DESIGN OF A FAILED LANDFILL SLOPE

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ABSTRACT: This paper describes a slope failure that occurred in a 135 acre (546 km^2) municipal solid waste landfill in which lateral displacements of up to 900 ft (275 m) and vertical settlements of up to 200 ft (61 m) were measured. The slide involved approximately 1.5 million cubic yards (1.1 million m^3) of waste making it the largest slope failure in a municipal solid waste facility in the history of the United States. Failure developed through the natural colluvial soil underlying the 350 feet (107 m) of waste that was not excavated prior to waste placement. This paper presents a chronology of the events leading to the waste slide and the studies conducted to determine the cause of the failure. In addition, the static and seismic re-design of the failed slope is discussed.

Introduction

An existing municipal solid waste landfill occupies 135 acres (546 km^2) approximately 9 miles (15.3 km) northeast of Cincinnati, Ohio. This facility is currently permitted for 234 total acres (947 km^2) of waste placement and encompasses a total of 436 acres (1765 km^2) of contiguous property. The site lies on rolling terrain between the Ohio and Great Miami River valleys. The landfill is the largest facility in the State of Ohio based on annual waste receipts and accepts an average of 5,000 tons (4.5×10^6 kg) of residential, commercial, and industrial solid wastes per day. In summary, the landfill handles approximately 12% of the total amount of Ohio solid waste disposed of in Ohio landfills and is an important piece of infrastructure in south-western Ohio.

Disposal at this site began in 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 natural colluvial soils on the bottom and sides of the ravine were not excavated prior to solid waste placement. Prior to the 1980's, problematic native soils usually were not excavated prior to waste placement. Colluvium covers most of the site and is a poorly sorted mixture of fine-grained soil and angular fragments of weathered limestone and siltstone of the

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Grant Lake Formation. Colluvium is derived from physical and chemical weathering of bedrock and thus the engineering properties of the colluvium are related to the composition of the bedrock. Colluvial deposits are often marginally stable because they are continually in a near failure state due to constant downslope movement caused by gravity. These deposits are usually less than 30 feet (9.2 m) thick and are thinnest near the crest and thickest near the toe of each slope. In the Cincinnati area, the colluvium layer is usually less than 20 feet (6.1 m) thick and is underlain by gently dipping interbedded shale and limestone of Late Ordovician age. The Cincinnati area is a rolling, gently sloping upland that has been dissected in a dendritic pattern by ancient drainage systems (Baum and Johnson 1996). 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; Fleming 1983). Most of these landslides occur in slopes underlain by colluvium.

Colluvium is an extremely common surficial deposit (Turner 1996). Therefore, this case history provides important information for the operation and expansion of the large number of landfills and structures that are founded on colluvium in Ohio, Kentucky, and Indiana.

Landfill Expansion

Ohio Environmental Protection Agency regulations promulgated under House Bill 592 in 1988, and became effective in March, 1990, requires that solid waste landfills be updated to include best available technology design components. These include a composite liner, a leachate collection and removal system, ground water monitoring and siting criteria. Landfill areas that had been partially filled prior to the update requirements were permitted to continue operations under certain conditions. For the purposes of this article these areas are referred to as "grandfathered".

In 1994 the Cincinnati landfill owner/operator was granted a permit for a 120 acre (485 km^2) lateral expansion. The lateral expansion involved creating a 140 foot (42.7 m) deep excavation at the toe of the existing landfill and installing a composite liner system in the expansion. The liner system was to consist of five feet (1.5 m) of compacted clay, 60 mils (1.5 mm) thick geomembrane, and a leachate collection and removal system.

The large and deep excavation was nearly complete at the time of failure after being open for eighteen months prior to the waste slide. However, the compacted clay of the composite liner system had been only placed on a portion of the 3H:1V slope adjacent to the "grandfathered" area (see Figure 1).

Figure 1 presents a cross-section through the failed slope prior to the waste slide. The continual collection of surface runoff in the excavation caused many construction delays. As a result, the additional capacity that would be provided by the expansion was not going to be ready for waste placement certification for quite awhile.

