Municipal Solid Waste Slope Failure. I: Waste and Foundation Soil Properties

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Abstract: This paper describes a slope failure in a municipal solid waste landfill, with lateral and vertical displacements of up to 275 and 61 m, respectively. The wasteslide involved approximately 1.2 million m^3 of waste, making it the largest landfill slope failure to occur in the United States. Failure developed through the weak native soil underlying the waste. The analyses and related studies conducted to determine the cause of the failure are the subject of this and a companion paper by Stark et al. (2000). To facilitate the analyses, this paper investigates shear strength of municipal solid waste using field and laboratory test results and back-analysis of failed waste slopes. It also presents details of a geological study and laboratory testing program undertaken to quantify the mobilized shear strength of the weak native soil.

INTRODUCTION

On March 9, 1996, the largest slope failure in a United States municipal solid waste (MSW) landfill, based on volume of waste involved, occurred (Fig. 1), and it provides the industry with some lessons for the operation, expansion, and stability of existing landfill slopes. The slide involved approximately 1.2 million m^3 of waste, making it the largest waste slope failure by a factor of approximately two. The largest previous waste slope failure involved 500,000 m^3 in Maine (Reynolds 1991). This paper describes the landfill site and shear behavior of the involved materials.

The MSW landfill is located approximately 15.3 km northwest of Cincinnati, Ohio. The facility was permitted for 546,500 m^2 of waste placement and encompasses a total of 1,765,000 m^2 of contiguous property. At the time of the failure, the landfill was the largest solid waste facility in the State of Ohio, based on waste receipts, and it accepted an average of 1.2×10^9 kg of residential, commercial, and industrial solid wastes per year. In summary, the landfill handled approximately 12% of the total amount of solid waste placed annually in Ohio landfills.

Disposal at this site began around 1945 as part of a swine farm. The landfilling operation initially consisted of pushing waste over the edge of an existing ravine. It is important to note that the native soils on the bottom and sides of the ravine were not excavated prior to solid waste placement. It will be shown subsequently that failure occurred through the weak native soil underlying the waste. At present, the native soil is being excavated and used for other purposes, such as a compacted clay liner (CCL).

Ohio Environmental Protection Agency regulations promulgated in 1990 require that existing solid waste landfills be updated to include the best available technology design components. These features include a composite liner, consisting of a 1.5 m thick CCL, a geomembrane, a leachate collection and removal system, a ground-water monitoring system, and new siting criteria. Landfill areas that had been partially filled prior to the 1990 requirements were permitted to continue operations under certain conditions.

In February 1994, the landfill owner/operator was granted a permit for a 486,000 m^2 lateral expansion, which involved creating a large excavation adjacent to the north slope of the existing landfill with a maximum depth of 45 m and installing a composite liner system in the expansion area. At the time of failure, the depth of the excavation near the north slope toe was 30 to 35 m (Plate 1). In addition, a 2.5 to 6.0 m high, nearly vertical excavation was constructed through the MSW and brown native soil at the toe of the existing slope (Plate 1) in September 1995 to create an access road and to allow the composite liner system to be anchored near the existing landfill. Prior to this excavation, the slope toe was adjacent to the deep excavation, and the brown native soil was daylighted directly into the excavation. A seepage collection trench was constructed about 9 to 12 m below the access road to collect continual leachate exiting the landfill and/or seepage from the intact or weathered bedrock. On March 9, 1996, the existing north slope of the existing landfill slid into and essentially filled this deep excavation. Fig. 2 is a plan view of the site after the failure showing the extent of the lateral expansion and the geometry of the waste slide. The aerial survey used to generate the topography was made on March 10, 1996, one day after the slide.

GEOLOGIC SETTING

The Cincinnati area is a rolling, gently sloping upland that has been dissected in a dendritic pattern by ancient drainage systems (Ford 1967). Many of the tributaries occupy broad terraced valleys, and steep hillsides line the valleys (Fleming and Johnson 1994). The landfill site has experienced three and possibly four continental ice advances (Ford 1967). Glacial deposits cover most of the upland areas and form terraces along the Ohio River and its tributaries. Outwash and alluvium occupy the valley floors, while soil, locally derived from bedrock and/or glacial deposits, covers the bedrock on most hillsides (Baum and Johnson 1996).

The native soil underlying the MSW was primarily derived from the gray shale and limestone of the Grant Lake Formation (Schumacher et al. 1991). The slightly dipping (1–2 m/km) bedrock is of Ordovician age (425–500 million years). Bedrock formations exposed at the facility are the Kope, Fairview, and Grant Lake, in ascending order (Ford 1967). All three of these formations consist of calcareous shale, with varying amounts of interbedded limestone. The shales are not tightly cemented and slake readily to their constituent grains (Fleming 1975; Fleming and Johnson 1994). The native soil

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FIG. 1. Aerial View of Wasteslide on March 16, 1996 (Seven Days after Wasteslide)

was probably derived from the Corryville Member [gray shale (≥60%) and gray limestone, at elevations +227 to +240 m MSL] and the Mount Auburn Member [gray shale (≥60%) and gray limestone, at elevations +240 to +250 m mean sea level] of the Grant Lake Formation (Ford 1967; Schumacher et al. 1991). The brown native soil occurs under the MSW at elevations between +220 to +270 m. The Grant Lake Formation usually consists of 50 to 65% shale and the remainder limestone. The limestone beds are 50 to 100 mm thick, and in some sections the limestone beds are 75 to 300 mm apart, while in others they are 50 to 75 mm apart.

The brown native soil at the landfill site consists variably of colluvial and residual soils. At this site, the residual soil is similar to the colluvium, that is, a heterogeneous mixture of fine-grained soil with or without rock fragments derived from the local bedrock, except that it has not been transported. At this site, the residual soil is usually mottled brown, exhibits a laminar structure, and usually contains rock fragments. The colluvial soil, if present, is brown in color due to advanced weathering or decomposition and may contain random rock fragments. The colluvial soil has been transported and thus may exhibit a structure that reflects mechanical mixing due to downslope movement. However, the mechanical mixing can be limited to a thin zone, that is, the zone over which shear displacement occurred, and the overlying soil may exhibit a laminar structure. As a result, in some instances it may be difficult to distinguish between colluvial and residual soils (Terzaghi et al. 1996). For the purpose of this paper, the overburden soil at the site (colluvial and/or residual soil) will be referred to as the brown native soil.

