As will be described subsequently, the difference

TABLE 1. Summary of Input Parameters for 3D Back-Analysis of Wasteslide

Material type (1)	Total unit weight (kN/m ³) (2)	Saturated unit weight (kN/m ³) (3)	Effective Stress Shear Strength Parameters	
			Cohesion (kPa) (4)	Friction angle (degrees) (5)
Interim soil cover	17.3	17.8	0	30
Solid waste	10.2	11.8	40	35
Brown native soil	19.0	19.7	≅0	Back-calculated

between the 2D and 3D back-calculated/mobilized (mob) effective stress friction angles (ϕ'_{mob}) for the brown native soil is 3 to 4°, or 25 to 30%.

Janbu's simplified method of slices (Janbu 1968) was used to conduct the 3D limit equilibrium back analysis. Janbu's simplified method was extended to three dimensions and coded in the microcomputer program CLARA 2.31 (Hungry 1988). The 3D geometry of the slide mass and piezometric level were described using eighteen 2D cross sections. The program interpolates between the 2D cross sections to define the 3D geometry. The original ground topography was estimated from the 1955 Quadrangle Sheet (USGS 1955), and the waste surface geometry on March 9, 1996, was estimated from the February 6, 1996 (one month prior to the wasteslide) aerial survey and the disposal volume placed at the top of the landfill since February 6, 1996. Unfortunately, the ground topography prior to waste placement in 1945 is not available, but results of the subsurface investigation suggest that the 1955 Quadrangle Sheet provides a good estimate of the topography prior to waste placement. Table 1 summarizes the input parameters used in the 3D analysis. A seismic coefficient was not used in the back-analysis to simulate blasting because the blasting probably excited only a limited portion of the failure surface at a given time, and no blasts were detonated at the time of failure.

Shear Strength and Unit Weight of Municipal Solid Waste

One of the uncertainties in the 3D analysis was the shear strength of the solid waste at this site. A companion paper (Eid et al. 2000) presents the analyses, test results, and other case histories used to estimate the site-specific waste shear strength parameters (friction angle, ϕ' , of 35° and cohesion, c' , of 40 kPa) shown in Table 1.

A sensitivity analysis of the relationship between the total or moist unit weight of the MSW and the ϕ'_{mob} for the brown

native soil and was conducted using a 3D analysis and the estimated piezometric level (described subsequently). The 3D analysis showed that the ϕ'_{mob} of the brown native soil ranges from approximately 11 to 13° for MSW total unit weights ranging from 14 to 8 kN/m³, respectively. The decrease in ϕ'_{mob} with increasing unit weight is probably caused by the unit weight increasing the normal stress on the deep, long, and nearly horizontal failure surface faster than the corresponding increase in driving shear stress. Since the typical range of total unit weight for in-place waste is 8 to 14 kN/m³ (Fassett et al. 1994), it was concluded that the variation in total unit weight only could affect the ϕ'_{mob} of the brown native soil by 1 to 2°. An average total and saturated unit weight of 10.2 and 11.8 kN/m³, respectively, were used in the subsequent analyses (Table 1).

Estimated Piezometric Level

The piezometric level acting on the slide plane at the time of failure was not measured. As a result, the piezometric level had to be estimated to conduct the 3D back-analysis. The total piezometric pressure acting on the failure surface at this site is composed of the piezometric pressure caused by leachate in the waste and the fluid in the jointed bedrock underlying the brown native soil. The following paragraphs discuss how the total piezometric level was estimated for cross sections A–A' (Fig. 6) and B–B' (Fig. 2).

The piezometric level in cross section A–A' (Fig. 6) was based on observations after the slide, information from Boring G (Fig. 2 in Eid et al. 2000), and Boring L/F-A (Fig. 4), which is located on the western flank of the ravine underlying the MSW and is the counterpart to Boring G because both borings are located on a flank of the ravine underlying the MSW and thus should exhibit comparable leachate and bedrock piezometric levels.

In general, the waste pile was observed to be fairly dry immediately after the failure. Additional evidence of this dry state included the ability to read pieces of dry newspaper obtained from Boring G at a depth of 23 m after the failure; the absence of significant leachate observed exiting the 30 to 50 m high vertical scarp; and borings in the deep excavation (e.g., Boring D); and shafts to construct gas wells that did not yield significant depths of leachate after the failure. Since dry newspaper was found at a depth of 23 m in Boring G, the maximum depth of leachate prior to failure was assumed to be 7 to 8 m because the depth of waste prior to the slide at Boring G was approximately 30 m. This is in agreement with Boring L/F-A, which yielded a leachate depth of 6 to 6.5 m. Further confirmation was observation of a leachate pool that developed about three months after the failure at the base of the graben near the Toter Barn, which had a leachate elevation of +268.4 m, or a depth of 9 to 10 m above the native soil. Since the leachate pool was influenced by precipitation into the graben after the failure, the leachate level near the scarp along cross section A–A' was estimated to be 7 to 8 m at the time of failure.