The delays in the construction of the lateral expansion resulted in a shortage of capacity at the site. At the time of failure, the site was overbuilt by 1,236,000 cubic yards (944,300 m^3) (Civil 1996). The slope that failed was overbuilt by approximately 957,000 cubic yards (731,100 m^3). Figure 2 presents an aerial view of the site on February 6, 1996, which is approximately one month prior to the waste slide. It can be seen that waste placement was

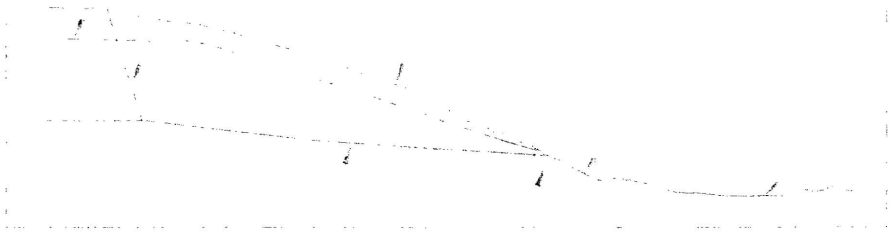

Figure 1. Slope cross-section prior to March 9, 1996 failure

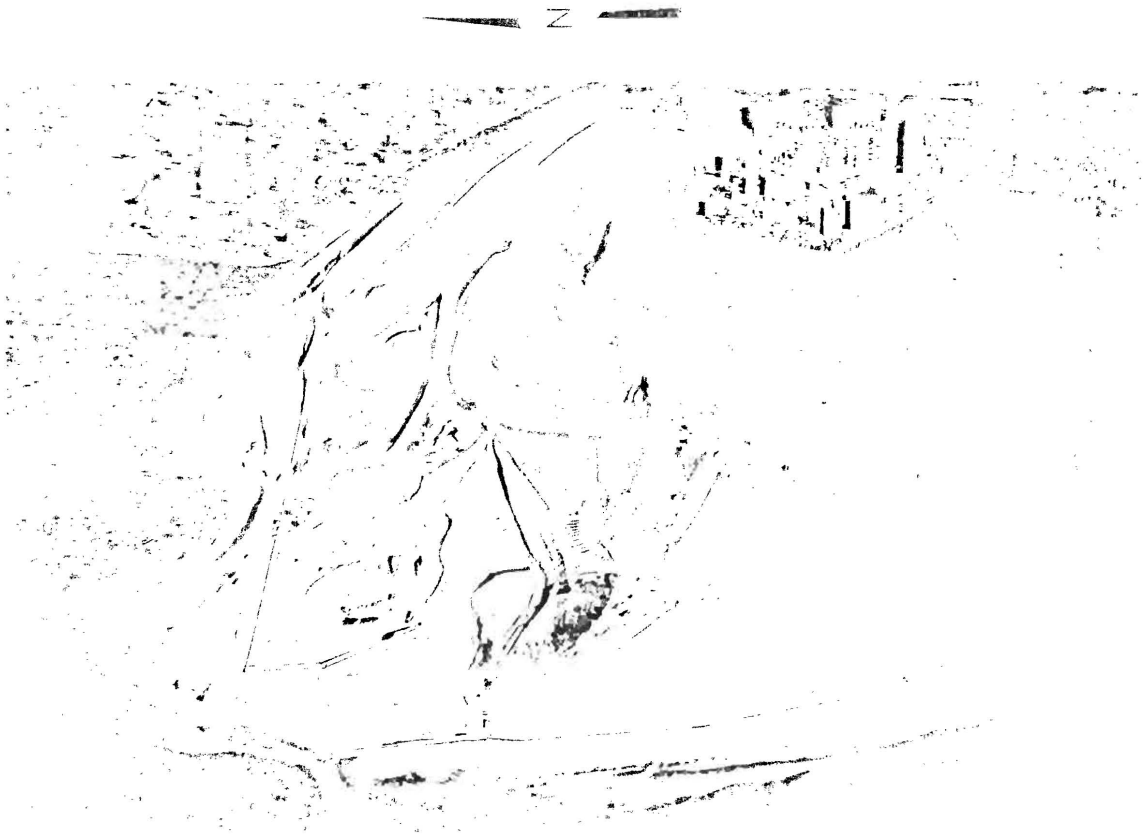

Figure 2. Aerial view of landfill on February 6, 1996, which is 32 days prior to the waste slide