MATERIALS INVOLVED IN SLIDE

A subsurface investigation was initiated by the owner/operator 54 days after the wasteslide to help determine the cause of sliding and to estimate appropriate shear strength parameters for design of the reconstructed slope. The subsurface investigation consisted of 13 borings in the slide area. Seven of these borings were drilled in the existing landfill area, and six were drilled through the slide mass in the lateral expansion area. At some of these locations, two borings were drilled to install a slope inclinometer and a piezometer. Three of the borings were drilled outside of the slide area shown in Fig. 2. As a result, only eight boring locations are shown in Fig. 2. Cross section A–A′in Fig. 3 shows the location of borings B, C, D, and G, which were important in determining the location of the failure surface. The other 10 borings also were used to derive the cross section but for clarity purposes are not included in Fig. 3. The results of the borings, field observations, and photographs indicate that the thickness of the brown native soil prior to the failure ranged from 2 to 5 m.

Samples of the weak, saturated brown native soil (Fig. 4) overlying the gray shale and limestone bedrock were obtained in Boring G. The weak, saturated brown native soil shown in Fig. 4 was obtained from Boring G at a depth of 24.3 to 24.5 m, using a 75 mm inside diameter (I.D.) split-spoon sampler. Boring G is located just in front of the graben area, and thus little translational movement probably occurred at this location, which helps to explain the lack of a well-defined failure surface. Above the weak layer, the brown native soil is much stiffer, and below it are limestone rock fragments.

Gray shale was found underlying the MSW in Boring C, and the brown native soil was not present. It is believed that the absence of the brown native soil in Boring C was caused by the slide mass scraping off the native soil and carrying it into the excavation during the slide. Brown native soil was found in Boring D, even though the native soil had been removed in the early stages of the deep excavation. In addition, the brown native soil obtained from Borings D and G will be

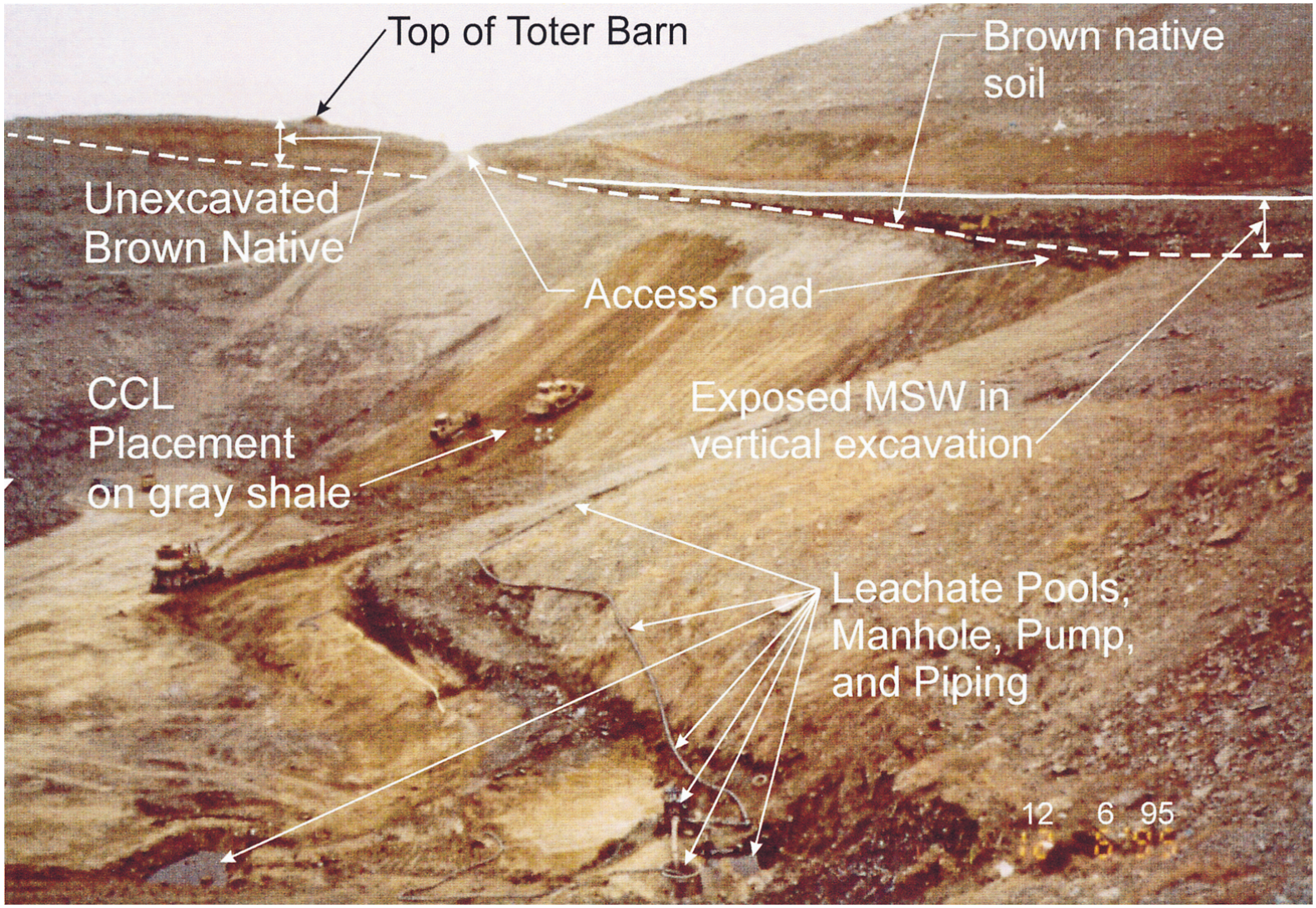

PLATE 1. Construction Activities in Lateral Expansion and Toe of Existing Slope on December 6, 1995 (Note: Equipment for Scale)