The leachate level was assumed to parallel the bedrock and maintain a depth of 7 to 8 m to approximately the middle of the slide mass and then decrease to the top of the brown native soil at the slope toe. The leachate level was assumed to decrease to the top of the brown native soil at the slope toe in cross section A–A' because of the following observations: (1) No leachate outbreaks were observed in the cover soil or near the toe prior to failure; (2) the vertical cut for the access road exposed permeable MSW and the underlying brown native soil and only minor seepage, not large quantities of liquid, exited this area prior to the slope cracking on March 4, 1996; (3) no significant quantities of liquid exited the slope toe between 8:00 and 8:30 a.m. on March 9, when it started to move

toward the access road; (4) an employee (Wells 1998) in the deep excavation at the time of the failure did not witness a significant amount of liquid exiting the slope at the onset of failure; (5) the eastern flank is at a higher elevation than the ravine axis, and thus the leachate would flow toward the ravine and not the flank where the failure initiated; and (6) initial cracking occurred at the top of the slope and not at the toe, indicating that failure initiated under the slope and thus there was not a large buildup of liquid at the toe; otherwise, cracking would have initiated at the toe. As a result, the leachate level in cross section A–A' was assumed to parallel the bedrock at a depth of 7 to 8 m until midway between the scarp and slope toe, which is about the location of the west ridge placement area, and then decreased to the top of the brown native soil at the slope toe. Based on this information, it is anticipated that the piezometric level shown in cross section A–A' provides the best estimate of the piezometric conditions in the slope prior to failure.

The piezometric level in cross section B–B' (Fig. 2) was based on the piezometric level derived for cross section A–A', observations before and after the slide, and piezometer L/F-B (Fig. 4). Based on the natural topography, the leachate level was estimated to be about 15 m deeper at the axis of the ravine. This was estimated from the 15 m change in the top of the native soil from Boring G to the axis of the ravine. Piezometer L/F-B, which is located outside of the southern end of the ravine, showed a leachate level of 14 m. This leachate level is in agreement with a maximum depth of leachate in the ravine axis of 22 to 23 m, which is 15 m plus the 7 to 8 m assumed for cross section A–A'. The maximum depth of leachate occurring at the ravine axis is also in agreement with the leachate manhole, pump, and associated piping being located near the ravine axis (Plate 1 in Eid et al. 2000).

Another source of piezometric pressure acting on the failure surface was probably the underlying jointed shale and limestone bedrock. Civil (1996a) and Kenter et al. (1996, 1997a, b) reported that a "seep" from the toe of the existing waste pile and exposed bedrock was present throughout the winter prior to the wasteslide. This seepage required construction of a collection trench about 9 to 12 m below the access road near the ravine axis. Additional evidence of this bedrock seepage is shown in Fig. 1, where seepage broke through the compacted clay (CCL) and is visible on the 3H:1V slope. To satisfy regulatory requirements, the CCL hydraulic conductivity was probably at or less than 10⁻⁹ m/s. In addition, placement of the CCL could have facilitated buildup of piezometric and gas pressure from the blasting in the bedrock. The presence of piezometric pressure in the bedrock also may have contributed to the weak and saturated nature of the brown native soil found underneath the stiff brown native soil in Boring G (Fig. 4 in Eid et al. 2000). Other sites in Ohio have also experienced ground-water migrating through a CCL placed over jointed bedrock. At these sites, a drainage layer was installed between the CCL and bedrock to relieve the piezometric pressure. The presence of piezometric pressure in the jointed bedrock may help explain the timing of the failure because the site was starting to experience springtime rainfall. The snowfall and rainfall at the Cincinnati Airport for the months of October 1995 to March 9, 1996, are 109, 55, 87, 109, 50, and 18 mm, respectively. The majority of this precipitation probably did not infiltrate the MSW because of the presence of a soil cover and the landfill steep slopes. However, it could have infiltrated the surrounding natural ground surface and influenced the piezometric pressure in the jointed bedrock. Based on hydraulic fracturing theory (Widjaja et al. 1984), the piezometric pressure required to rupture the CCL at zero confining stress corresponds to a water level of 1.5 to 3 m. As a result, a piezometric head of 1.5 to 3 m was added to develop the total

piezometric level acting on the failure surface for the back-analysis.