occurring at the top of the landfill (note: vehicle turnaround at the center or top of the waste pile) and along an adjacent slope just below the top vehicle turnaround (note: access road and smaller vehicle turnaround to the west). At the time of the slide, waste was being placed along the adjacent slope. The 140 foot (42.7 m) deep excavation created for the lateral expansion starts to the north of the vehicle turnarounds. Waste continued to be placed at the top of the landfill until a few days prior to the slide.

As part of the 1994 expansion permit, the maximum elevation of the landfill was permitted at +1065 feet (324.8 m). The last aerial survey, one month prior to the waste slide, revealed that the maximum elevation of the landfill was approximately +1109 feet (338.2 m). Figure 1 shows the waste grades from the annual report dated December 21, 1994 and the February 6, 1996 aerial survey. It can be seen that a 30 to 60 foot (9 to 18 m) thick layer of waste was placed over the majority of the slope between December 21, 1994 and February 6, 1996. In addition to exceeding the maximum permitted elevation, the average slope inclination of the "grandfathered" area adjacent to the excavation was 2.6 Horizontal:1Vertical with some portions steeper than 1.85H:1V. A 15 to 20 foot (4.6 to 6.1 m) high nearly vertical excavation was also constructed at the toe of the slope for a composite liner anchor trench and the required access road. This excavation daylighted the weak brown colluvial layer as the slope continued to be filled (see Figure 3). This overbuilt and resulting steep slope caused driving forces that exceeded the shear resistance of the underlying brown colluvial soil.

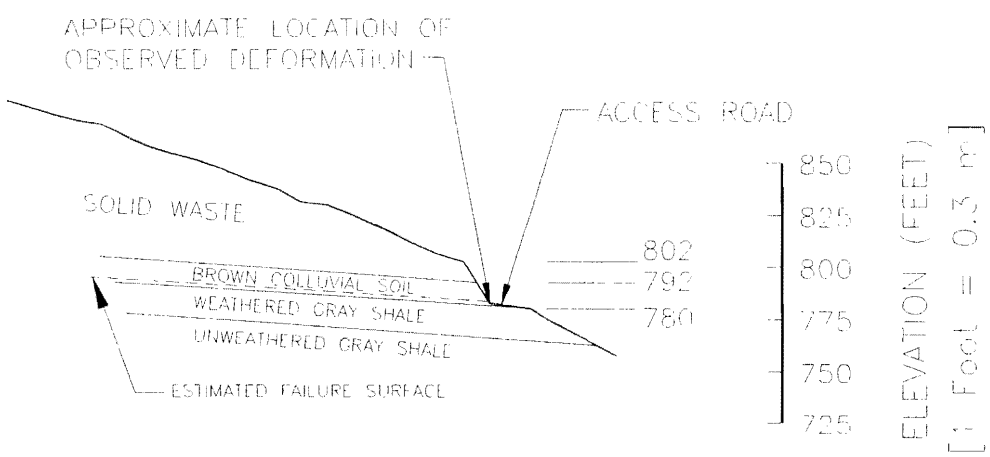

Figure 3. Detail of landfill toe prior to waste slide

Failure Mechanism

A subsurface investigation was initiated by the owner/operator 54 days after the waste slide to estimate the cause of sliding and determine appropriate shear strength parameters for design of the reconstructed slope. The subsurface investigation consisted of 13 borings in the slide area. The location of three of the borings is important in determining the location of the failure surface. Boring C corresponds to the toe of the original slope while Boring G corresponds to the graben area adjacent to the scarp (Figure 4). Samples of the brown colluvium from above

the gray shale were obtained in Boring G (see Figure 3). However, in Boring C the gray shale was found underlying the waste and the brown colluvium was not present. The absence of brown colluvium in Boring C was caused by the slide mass scraping off the colluvium and carrying it into the excavation. This was verified because the brown colluvium was found in Boring D even though the brown colluvium was removed in the early stages of the deep excavation.