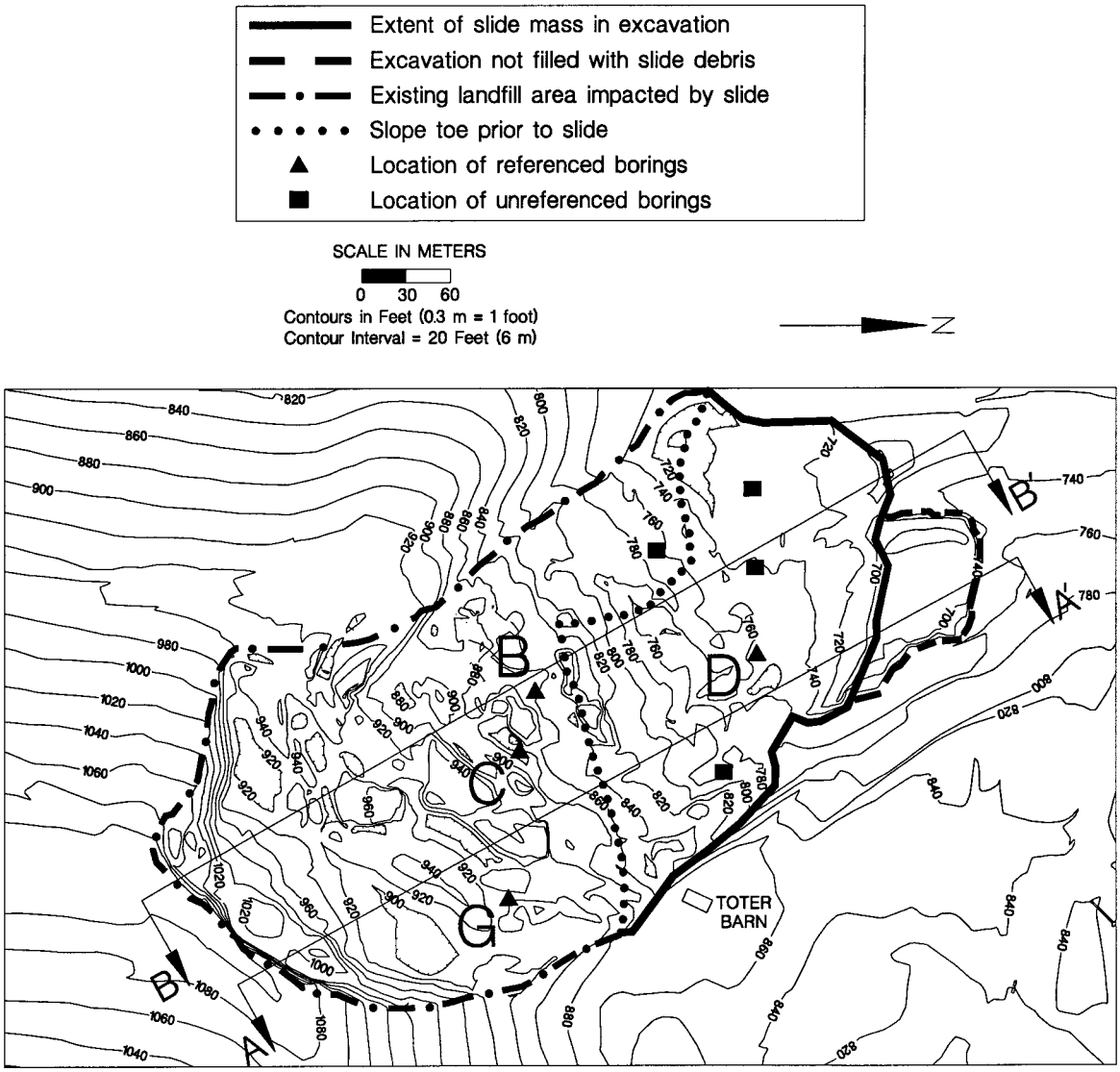

FIG. 2. Plan View Showing Lateral Expansion and Extent of Slide Mass (Contour Lines after Failure)

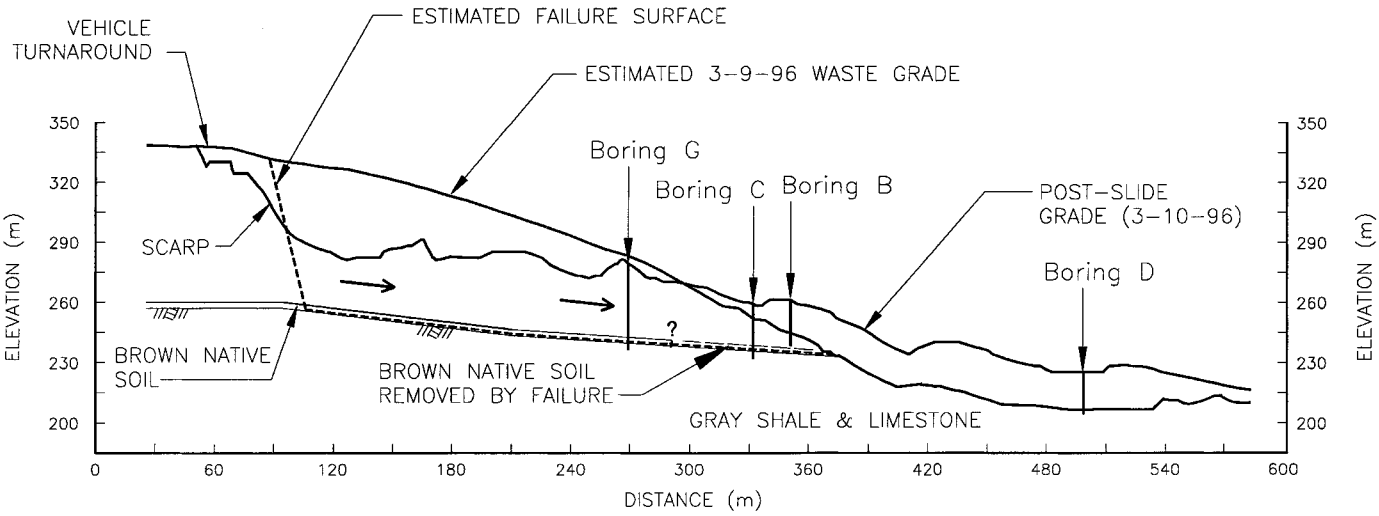

FIG. 3. Slope Cross Section A–A′ Showing Slide Mass Geometry and Boring Locations

shown to have similar index properties and shear strength characteristics.

Based on slope inclinometer data (Fig. 8 in Stark et al. 2000), photographs, and field observations, the failure surface is estimated to have passed through the solid waste at a steep inclination to the underlying weak, saturated brown native soil (Fig. 3). The failure surface continued in the brown native soil until it daylighted at the vertical face of the excavation at the toe of the existing slope. As a result, the shear behavior of the MSW and brown native soil is investigated herein.