Piezometric Sensitivity Analysis

Using 18 cross sections, including A–A' and B–B', the 3D shape and height of the total piezometric level were estimated. The maximum piezometric level was assumed to occur at the bottom or axis of the ravine near the scarp and to be 24.5 m, which corresponds to a leachate level of 22 to 23 m and 1.5 to 3 m of piezometric head in the native soil/bedrock. The lowest piezometric level occurred on the eastern flank of the ravine because of differences in foundation elevation. This resulted in the leachate level being lower on the eastern side of the slide mass than at the ravine axis, which was initially puzzling because the failure initiated on the eastern flank and not at the ravine axis. The failure probably did not initiate at the ravine axis because of the 3D effect, where the access road makes a sharp northeasterly turn (Fig. 1). The confinement provided to the slide mass by the buttressing waste on the western side of the slide mass increased the local factor of safety (FS) such that failure occurred on the eastern flank instead of at the ravine axis or the location of the maximum piezometric level.

Fig. 7 presents a sensitivity analysis of the relationship between maximum piezometric level at the ravine axis near the scarp and the ϕ'_{mob} for the brown native soil using 2D and 3D analyses. A 2 m increase in the piezometric level in Fig. 7 corresponds to the entire level being increased by 2 m. Therefore, the depth at the axis of the ravine and eastern flank would both increase by 2 m. The 3D analysis shows that the ϕ'_{mob} of the brown native soil ranges from approximately 11.5 to 14.5° for a practical range of maximum piezometric level of 11 to 30 m, respectively. Using a maximum piezometric level of 24.5 m, the average mobilized friction angle for the brown soil is 13.5°. When side resistance is considered, the 3D FS shows less sensitivity to the estimated piezometric level than the 2D FS. This is attributed to the larger shear forces along the sides of the nonsymmetrical slide mass reducing the effect of the piezometric level on the calculation of effective and driving stress at the base of the slide mass.

Fig. 7 also shows ϕ'_{mob} for the brown native soil if the side resistance is not considered in the 3D analyses. This analysis

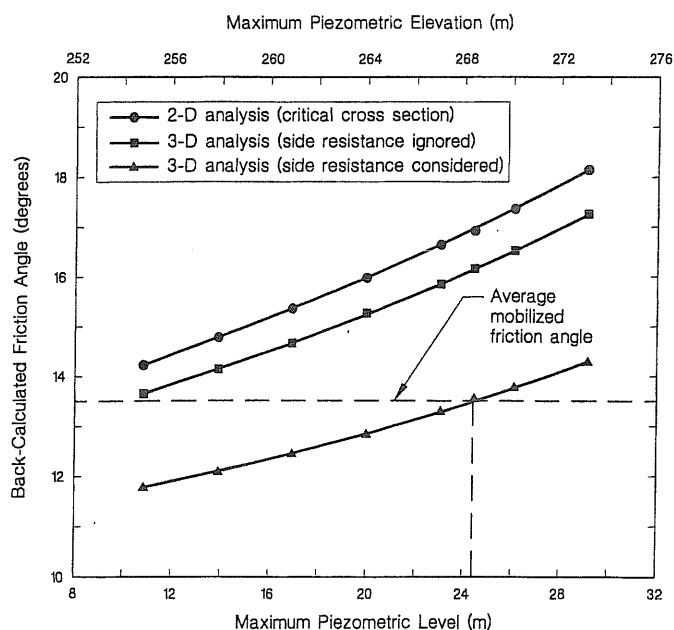


FIG. 7. Maximum Piezometric Level at Ravine Axis and Associated Back-Calculated Friction Angle for Brown Native Soil

is similar to a 2D stability analysis condition in that no 3D side forces are included in the analysis. This analysis is not a true 2D analysis because the entire slide mass is considered instead of a single cross section. As a result, this analysis provides an FS that is between the 3D and 2D analyses. It can be seen that ignoring the side resistance leads to an increased mobilized friction angle by 15 to 20%.

Finally, Fig. 7 shows the 2D analysis yielded a ϕ'_{mob} of approximately 14 to 18° for the brown native soil for a maximum leachate level ranging from 11 to 30 m, respectively. The analyses were performed using the same stability method—that is, Janbu's simplified method of slices (Janbu 1968)—as the 3D analyses using the 2D option in CLARA 2.31 (Hung 1988) and cross section B–B'. As expected, the 2D analysis yields the highest values of ϕ'_{mob} . For a maximum piezometric level at the scarp of approximately 24.5 m, the 2-D ϕ'_{mob} of the brown native soil is about 17°. [Use of Spencer's (1967) stability method and cross section B–B' yielded a ϕ'_{mob} of 16°.] For comparison with cross section B–B', the 2D ϕ'_{mob} calculated using both cross section A–A' and the piezometric level shown in Fig. 6 was 14°, instead of 17°. Since toe movement initiated in the vicinity of cross section A–A', the 2D ϕ'_{mob} of 14° may be more representative than 17°.