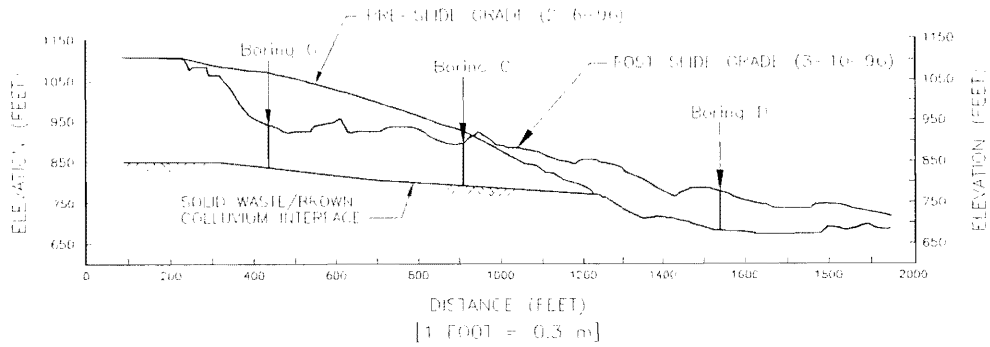

Figure 4. Slope cross-section showing slide mass geometry and boring locations

Based on field observations and the results of the subsurface investigation, the failure surface was estimated to have passed through the solid waste at a near vertical inclination to the underlying brown colluvium (see Figure 1). After reaching the colluvium, the failure surface passed through the brown colluvium, and thus along the original ground surface, and daylighted at the vertical face of the excavation at the toe of the "grandfathered" slope. This failure surface is in agreement with the observed translatory nature of the waste slide.

Cause of the Waste Slide

After the waste slide a number of failure mechanisms were investigated including: rock blasting in the adjacent excavation, seismicity, weak layers of sludge or waste, shear behavior of the brown colluvium, and overbuild. The following paragraphs investigate the feasibility of each of these mechanisms.

The most recent blasting occurred on March 5, which is one day after the initial cracking was observed at the top of the slope. This means that the slope failure had already started prior to this blast. The calculated ground velocity induced by the blast on March 5 is low and probably had no impact on the stability of the slope.

No seismicity was reported or measured within a 100 mile (170 km) radius of the site for thirty days prior to the waste slide. Therefore, seismicity was not considered to be a factor. There is a history of sludge disposal at the site but the slide was translatory and occurred along a weak layer at the bottom of the waste and not through the waste. In addition, no weak zones were encountered in the waste during the subsurface investigation. As a result, this investigation focused on the shear behavior of the brown colluvium and the overbuild that occurred in this slope.

Back Analysis of Waste Slide

To investigate the shear behavior of the brown colluvium, a three-dimensional (3D) back-analysis of the waste slide was conducted to estimate the shear strength of the brown colluvium. A 3D analysis was used to account for the complex geometry of the original ground surface and leachate level. 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 better agreement with the field or mobilized strength. Two-dimensional (2D) analyses do not account for 3D end effects. These end effects increase stability and thus 2D back-analyses yield too high, or unconservative, estimates of the field shear strength.

In summary, a 2D analysis is appropriate, and recommended, for slope design because it yields a conservative estimate of the factor of safety. It is conservative because the end effects are not included in the 2D estimate of factor of safety (Duncan 1992). A 3D analysis is recommended for back-analysis of slope failures so that the back-calculated shear strength does reflect the 3D end effects.

Bishop's simplified method of slices (Bishop 1955) was used to conduct the 3-D limit equilibrium regressive analysis. Bishop's simplified method was extended to three-dimensions by Hungr (1987) and coded in the microcomputer program CLARA 2.31 (Hungr 1988). The 3D geometry of the ravine and leachate level were described using eighteen 2D cross-sections. The waste translated down the axis of an existing ravine underlying the waste and into the excavation for the lateral expansion. The program interpolates between the 2D cross-sections to define the 3D geometry. The original ground topography was estimated from the 1955 Quadrangle Sheet (United States Geological Survey 1955) and the surface topography was estimated from the February 6, 1996 (one month prior to the waste slide) aerial survey (see Figure 2). Table 1 summarizes the input parameters used in the 3D analysis.