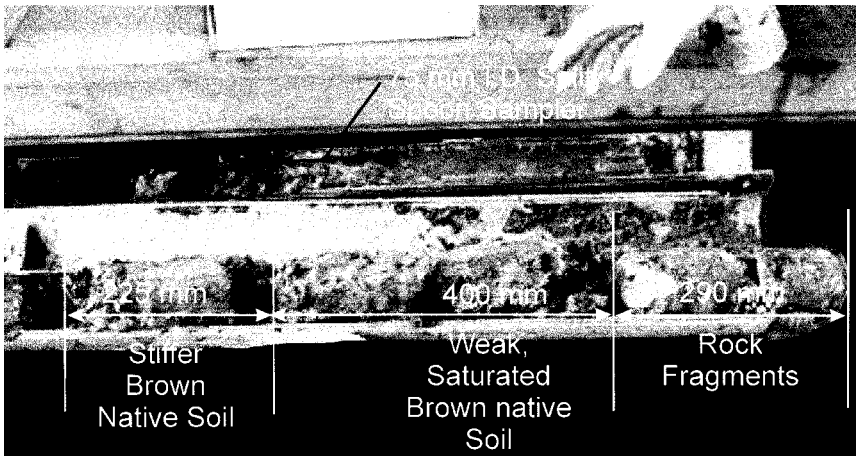

FIG. 4. Weak, Saturated Brown Native Soil from Boring G at Depth of 24.3 to 24.5 m

SHEAR STRENGTH OF MUNICIPAL SOLID WASTE

Published test data and back-analysis of field case histories were reviewed and analyzed to select reasonable shear strength parameters for the MSW to be used in the stability analysis described by Stark et al. (2000). Determination of MSW shear strength properties is difficult because of the inconsistent composition of landfill material; the difficulty in sampling and testing; time-dependent properties; and strain incompatibility between the MSW and underlying material(s). Different methods, including direct measurements (laboratory and field testing) and back-calculation using load tests and case histories, have been used to estimate MSW shear strength. Laboratory measurements have been performed using small-scale triaxial compression tests (Stoll 1971; Cooper Engineers 1986; Earth Technology 1988; Gabr and Valero 1995); large-scale triaxial compression tests (Jessberger and Kockel 1991); small-scale unconfined compression tests (Fang et al. 1997); small direct shear tests (*Puente Hills* 1984; Siegel et al. 1990; Gabr and Valero 1995); and large direct shear tests (Landva and Clark 1990; Edincliler et al. 1996; GeoSyntec 1996).

Field measurements have been made using large direct shear tests (Richardson and Reynolds 1991; Houston et al. 1995; Withiam et al. 1995); vane shear tests (Earth Technology 1988); standard penetration tests (Earth Technology 1988); and cone penetration tests (Hinkle 1990; Oakley 1990; Siegel et al. 1990; Jessberger and Kockel 1991). Back-calculation of the shear strength of MSW has also been made using plate load tests (Eliassen 1942; Pagotto and Rimoldi 1987; Howland and Landva 1992); test fills or embankments (''Slope stability'' 1975; Oweis et al. 1985); unfailed waste slopes (Kavazanjian et al. 1995); and failed waste slopes (Oweis et al. 1985; Dvirnoff and Munion 1986; Erdogan et al. 1986; Reynolds 1991; Howland and Landva 1992; Chilton et al. 1994).

In Fig. 5, only data from large-scale direct shear tests and back-calculation of failed waste slopes are presented. The other laboratory data were excluded due to the use either of processed [e.g., Jessberger and Kockel (1991)] or simulated [e.g., *Puente Hills* (1984)] waste in the testing program and/or a small specimen size when compared to the maximum particle size of the MSW. Vane shear test results were not included because fairly homogenous material is required to obtain a meaningful result from this test (Jessberger and Kockel 1991). Penetration test results also were not included because no meaningful correlation between MSW shear strength and penetration resistance has been developed (Mitchell and Mitchell 1992). Back-calculated shear strength from plate load tests and test fills was not considered because failure planes were not located, that is, were assumed, which can result in an inaccurate estimate of the shear strength. Back-calculated shear strength parameters from landfill slopes that had not failed also were not included because the actual factor of safety is not known, and the back-calculated shear strength is sensitive to the assumed factor of safety.

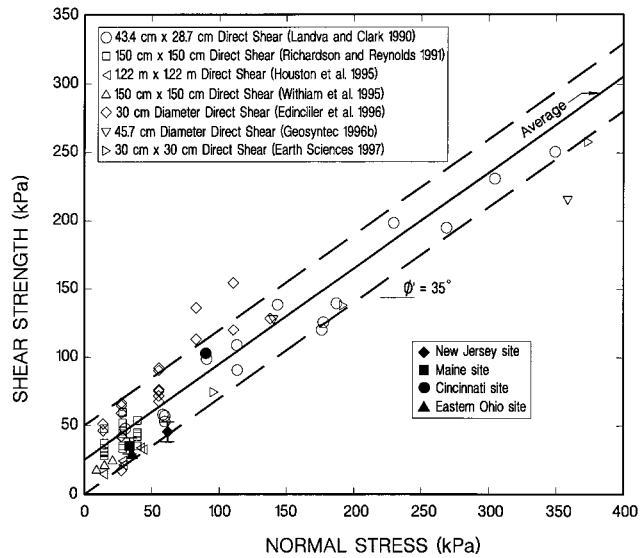

FIG. 5. Summary of Measured and Back-Calculated Data on Shear Strength of Municipal Solid Waste

The Mohr-Coulomb strength criterion was assumed applicable because the shear resistance of MSW increased with increasing normal stress (Fig. 5). Regression analysis of the data points shown in Fig. 5 suggests that the shear strength of MSW can be defined by a narrow band with an effective stress friction angle, ϕ', of approximately 35°, and cohesion, c', that ranges from 0 to 50 kPa. Based on the literature study and back-calculation of field case histories (discussed subsequently), an average c' and ϕ' of 25 kPa and 35°, respectively, may be appropriate for the design of municipal solid waste containment facilities. This combination is slightly higher than combinations published by others, for example, Singh and Murphy (1990), Kavazanjian et al. (1995), and Houston et al. (1995). The lower and upper bounds shown in Fig. 5 could represent the shear strength of MSW that contains more soil, sludge, and/or other soil-like materials (e.g., Eastern Ohio site) and plastics (e.g., Cincinnati site), respectively.