The difference between the 2D and 3D analyses without side resistance illustrates the effect of representing the slide mass with a 2D section instead of using the 3D geometry. This difference in ϕ' corresponds to approximately 1°. In addition, the difference between the 2D and the 3D analyses with side resistance depicts the degree of unconservatism induced by using a 2D analysis in the back-calculation of this slope failure. It can be seen for a maximum piezometric level at the scarp of 24.5 m that the 2D and 3D ϕ'_{mob} differ by 3 to 4°, or 25 to 30%.

Back-Calculated Shear Strength of Brown Native Soil

Using a vertical scarp height of 30 m to estimate the waste shear strength (Eid et al. 2000) and the possible range of piezometric levels, the 3D back-analysis revealed that the shear strength of the weak, saturated brown native soil at the time of failure could be characterized using an average effective stress angle of internal friction of 13.5° and an effective stress cohesion of zero. Since the measured fully softened friction angle of the brown native soil is 23° (Fig. 7 in Eid et al. 2000), it was thought that the 2D and 3D ϕ'_{mob} (14 to 18° and 11.5 to 14°, respectively) correspond to a postpeak condition. Even if the maximum piezometric level is significantly different than 24.5 m, for example, 11 to 30 m, ϕ'_{mob} of the brown native soil still corresponds to a postpeak condition.

Laboratory test results (Eid et al. 2000) and an empirical correlation (Stark and Eid 1994) show that the drained residual friction angle of the brown native soil from Boring G is approximately 10°. Since the 3D back-analysis estimated the average mobilized friction angle to be 13.5°, it was concluded that the brown native soil mobilized a shear strength between the fully softened (23°) and residual (10°) values at the time of failure.

CAUSE OF WASTESLIDE

The main cause of the wasteslide is the mobilization of a postpeak shear strength in the brown native soil. However, a number of additional causes for the wasteslide and possibilities for mobilizing a postpeak strength were investigated, including weak layers of sludge or waste; seismicity; excess pore-water pressures due to waste placement; shear behavior of the brown native soil; strain incompatibility and progressive failure; lateral displacement of MSW; slope overbuild; rock blasting in the adjacent excavation; and toe excavation. The following

paragraphs investigate the significance of each of these phenomena for finding the cause of the wasteslide and for mobilization of a postpeak strength in the brown native soil.

Weak Waste Layers and Seismic Activity

There is a history of sludge disposal at the site, but the slide was translational, occurring through the brown native soil underlying the waste, as discussed previously, and not through the waste. Therefore, failure of a continuous, weak waste layer was not considered to be a causative factor. No seismic activity was reported or measured within a 170 km radius of the site for 30 days prior to the wasteslide. Therefore, seismicity was not considered to be a significant factor.

Excess Pore-Water Pressure Due to Filling

Based on annual owner/operator aerial surveys, a 9 to 20 m thick layer of waste was placed over the existing 60 m high waste pile within the 14 months prior to the wasteslide (Fig. 2). This corresponds to an average filling rate across the slope of 0.02 to 0.05 m/day. It was thought that this filling might generate excess pore-water pressures in the brown native soil and consequently a decrease in the effective normal stresses acting on the failure surface. Boussinesq stress distribution theory showed that the increase in vertical stress at the base of the 60 m high waste pile due to the placement of 0.02 to 0.05 m of waste/day over the slope was small. The maximum vertical stress increase in the brown native soil was found to occur at the slope midheight, that is, only about 38 m of existing waste and 14 m of new waste in 14 months, and ranges from 20 to 25% of the existing vertical stress. Based on Terzaghi's theory of consolidation (Terzaghi et al. 1996), a degree of consolidation of 90%, double vertical drainage, layer thickness of 3 m, and coefficient of consolidation of 4 m²/year estimated from oedometer tests on specimens of the native brown soil, the majority of the excess pore-water pressure should have dissipated during the 14 month waste placement period. In summary, if excess pore-water pressures were generated in the brown native soil due to waste placement, it would have been only a minor contributing factor in the cause of the failure.