Back-Calculated Shear Strength of Brown Colluvium

The 3D back-analysis revealed that the shear strength of the brown colluvium at the time of failure could be characterized using an effective stress angle of internal friction of approximately 12 degrees and an effective stress cohesion of zero. Due to the type of soil deposit and the waste placement technique, it was initially thought that this low value of friction angle might correspond to a residual or minimum value. Laboratory ring shear tests were conducted at the University of Illinois on samples of the brown colluvium provided by the owner/operator using the procedure described by Stark and Eid (1993). Figure 5 presents the residual failure envelopes for the brown colluvium obtained in Borings G and D and the Gray Shale directly underlying the waste in Boring C. It can be seen that the brown colluvium from Borings D and G yielded similar drained residual failure envelopes. As reported by Stark and Eid (1994), the drained residual failure envelope for the more plastic brown colluvium in Boring G is stress dependent, and thus the failure envelope was approximated with a friction angle of 11 degrees to represent the shear strength at the range of applied stresses. The brown colluvium in Boring G is more plastic, and thus more stress dependent, than the colluvium obtained from Boring D. The United States Geological Survey (Fleming and Johnson 1994) also conducted torsional ring shear tests on the brown colluvium in the Cincinnati area and

measured a "Best Fit" residual cohesion and friction angle of 115 psf (5.5 kPa) and 12 degrees, respectively.

Table 1. Summary of input parameters for 3D back-analysis of the waste slide.

Material Type	Moist Unit Weight (pcf)	Saturated Unit Weight (pcf)	Effective Stress Shear Strength Parameters	
			Cohesion (psf/kPa)	Friction Angle (degrees)
Interim Soil Cover	120	125	0	30
Solid Waste	65	75	0	33
Brown Colluvium	120	125	0	back-calculated

The empirical correlation proposed by Stark and Eid (1994), which relates liquid limit and clay-size fraction to the drained residual friction angle, also confirmed a residual friction angle of 10 to 12 degrees for the brown colluvium used in the ring shear tests. The liquid limit, plastic limit, and clay-size fraction of the brown colluvium tested are 69, 28, and 55%, respectively (see Figure 5). The liquid limit of the brown colluvium samples ranges from 27 to 77 or from low to high plasticity. The clay-size fraction ranges from 26 to 66 with the higher values of clay-size fraction usually corresponding to higher values of liquid limit. Considering this data and the large scale impact of mobilizing a residual strength on slope stability, the following paragraph presents a discussion on the stability of colluvial slopes.