The waste shear strength parameters shown in Fig. 5 are high and indicate a material with high shear strength. The mechanisms that yield high shear strength in MSW are not clear, but the interconnection of the plastics and other materials is probably a contributing factor. This case history, as

well as others, continues to suggest that typical MSW is a strong material and that testing and stability evaluations should focus on the other involved materials, for example, geosynthetics or foundation soils. This suggestion is supported by the fact that nearly vertical cuts and scarps in landfills, including the Cincinnati site, have been observed to remain stable for months to years.

Back-Calculated Shear Strength from Failed Waste Slopes

To verify the use of the MSW shear strength band shown in Fig. 5, three landfill slope failures were back-analyzed. Two of these landfill failures involve a rotational failure through a soft saturated soil underlying the waste and occurred in Maine (Reynolds 1991; Richardson and Reynolds 1991; and Chilton et al. 1994) and New Jersey (Oweis et al. 1985; Dvirnoff and Munion 1986; and Erdogan et al. 1986). The third case history involves a translational landfill failure along a geosynthetic interface in eastern Ohio (Stark et al. 1998). Nine other landfill slope failures (Stark 1999) were analyzed but not included in Fig. 5 because of *significant* uncertainties in the analysis, such as incomplete geometry, piezometric/leachate pressure, shear strength, and/or subsurface information or stratigraphy. The back-analyses of the three case histories mentioned above are presented in Fig. 5 using the back-calculated shear strength and average effective normal stress acting on the failure surface through the waste. These three case histories are in agreement with the proposed trend lines.

Estimation of Waste Shear Strength Parameters for Cincinnati Site

In accordance with Duncan and Stark (1992), the best estimate of ϕ' was used to estimate the corresponding value of c' for the Cincinnati site. This approach appears applicable to internally reinforced materials, such as MSW, because reinforced soils exhibit a higher c' and the same ϕ' as the unreinforced soil (Gray and Ohashi 1983; Maher and Gray 1990). In addition, several laboratory testing programs on MSW have shown that c' is influenced more than ϕ' by the removal of plastics and other materials (Edincliler et al. 1996; Benson and Khire 1994; Foose et al. 1996). As a result, ϕ' of the MSW at the Cincinnati site was estimated to be 35° (Fig. 5) and then used to estimate the corresponding cohesion from the 30 m high, nearly vertical and unbuttressed portion of the scarp that was stable for approximately eight to nine months until it was reduced by the owner/operator (Fig. 1). Stability analyses, assuming circular and log-spiral failure surfaces from the toe of the vertical scarp through the MSW, were conducted using an average exposed vertical height of the scarp (H) of 30 m, a unit weight (γ) of the waste of 10.2 kN/m^3, and a ϕ' for the waste of 35°. The resulting values of c' ranged from 38 to 42 kPa. The scarp height was varied from 30 to 60 m in a sensitivity analysis, and the resulting change in cohesion (40 to 80 kPa, respectively) only yielded a decrease in the back-calculated friction angle for the brown native soil of 1 to 2°. Since no fluid was observed exiting the exposed portion of the scarp, fluid pressures were not included in these analyses. Based on these analyses, site-specific values of c' and ϕ' of 40 kPa and 35°, respectively, were used in the stability analyses of the slope failure in the companion paper (Stark et al. 2000). The back-calculated shear strength for the Cincinnati site is also shown in Fig. 5 and plots slightly above the average trend line.

Strain Incompatibility with MSW

The companion paper (Stark et al. 2000) suggests that strain incompatibility between the MSW and underlying brown native soil can result in mobilization of shear strengths in the MSW and brown native soil that are less than the peak values. A similar conclusion was reached for geosynthetic interfaces (Byrne 1994; Stark and Poeppel 1994) and foundation soils (Mitchell et al. 1995) because of their strain incompatibility with MSW. Therefore, it is important to use MSW shear strength parameters that are strain compatible with the adjacent material(s) in stability analyses. Fig. 6 shows that the peak shear strength of the brown native soil (23 kPa) is mobilized at a torsional ring shear displacement of 2 to 3 mm and then undergoes a postpeak strength loss. The peak shear strength of the waste (89 kPa) is achieved at a direct shear displacement of about 40 mm, and it remains approximately constant at that value for larger displacements. These peak shear stresses correspond to normal stresses of 50 and 55 kPa for the brown native soil and MSW, respectively. As a result, it is conceivable that failure might occur first in the native soil when only a fraction of the MSW peak strength is mobilized. After failure occurs in the native soil, the peak strength of the MSW would be mobilized at a time when the shear strength of the native soil had declined to a value significantly below the peak. Therefore, using a peak strength for both the soil and MSW would be unconservative. Strain incompatibility is most likely when the stress-strain characteristics of the embankment and foundation soil differ markedly, and it results in the percentage of peak strength mobilized in the MSW being smaller than the percentage mobilized in the native soil. Chirapuntu and Duncan (1976) used nonlinear finite-element analyses to 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. This might explain the difference between the recommended MSW strength parameters, c' = 25 kPa and ϕ' = 35°, and the fact that vertical waste slopes have remained stable for months to years.