Shear Behavior of Brown Native Soil

Review of landfill operational information indicates that this slope had not experienced a global failure of the north slope since waste placement commenced, which would suggest that a drained fully softened friction angle (23°) should have been mobilized and not a postpeak value. Therefore, mobilization of a postpeak shear strength at this site initially was perplexing. As a result, a study was conducted to determine the shear strength mobilized in other slope failures involving similar geologic and colluvial/residual soil conditions. As noted in the companion paper (Eid et al. 2000), the parent material (shale, in contrast to granite) and a number of other factors, such as weathering, soil formation, deposition, shear displacement prior to waste placement, and aging processes of similar deposits, have resulted in mobilization of a postpeak shear strength at other sites. Other significant causes (discussed subsequently) for shear displacement and mobilization of a postpeak strength in the brown native soil include strain incompatibility between the MSW and brown native soil and progressive failure, time-dependent lateral displacement of the MSW, blasting activities, stress concentration(s) caused by a toe excavation, and waste placement activities that usually involved pushing waste from the top to the bottom of the ravine.

Strain Incompatibility and Progressive Failure

Strain incompatibility between the MSW and underlying brown native soil and subsequent progressive failure can result in mobilization of shear strengths that are less than the peak values. The large difference between the stress-strain characteristics of the MSW and brown native soil (Fig. 6 in Eid et al. 2000) increased the likelihood of strain incompatibility. As a result, failure probably occurred first in the native soil when only a fraction of the MSW peak strength was mobilized. After failure occurred in the native soil, the peak strength of the MSW was mobilized at a time when the shear strength of the native soil had declined to a value significantly less than the fully softened value. Chirapuntu and Duncan (1975) show that the reduction in shear strength caused by progressive failure for a foundation soil and overlying embankment can range from 0 to 20% and 20 to 90%, respectively.

To investigate the effect of strain incompatibility on ϕ'_{mob} , a 3D analysis was conducted with a maximum piezometric level of 24.5 m and zero shear strength ($c' = \phi' = 0$) assigned to the MSW. The resulting ϕ'_{mob} for the brown native soil is 18° versus 13.5° for MSW shear strength parameters of $c' = 40$ kPa and $\phi'_{mob} = 35°$. This result has the following important implications for the understanding of this case history: (1) Only a small percentage of the peak MSW shear strength was mobilized because of strain incompatibility with the underlying brown native soil; and (2) the 3D ϕ'_{mob} of 18°, assuming zero shear strength for the waste, is still significantly less than the fully softened ϕ' of 23°. This analysis further suggests that a postpeak strength was mobilized in the weak, saturated brown native soil, probably in part by the large strain incompatibility between the MSW and native soil.

In summary, strain incompatibility probably facilitated development of a postpeak strength in the brown native soil and the progressive failure mechanism described previously. Brandl (1998) concluded that strain incompatibility must be considered in landfill stability analyses, especially if the subsoil consists of clays or silts with a low residual ϕ' . If the subsoil has a low residual ϕ' , Brandl (1998) recommends comprehensive monitoring of slope deformations to warn of potential failure. A similar conclusion concerning postpeak strength was drawn for geosynthetic interfaces (Stark and Poepfel 1994) and other foundation soils (Mitchell et al. 1995) because of their strain incompatibility with MSW.

Lateral Displacement of MSW

Fig. 8 presents data from slope inclinometer F (Fig. 4) on the eastern slope from July 9, 1996, to January 17, 1997. It can be seen that the MSW shows a consistent out-of-slope movement. Siegel et al. (1990) and Coduto and Huitric (1990) have reported similar time dependent out-of-slope movement of MSW slopes. More important, the out-of-slope movement is inducing shear displacement in the brown native soil that underlies the eastern slope. The corresponding shear displacements may have been even greater under the north slope because the toe was excavated and the underlying topography dips toward the deep excavation. In summary, the time-dependent lateral displacement of MSW also may have induced shear displacement in the brown native soil that could have facilitated mobilization of a postpeak shear strength.

Slope Overbuild

Another major contributor to the wasteslide was the overbuild of at least 776,940 m³ on the north slope at the time of failure. Using a maximum piezometric level of 24.5 m, the slope toe being excavated, and a ϕ'_{mob} of the brown native soil of 13.5°, the 3D FS for the slope was calculated as the max-