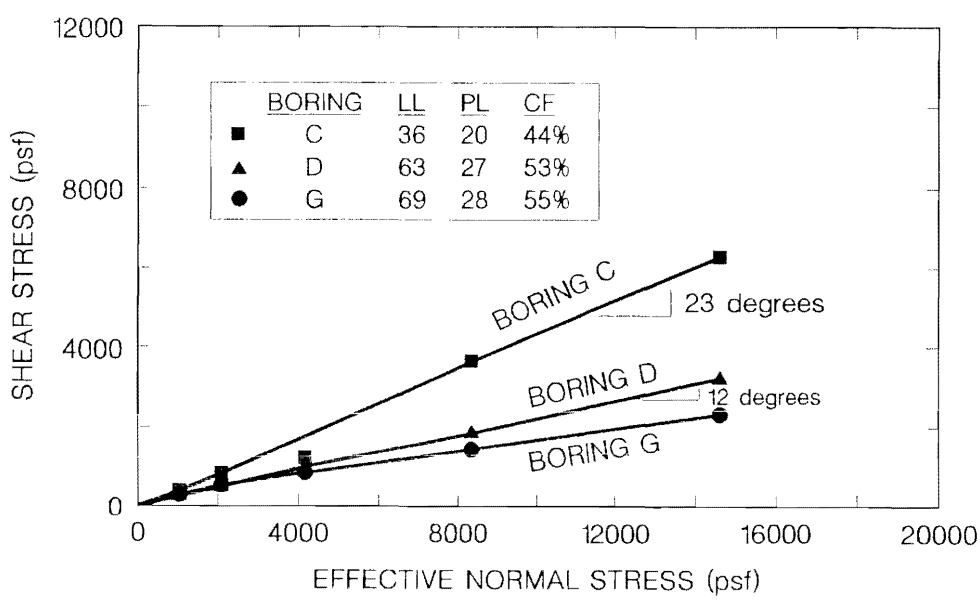

Figure 5. Drained residual failure envelopes for brown colluvium and gray shale

Surficial colluvial deposits, such as those in the Cincinnati area, usually contain at least one thin, weak, highly plastic layer of cohesive soil that is overlain and/or underlain by less plastic, and thus stronger, soil. The existence of this thin, weak, highly plastic layer is not easily determined, but may be confirmed by determining the index properties of the colluvium at small (3 to 6 inches or 76 to 152 mm) intervals (Fleming and Johnson 1994). Field observations suggest that the potential failure surface will locate the most plastic, i.e., weakest, material and not the average. Therefore, it is not recommended to use the average liquid limit or clay-size fraction to estimate the drained residual friction angle from an empirical correlation, such as presented by Stark and Eid (1994).

Of course, the colluvium directly underneath the waste should be tested to measure or estimate, using index properties, the drained residual strength because of the natural variability that occurs in colluvium across a site. This variability is caused by a number of mechanisms including differences in the parent rock(s), chemical and physical weathering processes, colluvium thickness, inclination of the ravine, and thus the rate and magnitude of downslope movement, as well as the effect(s) of leachate on the engineering properties of the underlying soil. Colluvium samples from other areas or ravines around the site may not accurately represent the material directly under the waste, which will ultimately control the static and seismic stability of the waste slope.

In summary, a 3D back-analysis, drained torsional ring shear tests, empirical correlations, and previous failures of colluvial slopes (e.g., Hamel and Flint 1972) suggest the brown colluvium underlying the solid waste mobilized a drained residual strength at the time of the waste slide.

Effect of Overbuild on Interim Slope Stability

The other major contributor to the waste slide was the 1.2×10^6 cubic yards (9.2×10^5 m^3) of overbuild. Using the back-calculated friction angle of the brown colluvium, 10 to 12 degrees, the 3D factor of safety for the slope was calculated as the maximum height of the landfill increased from elevation +1005 feet (306 m) to elevation +1109 feet (338 m). The analysis utilized the leachate level estimated at the time of failure and the excavation for the lateral expansion which daylighted the colluvium. The analysis revealed that the factor of safety decreased from approximately 1.15 at elevation +1005 feet (306 m) to about unity at elevation +1109 feet (338 m). Therefore, it was concluded that the overbuild was the triggering mechanism for the waste slide and the main cause was the mobilization of a residual shear strength in the brown colluvium.

Slope Reconstruction

The slope was reconstructed by constructing two rock berms at the toe of the slide mass and rebuilding the slope from the toe of the slide mass up towards the scarp. This involved placing new solid waste at the toe of the slope and excavating overbuilt waste from the top of the slope to achieve the final grade. The inclination of the reconstructed slope is approximately 5H:1V and satisfies static and seismic factors of safety of 1.5 and 1.3,

respectively, and a permanent seismic deformation of less than 12 inches (0.3 m). The majority of the slope reconstruction was completed by January, 1997.

One disadvantage of this slope reconstruction plan was the time that a large amount of solid waste was exposed as the slope was reconstructed from the toe to the scarp. This lead to eighteen fires at the site, three of which became uncontrolled. The fire on May 23, 1996 spread to approximately 5 acres (20.2 km^2) in size near the middle of the slope. This fire was covered by leveling the slide blocks or "wastebergs" with large excavators and bulldozers. It took five days, working twenty-four hours per day, to cover this fire. The fire on July 23, 1996 involved the vertical scarp. Access could not be gained to the scarp because of the deep graben at the base of the scarp. In addition, the scarp could not be safely reduced from the top because of the nearly vertical height of 100 to 150 feet (30 to 46 m). As a result, the local Fire Chief ordered new waste to be used to buttress the scarp to gain access for fire elimination purposes and allow emplaced waste to be pushed from the top of the scarp down to the bottom to smother the fire. This fire was extinguished after approximately eight days. The local Fire Chief also ordered placement of new waste in the remaining portions of the graben to facilitate scarp removal and thus reduce the remaining fire hazard.