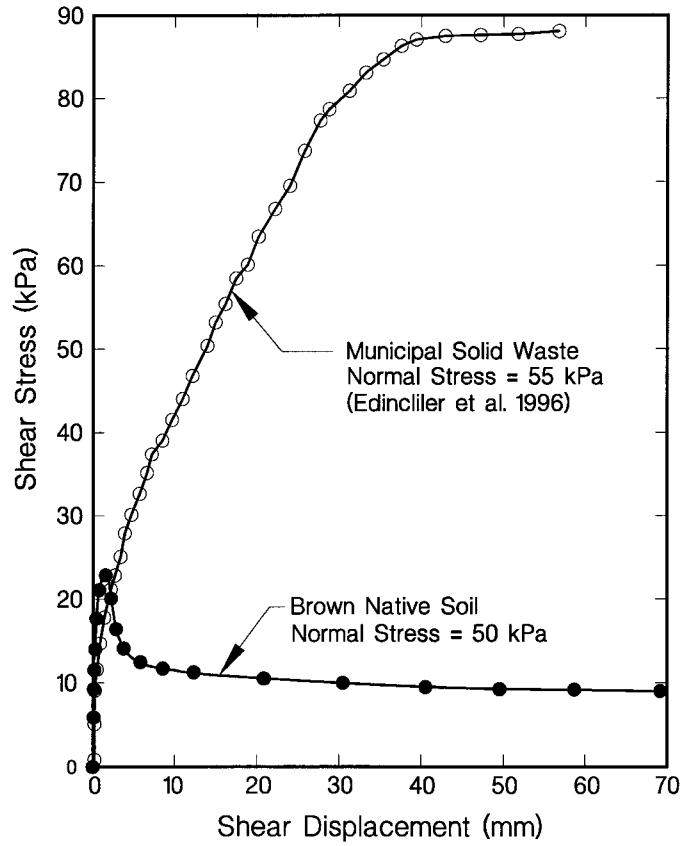

FIG. 6. Shear Stress-Displacement Relationships for Municipal Solid Waste and Brown Native Soil

Since vertical MSW slopes have been stable for months to years and the recommended values of $c' = 25$ kPa and $\phi' = 35°$ were verified using field case histories, it may be assumed that only a percentage of the MSW peak strength is mobilized during a slope failure. This may appear contradictory, with the peak strengths from laboratory direct shear tests on MSW plotting in the same range as the back-calculated values in Fig. 5. However, this is attributed to sample sieving/sorting and specimen reconstitution processes removing some of the in-situ interaction/interlocking between the various pieces of waste and yielding a laboratory peak strength that is lower than the in-situ value.

In summary, strain compatibility may preclude mobilization of the peak shear strength of the MSW and soil or geosynthetics underlying the MSW. The waste shear strength parameters recommended from Fig. 5 appear to represent the MSW shear strength mobilized during failure and thus incorporate the effect of strain compatibility with underlying soil and/or geosynthetic materials. Strain incompatibility can result in the mobilized strength of the underlying material being up to 20% lower than the peak value.

BEHAVIOR OF COLLUVIAL SLOPES

Another major uncertainty in the slope stability analyses used to investigate the cause of the Cincinnati failure was the shear behavior of the brown native soil underlying the MSW. Extensive research has been conducted on native soil slopes in the Cincinnati area due to the widespread landsliding that has occurred, for example, Baum (1983, 1994), Behringer and Shakoor (1992), Fleming (1983), and Gokce (1981, 1992). Cincinnati and vicinity has one of the highest annual per capita costs of damage due to landsliding in the United States (Fleming and Taylor 1980). Most of these landslides occur in slopes underlain by native soil. The native soil deposits are usually less than 10 m thick and are thinnest near the crest and thickest near the toe of each slope. Fleming and Johnson (1994) conclude that two distinct types of landslides occur in the native soil in the Cincinnati area. Landslides in thin soil deposits, that is, less than 2 m, typically occur in the spring after the ground has thawed and before the vegetation has fully blossomed. Their movements are associated with rainfall. The slides occur because the water levels in thin soil deposits rise during a rain event, which reduces the effective stress acting on the sliding surface. The thin overburden imposes a low normal stress on the sliding surface, so a small change in water level can reduce the effective stress sufficiently to cause instability. Sliding usually occurs at the weathered/unweathered bedrock interface because the infiltration builds up on the lower-permeability unweathered bedrock, and the permeability of the overlying weathered material usually increases due to freeze-thaw cycles to allow the infiltration (Fleming 1975). This type of sliding was observed at the site on a small natural slope adjacent to the landfill.

Landslides in thick native soil deposits, that is, more than 2 m, can occur at any time. These deeper failures are often in response to a disturbance, such as filling at the top and/or excavation at the toe. Their movement is less related to increases in water level because the thick overburden induces a larger normal stress on the sliding surface, so that a small change in water level does not lower the effective stress enough to result in instability. The brown native soil under the waste is approximately 2 to 5 m thick and is overlain by a waste depth that exceeds 100 m near the center of the Cincinnati landfill. As a result, the effective stress along the potential sliding surface ranges from 60 to 746 kPa, and thus the slope is probably not adversely impacted by small increases, for example, 2 m (20 kPa), in water level. Since the Cincinnati failure involved a long translational failure surface under a thick slide mass (Fig. 5), a large change in water/leachate level would have had to occur over a significant portion of the failure surface to influence stability. The water/leachate level was not measured prior to the failure, so Stark et al. (2000) used observations and measurements after the failure to estimate the level.

Weathering of shale can lead to removal of most, if not all, of the cementation/induration effect(s) of the parent rock. Thus, the native soil will exhibit a shear strength that is much lower than that of the parent rock and possibly equal to the fully softened shear strength (Skempton and Hutchinson 1969), which corresponds to the peak strength of a normally consolidated specimen (Skempton 1985). In addition, significant mineralogical and texture changes can occur due to the physical and chemical weathering process (Jackson and Sherman 1953; Leith and Craig 1965; Morgenstern 1969), which can result in a higher plasticity than that of the parent rock. Of course, the parent rock is instrumental in determining the characteristics of the resulting weathered material. For example, some shale can weather to a high-plasticity clayey soil and thus exhibit a low shear strength. Fleming and Johnson (1994) have readily documented the development of high-plasticity soil from shale weathering in the Cincinnati area. Conversely, nonshale materials such as granite can weather, exhibit a high shear strength, and serve as an excellent building material, for example, decomposed granite in southern California. Of course, this paper is focused on the brown native soil derived from shale at the Cincinnati site.

Shear Strength Mobilized in Colluvial Slopes

For colluvial soils, the downslope movement or transportation causes shear displacement in the material, resulting in a shear strength that is less than the peak value [e.g., Skempton and Hutchinson (1969)]. This downslope movement can result in a weak layer or zone in the colluvium. These layers or zones can combine to create a continuous shear surface or shear zone that results in a postpeak shear strength being mobilized along that surface or zone. A literature review yielded a number of references [e.g., D'Appolonia et al. (1967); Skempton and Hutchinson (1969); DuMontelle et al. (1971); Hamel and Flint (1972); Fleming (1975, 1983); Wu and Sangrey (1978); Wu et al. (1987); Lambe and Riad (1991); Fleming and Johnson (1994); Turner (1996)] that conclude colluvial soil derived primarily from shale often mobilizes a shear strength that is less than the peak value, and thus a postpeak or residual shear strength is recommended for the design of colluvial slopes. Skempton and Hutchinson (1969) state that the Walton's Wood case history, which involved a colluvial slope, ''was probably the first occasion in which laboratory residual strength could be correlated with the strength of natural surfaces.''