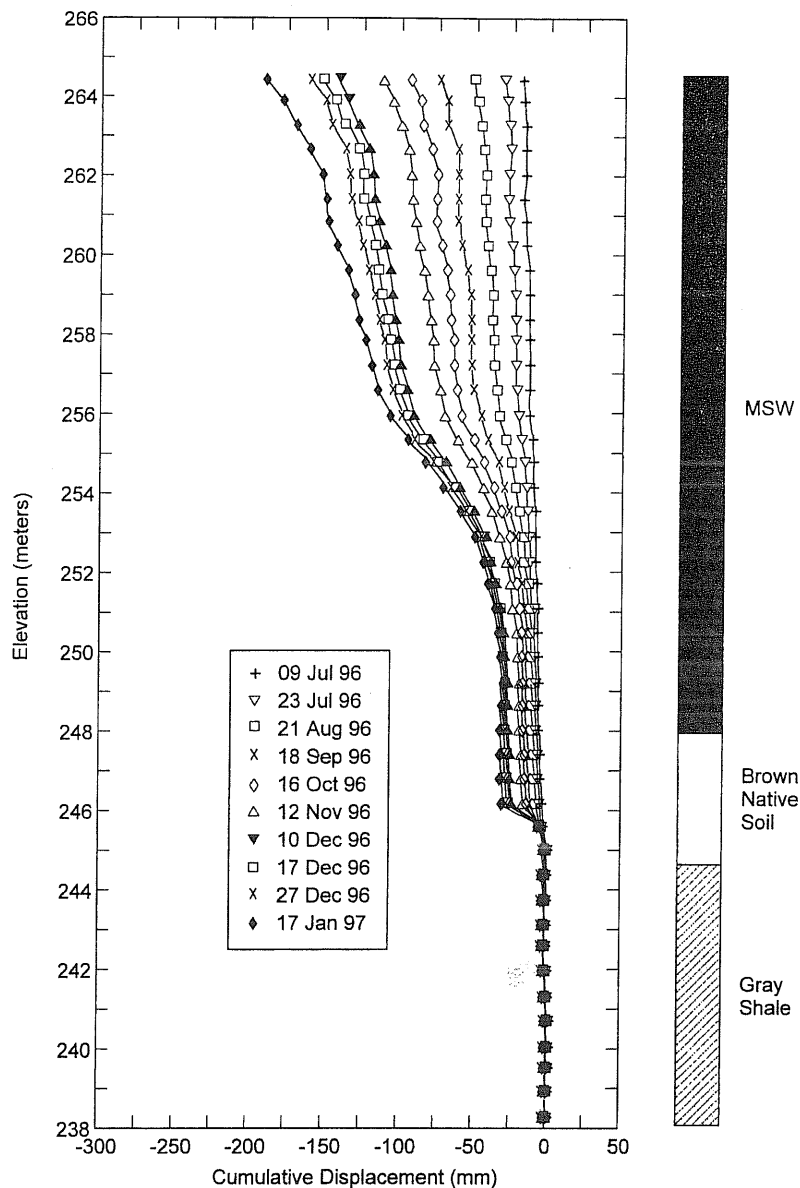


FIG. 8. Slope Inclinometer F from Eastern Slope after Wasteslide

imum height of the landfill increased from elevation +320 m to approximately elevation +340 m. The analysis reveals that the 3D FS decreased from slightly less than 1.2 at elevation +320 m to unity at elevation +339 m. An elevation of +339 m is in agreement with the estimated maximum elevation of +338 to +340 m at the time of failure. Therefore, it was concluded that the overbuild was a factor in the cause of the wasteslide because it reduced the FS to near unity. This marginal stability allowed the rock blasting to have a greater impact on the progressive failure, that is, strength loss and thus transfer of shear stress along the failure surface, in the brown native soil.

Rock Blasting in Adjacent Excavation

It was anticipated that the rock blasting, using an ammonia nitrate/fuel oil explosive adjacent to the steep, marginally stable slope, contributed to mobilization of a postpeak strength condition and occurrence of the wasteslide by inducing shear stresses and displacement in the brown native soil and possibly causing gas pressure to develop in the jointed bedrock. To investigate the possibility of blast-induced displacement in the brown native soil, a multiple sliding block analysis (Dowding 1996) was conducted, instead of a rigid block analysis (New-

mark 1965), to account for the one- or two-order-of-magnitude difference in frequency content, and thus predominant wavelength, between earthquakes and construction blasting vibrations.