Design of Reconstructed Slope

Static Stability Analyses

Slope stability analyses were conducted to investigate the stability of the reconstructed slope. A number of cross-sections were analyzed to evaluate the stability of the reconstructed slope. Figure 6 shows the typical geometry of the reconstructed slope and the two potential failure surfaces. The upper failure surface extends from the scarp through the waste to the brown colluvium (see dashed line), along the solid waste/brown colluvium interface, and out through the solid waste (see dashed line). The search process involved varying the dashed portions of the failure surface and forcing the remainder of the failure surface to follow the solid waste/brown colluvium interface. The lower failure surface extends from the proposed surface of the reconstructed slope (see dashed line) down to the bottom of the rock excavation, along the bottom of the rock excavation, and out along the side of the rock toe

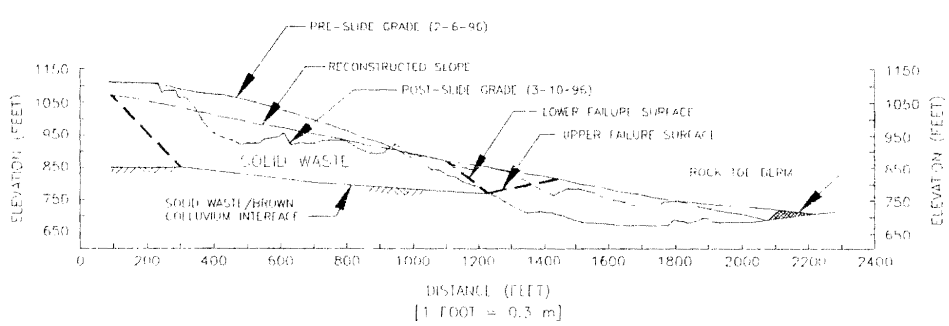

Figure 6. Slope cross-section showing reconstructed slope and critical failure surfaces

berm. This search process also involved varying the dashed portion of the failure surface and forcing the remainder of the failure surface to follow the bottom of the rock excavation and the side of the toe berm.

Since sliding had occurred through the colluvium, the reconstructed slope had to be designed using drained residual shear strength parameters for the brown colluvium. Based on ring shear tests conducted by the first author, the failure envelopes for the brown colluvium in Borings G and D (Figure 5) were used to model the colluvium shear strength along the upper and lower failure surfaces, respectively. Table 2 presents the input parameters used in the two-dimensional analysis of the reconstructed slope.

Table 2. Summary of input parameters for 2D analysis of the reconstructed slope.

Material Type	Moist Unit Weight (pcf)	Saturated Unit Weight (pcf)	Effective Stress Shear Strength Parameters	
			Normal Stress (psf)	Shear Strength (psf)
Interim Soil Cover	120	125	all	$\phi = 30^0$
Solid Waste	65	75	0 - 750 > 750	c = 500 psf $\phi = 33^0$
Rock Toe Berm	140	140	all	$\phi = 33^0$
Brown Colluvium in "Grandfathered" Area	120	120	0 1045 2090 4180 8355 14620	0 295 532 849 1444 2293
Brown Colluvium in Deep Excavation	120	120	all	$\phi_r = 12^0$
Gray Shale	130	130	all	$\phi_r = 23^0$

The microcomputer program XSTABL (Sharma 1994) was used for the limit-equilibrium stability analyses. Spencer's (1973) method, as coded in XSTABL, was used to calculate the static factors of safety and the yield accelerations that are to be used in the permanent seismic deformation analysis. The minimum static factor of safety for the upper and lower failure surfaces was calculated to be 1.82 and 1.50, respectively. As a result, the lower failure surface was found to be more critical than the upper failure surface but is still satisfactory.