Shale-derived colluvial and/or residual soil deposits, such as those in the Cincinnati area, often contain at least one weak, highly plastic layer of cohesive soil that is overlain and/or underlain by less plastic, and thus stronger, soil (Fleming and Johnson 1994). Fig. 4 shows a weak, saturated layer of the brown native soil from Boring G between stiffer soil and underlying rock fragments. The existence of this weak, highly plastic layer is not easily determined. D'Appolonia et al. (1967) showed that even continuous split-spoon samples were not successful in locating the weak layer in a similar colluvial deposit. Another example of the presence of a weak, high-plasticity layer in a colluvial soil is shown in Plate 2 and involves a recent waste slope failure in southeast Ohio. This failure surface was not discovered in most of the postslide borings that were conducted, even though continuous sampling methods were employed. The failure was apparently triggered by construction/excavation activities associated with the remedial measures that began at the site. Excavation into the

colluvium during remediation resulted in many failures along well-defined and continuous high-plasticity layer(s) [Plate 2(a)]. It can be seen that the weak layer extends away from the exposed failure surface almost parallel to the pipe in the foreground. Plate 2(b) shows a closeup of the weak layer exposed in Plate 2(a), and it is important to note (1) the thin and plastic nature of the failure surface/zone, and (2) the many rock fragments adjacent to but not in the failure surface. The same sorting of rock fragments from the weak layer was observed in other locations and also reported by Skempton (1985). As a result, only the layer of high-plasticity material, and none of the rock fragments, should be included in laboratory shear and index test specimens. The failure zone material in Plate 2(b) all passed the U.S. Standard Sieve No. 40.

PLATE 2. Failure Surface along High-Plasticity Layer at Ohio Site: (a) (Top) Failure Surface Overview; (b) (Bottom) Failure Surface Closeup

Being underlain and/or overlain by a hard or firm stratum can facilitate orientation of the native soil particles parallel to the direction of shear and mobilization of a postpeak shear strength during downslope colluvial movements. Similar conditions were reported by Simons (1976); Skempton (1985); and Lambe and Riad (1991) for the Carsington Dam slope failure; Otford Test Embankment failure; and Balsam Gap and Boone landslides, respectively. Hard stratum, porous stones, and an overconsolidated specimen are used to facilitate mobilization of a residual strength in laboratory shear testing (Stark and Eid 1993). At the Cincinnati site, the weak, saturated brown native soil layer (Fig. 4) is overlain by stiff brown native soil and underlain by weathered-to-intact limestone, which may have focused shear displacements in this material and facilitated mobilization of a postpeak shear strength.

Estimation of Shear Strength Parameters for Brown Native Soil

Laboratory ring shear tests were conducted, using the procedure described by Stark and Eid (1993), on samples of the brown native soil provided by the owner/operator. Remolded samples were used for the ring shear testing as suggested by Mesri and Cepeda-Diaz (1986) and because there was not enough material provided to conduct an undisturbed ring shear or direct shear test. Fig. 6 presents a typical shear stress-displacement relationship for a normally consolidated, remolded specimen of the brown native soil. The specimen exhibits a well-defined peak shear stress at a shear displacement of 2 to 4 mm because the specimen was not precut or presheared prior to shearing. The shear stress-displacement relationships were used to develop the peak failure envelope in Fig. 7 for the brown native soil from Boring G shown in Fig. 4. The peak or fully softened failure envelope corresponds to an effective stress ϕ' of approximately 23°. The fully softened ϕ' was estimated, using the index properties shown in Fig. 7 for the Boring G material and the empirical correlation proposed by Stark and Eid (1997), to be 19 to 25° for effective normal stresses ranging from 50 to 400 kPa, respectively. It can be seen that the brown native soil from Boring D yielded a slightly higher peak friction angle (24°) than that of Boring G. This could be caused by the mottled brown-gray soil of Boring

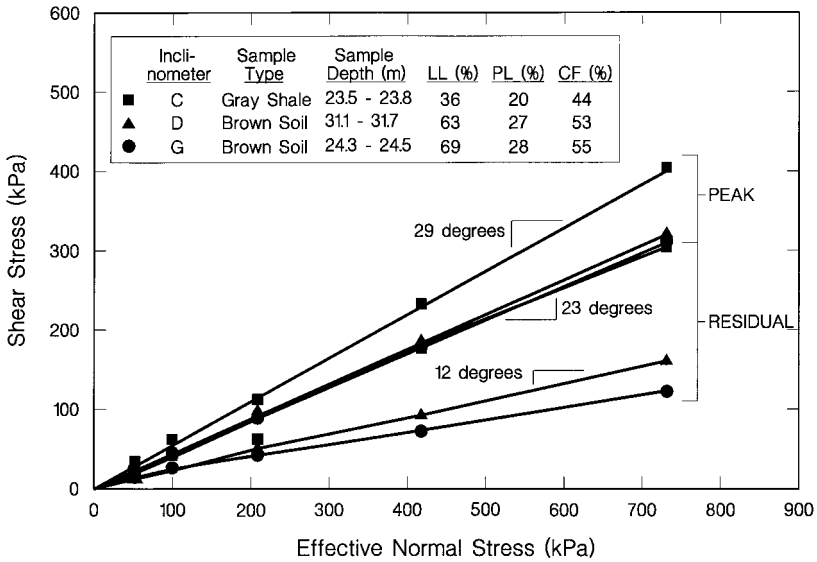

FIG. 7. Drained Peak and Residual Failure Envelopes for Brown Native Soil and Gray Shale

D being mixed with some gray shale in the slide debris that filled the deep excavation.

The fully softened friction angle for the gray shale also was estimated from ring shear tests and corresponds to 29°, which is in agreement with the correlation, that is, 27 to 30° for effective normal stress of 50 to 400 kPa, respectively, proposed by Stark and Eid (1997).