Using a maximum charge per delay of 91.7 kg for the March 1, 1996, blast (see Table 2 for summary of recent blasting activities); a distance from the blast to the scarp area where cracking was first observed on March 4, 1996, of approximately 400 m; a blast frequency of 30 to 70 Hz obtained from blasting records at a distance of 850 m; a number of significant cycles per blast of 4 to 6; an assumed FS of the north slope of 1.16; a peak particle velocity of 5.1 to 12.5 mm/s based on the range of cube-root scaling of the recorded peak particle velocities measured during each of the blasts in Table 2; and an estimated shear wave velocity of the jointed shale and limestone bedrock of 1525 m/s (Dowding 1996), a shear displacement of roughly 7 to 11 mm was estimated for the interface between the bedrock and brown native soil. This is significant because the ring shear displacement required to mobilize the fully softened shear strength of the remolded brown native soil is only 2 to 6 mm. An assumed FS of 1.16 was used because Dowding (1996) presents relationships between shear displacement, frequency, and FS, with the lowest value of FS

TABLE 2. Summary of Recent Blasting in Deep Excavation

Date (1)	Total weight of explosives (kg) (2)	Maximum weight per 8 ms delay (kg) (3)	Number of delay periods used (4)	Peak particle velocity vector sum at distance of 549 m (mm/s) (5)	Time of peak velocity vector sum (ms) (6)
2/2/96	4,260.3	92.6	10	5.5	379
2/6/96	3,148.9	92.6	6	5.2	332
2/9/96	3,704.6	92.6	7	5.3	305
2/12/96	2,408.0	92.6	7	4.1	417
2/13/96	2,408.0	92.6	7	5.5	317
2/14/96	1,852.3	92.6	5	4.6	558
2/15/96	2,224.6	79.5	7	4.0	344
2/16/96	2,224.6	79.5	7	2.9	286
2/20/96	2,202.4	66.7	15	3.6	450
2/21/96	2,269.1	66.7	15	2.4	524
2/22/96	3,098.6	79.5	3	1.9	400
2/23/96	2,198.7	75.8	3	3.0	454
2/27/96	2,589.6	83.5	3	3.0	499
3/1/96	3,301.5	91.7	3	1.9 ^a	387
3/5/96	4,981.3	95.8	3	2.4 ^a	476

^aPeak particle velocity vector sum at distance of 610 m.

being 1.16. The amount of shear displacement increases with decreasing FS; therefore, a shear displacement of 7 to 11 mm is a lower bound because the local FS was less than 1.16, as evidenced by cracking at the top of the slope on March 4.

The length of the slide block that can be displaced or accelerated in the same direction by a blast was estimated to be approximately one-eighth of the wavelength (19 to 51 m), or 3 to 7 m (Dowding 1996). Therefore, only a small length of the failure surface at a given instant was undergoing unidirectional shear displacement during the March 1, 1996, blast. Since cracking appeared on March 4, the local FS near the scarp was probably near unity on March 1. As a result, the blast on March 1, 1996, probably caused shear displacement and some shear stress transfer to occur in the brown native soil, which helped accelerate the progressive failure. The blasts prior to March 1 were also adversely influencing the slope because the slope was marginally stable, that is, $FS \leq 1.1$ to 1.2. In summary, the combination of the March 1 blast and continued waste placement at the top of the slope helped accelerate the progressive failure mechanism.

The initial observation of progressive failure was on March 4, with the cracks at the top of the landfill. Therefore, an even greater shear displacement and shear stress transfer were induced in the brown native soil due to the March 5 blast because the local FS was at or below unity in the scarp area, and the March 5 blast was the largest in the past 33 days. This blast further accelerated the progressive failure, which resulted in increased cracking at the slope crest in the following days, leading to the global failure on March 9, 1996. If no blasting had occurred on March 5, global failure probably would have occurred after March 9. Since the local FS was at or near unity in the eventual scarp area because cracking first appeared at the slope crest, the primary impact of the blasting was on this portion of the failure surface. The local FS at the slope toe was greater than unity, and thus no cracking was observed at the toe after the March 5 blast. This is also in agreement with the progressive failure mechanism and acceleration of the slide mass discussed previously.

Blasting started at the site in 1987 to increase airspace by removal of the shale and limestone bedrock at the slope toe. Therefore, the brown native soil had been subjected to many blasts and vibrations generated by construction and excavation activities since 1987. Early in the excavation process, the blasts were significantly greater than the February and March 1996 blasts and were also closer to the slope toe (Strachan 1998). As the FS decreased due to strain incompatibility and overbuilding of the slope, the blast-induced shear displacement

increased and likely caused some shear strength loss in the brown native soil.

In summary, the continuous blasting at the excavated slope toe probably reduced the shear strength of the brown native soil by causing an accumulation of localized shear displacement, as suggested by a landfill employee (Duffy 1997). The magnitude of shear displacement induced during each blast increased as the FS of the slope approached unity. A similar phenomenon was observed at San Luis Dam, where reservoir drawdowns were thought to induce shear displacement, and thus strength loss, in the foundation clay, causing a slide in the upstream slope (Stark and Duncan 1991). Therefore, the multiple block analysis suggests that the blasts before and on March 1, and certainly on March 5, contributed to the occurrence of the slope failure because the slope was already marginally stable.

Toe Excavation

Toe excavations create a large stress concentration at the base of a slope that can lead to localized shear deformations, progressive failure, and finally slope instability (Dunlop and Duncan 1970). A 3D analysis was conducted of the slope with a maximum piezometric level of 24.5 m and a brown native soil friction angle of 13.5° , assuming the deep excavation had not been constructed, as shown in Fig. 2. The brown native soil outside of the landfill was assigned a ϕ' of 23° , and the piezometric level was placed at the bottom of the brown native soil. This analysis yielded an FS of approximately 1.1. Therefore, the toe excavation removed some buttressing force, but, more importantly, daylighted the weak brown native soil and allowed the waste to descend into the excavation, which allowed the slide mass to accelerate. Clearly, the time that a toe excavation is left unbuttressed should be minimized to increase slope stability.

In summary, the main cause of the wasteslide was mobilization of a postpeak strength condition in the brown native soil. As discussed previously, the blasting adjacent to the slope accelerated the progressive failure mechanism and contributed to the occurrence of the wasteslide on March 9. If blasting had not occurred on March 1 and 5, global failure might have occurred after March 9. Since the site had exceeded capacity and waste placement was going to continue at the top of the slope for the foreseeable future, the failure would have occurred eventually.

CONCLUSIONS

The following conclusions can be discerned from this case history:

1. Cracking initially and repeatedly at the top of the slope and formation of a large slide block, graben, and nearly vertical scarp indicate that the failure was translational along a weak foundation layer. In general, the reappearance of cracking in the same location in a short period of time is more likely an indication of MSW slope instability than settlement, especially if the cracks are occurring at the top of a slope and/or there is evidence of both horizontal and vertical displacement. MSW settlement cracks usually will not reappear in the same location in a short period of time because the biological and mechanical processes that cause the settlement require time.
2. A 3D stability analysis was used to account for the 3D end effects and complex geometry of the scarp, slide mass, topography, and piezometric conditions to yield a representative back-calculated friction angle of 13.5° for the brown native soil. A 3D analysis is recommended for the back-analysis of slope failures to account for the shear resistance along the sides of a slide mass and complex geometry.
3. Given the failure mode and uncertainty in the leachate level and bedrock piezometric condition, the failure cannot be explained using the drained fully softened friction angle (23°) measured or the drained residual friction angle (10°) estimated for the brown native soil. As a result, it appears that the brown native soil underlying the waste mobilized a postpeak shear strength. A postpeak shear strength may have been mobilized due to a number of factors, including soil formation, deposition, softening, and prior downhill transport; strain incompatibility between the MSW and native soil and progressive failure; time-dependent lateral deformation of the waste; blasting in the adjacent excavation; stress concentrations caused by a toe excavation; and waste placement activities.
4. Strain incompatibility between the MSW and brown native soil and progressive failure probably occurred at this site and facilitated the development of a postpeak strength in the native soil. The 3D back-analysis shows that only a small percentage of the peak MSW strength was mobilized at the time of failure. As a result, strain-compatible values of shear strength should be used for the MSW and underlying foundation soil or geosynthetic interfaces. The greater the difference between the stress-strain characteristics of the MSW and the foundation soil or geosynthetic interfaces, the smaller the percentage of strength mobilized in the MSW and underlying material(s).
5. The multiple block analysis suggests that blasting-induced shear displacements in the brown native soil, albeit over a limited distance at a given time, adversely affected the soil shear strength and contributed to mobilization of a postpeak shear strength. It is anticipated that blasting near the marginally stable slope also contributed to the occurrence of the slope failure on March 9, 1996. Deformation analyses should be conducted to investigate the possibility of shear deformation and strength loss prior to blasting. If blasting is planned at a site, the factor of safety should be large enough to minimize the accumulation of shear displacement in the underlying materials.
6. Construction of the lateral expansion was delayed for a number of reasons, including leachate and surface water flowing into the excavation. Leachate collection and re-

moval systems should be installed at the toe of and possibly within existing landfill slopes to promote leachate removal or drainage, increase slope stability, and facilitate construction of lateral expansions, especially if an excavation will be constructed at the slope toe.

7. The slope that failed was an interim slope and thus was not required to be evaluated. It is recommended that interim slopes be evaluated, even though it may not be required by regulations, to ensure slope stability during construction. This is especially relevant when the toe is excavated or daylighted, waste placement is occurring at the top of the slope, and/or blasting is occurring near the slope. If there are construction delays, waste may need to be diverted and blasting discontinued to ensure that the slope is not overbuilt or destabilized, respectively.

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