Fig. 7 also presents the residual failure envelopes for the brown native soil obtained in Borings G and D and the gray shale directly underlying the waste in Boring C. In accordance with Stark and Eid (1994), the drained residual failure envelope for the brown native soil in Boring G is stress dependent and may be approximated with a friction angle of 10° to represent the shear strength at the range of applied stresses. The empirical correlation proposed by Stark and Eid (1994), which relates liquid limit (LL), clay-size fraction (CF) (<0.002 mm), and effective normal stress to the drained residual friction angle, yields a residual friction angle of 10 to 12° for effective normal stresses of 100 to 700 kPa for the brown native soil from Boring G. It can be seen that the brown native soil from Boring D yielded a higher residual friction angle (12°) than that of Boring G. The gray shale exhibits a lower plasticity and a much higher residual friction angle (23°) than the brown native soil. These data also suggest that sliding occurred through brown native soil and not the gray shale.

Selection of Index Properties for Brown Native Soil

Table 1 presents the LL and CF for other samples of the brown native soil tested for this project. It can be seen that the LL ranges from 36 to 77%, or from low to high plasticity. The CF ranges from 22 to 60%, with the higher values of CF usually corresponding to higher values of LL. These data lead to an important discussion on shear strength evaluation for slopes involving similar native soil.

Field observations suggest that the potential failure surface will follow the weakest (i.e., most plastic) material. In addition, field case histories, such as the site shown in Plate 2, suggest that these weak layers can be present over a large percentage, or maybe all, of the failure surface. Therefore, use of the average LL and/or CF of samples obtained inside or outside the potential slide mass is not recommended to estimate the drained residual friction angle from an empirical correlation. For example, the average LL and CF from Table 1 are 57 and 41%, respectively. Using the empirical correlations presented by Stark and Eid (1994, 1997), a drained residual and fully softened friction angle of 18 to 20° and 24 to 29° for normal stresses ranging from 50 to 700 kPa, respectively, would be estimated for these average values of LL and CF. This overestimation of the measured ϕ' values in Fig. 7 primarily results from the low CF. If the subsurface investigation shows that the high-plasticity layer is not continuous along the entire failure surface, different shear strengths can be assigned to different portions of the failure surface to reflect changes in plasticity instead of using the average LL and CF. This is anticipated to provide a better representation of the shear strength along the failure surface.

If future activities, such as vertical expansion, are proposed for an existing landfill, testing of the soil directly underneath the MSW is recommended to measure or estimate the shear strength because of the natural variability that occurs in soil laterally and vertically across a site and the presence of the MSW. Differences in shear strength from underneath and outside the MSW can be caused by a number of mechanisms, including differences in the parent rock(s), chemical and physical weathering processes, soil thickness and displacement, inclination of underlying materials, strain incompatibility and progressive failure, time-dependent lateral displacement of the waste, the impact of blasting and excavation at the slope toe, waste placement techniques, and the effect(s) of leachate on the engineering properties of the underlying soil. Soil samples from other areas around the site may not accurately represent the material directly under the waste, which will ultimately control the static and seismic stability of the MSW slope.

TABLE 1. Index Properties of Brown Native Soil in Wasteslide Area

Boring number (1)	Sample depth (m) (2)	Liquid limit (%) (3)	Clay-size fraction (%) (4)
D	29.3–29.4	68	41
D	29.6–29.7	63	37
D	29.7–29.9	63	37
D	30.2–30.5	65	36
D	31.3–31.4	77	38
D[a]	31.1–31.7	63	53
D	31.6–31.7	77	38
D	31.9–32.0	67	43
D	32.3–32.4	66	32
D	32.8–32.9	66	32
G[b]	23.6–23.8	36	31
G[b]	23.8–24.1	58	41
G	24.1–24.2	58	42
G	24.3–24.5	61	54
G[a]	24.3–24.5	69	55
G[b]	24.5–24.5	59	56
G	24.6–24.7	59	60
G[b]	24.7–24.8	48	44
G[b]	24.8–24.9	46	49
G[b]	25.5–25.6	42	42
G[b]	26.5–26.7	39	22
Grab sample	1.5	42	42

[a]Sample used for ring shear testing shown in Figs. 6 and 7.
[b]Reported by GeoSyntec (1996a).

CONCLUSIONS

The following conclusions can be discerned from studying the waste and foundation soil behavior for this MSW slope failure:

1. Large-scale laboratory and field test results and back-analysis of failed waste slopes suggest that the mobilized shear strength of MSW can be represented by an effective stress friction angle of 35° and cohesion ranging from 0 to 50 kPa, with an average of 25 kPa, depending on the waste constituents. A combination of effective stress cohesion and friction angle of 40 kPa and 35°, respectively, was estimated for the waste involved in this slope failure. These shear strength parameters appear to be strain compatible with underlying soil or geosynthetics, but strain compatibility may result in the underlying material mobilizing a postpeak shear strength.
2. The interconnection of plastics and other materials probably plays a significant role in developing the high shear strength of municipal solid waste, which has allowed vertical slopes to remain stable for months to years. As a result, testing and stability evaluations should also focus on the other involved materials, such as underlying geosynthetics and/or soils.
3. The parent rock has a large influence on the engineering characteristics of the derived residual and/or colluvial soil. For example, some shale can weather to a high-plasticity soil and thus exhibit a low shear strength. However, nonshale such as granite can weather to a low-plasticity material that exhibits high shear strength and can serve as a suitable building material (e.g., decomposed granite).
4. Colluvial/residual soil deposits similar to those in the

Cincinnati area often contain at least one weak, highly plastic soil layer that can control the shear behavior and thus the stability of the deposit. Continuous sampling is recommended in shale-derived colluvial soils to prove the absence of a weak, high-plasticity layer. Assuming the weak layer is located and oriented such as to promote instability, it is recommended that the clay matrix without rock fragments be used for shear testing, or that the plasticity and clay-size fraction values from this layer be used to estimate the shear strength. If the high-plasticity layer is not continuous along the failure surface, different shear strength values can be assigned to different portions of the failure surface to reflect changes in soil plasticity.

5. A postpeak shear strength may be required to characterize the shear strength of some shale-derived colluvial soils underlying existing MSW landfills because of the effects of soil formation, softening; deposition, and prior shearing; strain incompatibility between the MSW and soil; progressive failure; time-dependent lateral deformation of the MSW; blasting, if present; waste-placement activities; and stress concentrations caused by a toe excavation.

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