Seismic Stability Analyses

The seismic stability of the reconstructed slope was evaluated using a permanent deformation analysis. The permanent deformation chart developed by Makdisi and Seed

(1978) and shown in Figure 7 was used for the analysis. Based on Ohio and Subtitle D criteria, this site is located in a seismic impact zone. A seismic impact zone is an area with a probability of greater than or equal to 10% that the maximum horizontal acceleration (MHA) in lithified material will exceed 0.1g in 250 years. The bedrock MHA was estimated to be 0.16g from the acceleration map developed by Algermissen et al. (1990). The design earthquake Richter magnitude for the site is 6.1.

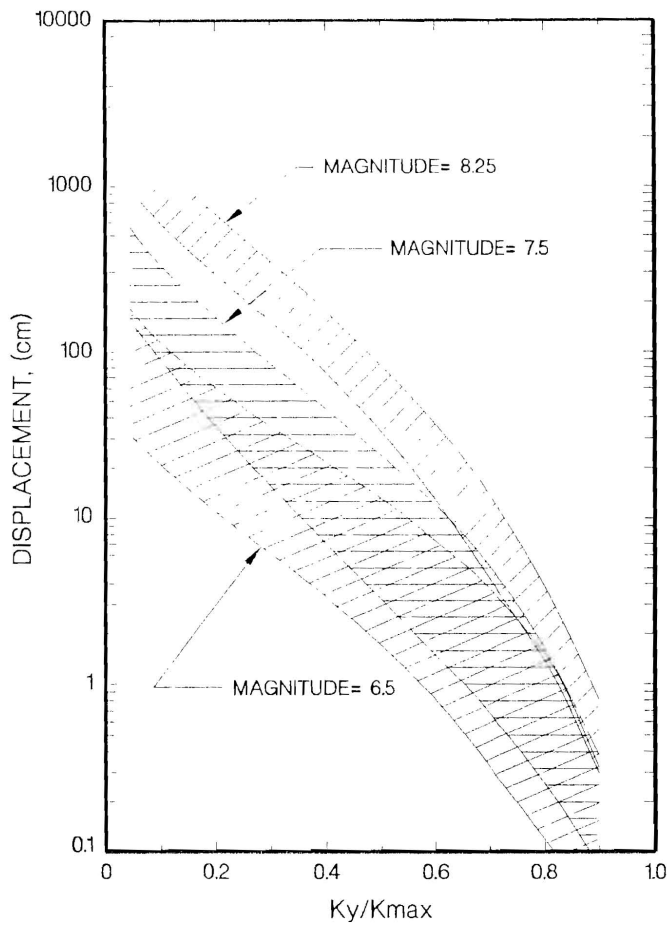

Figure 7. Permanent seismic deformation chart by Makdisi and Seed (1978)

For the permanent deformation analysis, the yield acceleration for the upper and lower failure surfaces in Figure 6 had to be estimated. This was accomplished by varying the seismic coefficient for the critical failure surface until the static factor of safety was reduced to unity. The yield acceleration for the upper and lower failure surfaces were calculated to be 0.13g and 0.08g, respectively. This is in agreement with the static analysis because the lower failure surface was more critical than the upper failure surface. The yield accelerations (Ky) were

divided by the maximum horizontal bedrock acceleration (Kmax) of 0.16g to obtain ratios of Ky/Kmax. The ratio of Ky/Kmax for the upper and lower failure surfaces was calculated to be 0.81 and 0.5, respectively. Using these ratios and the permanent deformation chart (Figure 7) developed by Makdisi and Seed (1978), the permanent deformation for the upper and lower failure surfaces was estimated to be 0.8 inch (20 mm) and 5 inches (130 mm), respectively. The upper bound of the range for an earthquake magnitude of 6.5 was used to estimate the permanent deformation. Because of uncertainties in using a simplified chart and the limitations of a permanent deformation analysis, the upper bound of each earthquake magnitude is recommended. Since the deep excavation and "grandfathered" area are not lined with a composite liner system that contains geosynthetics, a permanent deformation of 5 inches (130 mm) was deemed acceptable.

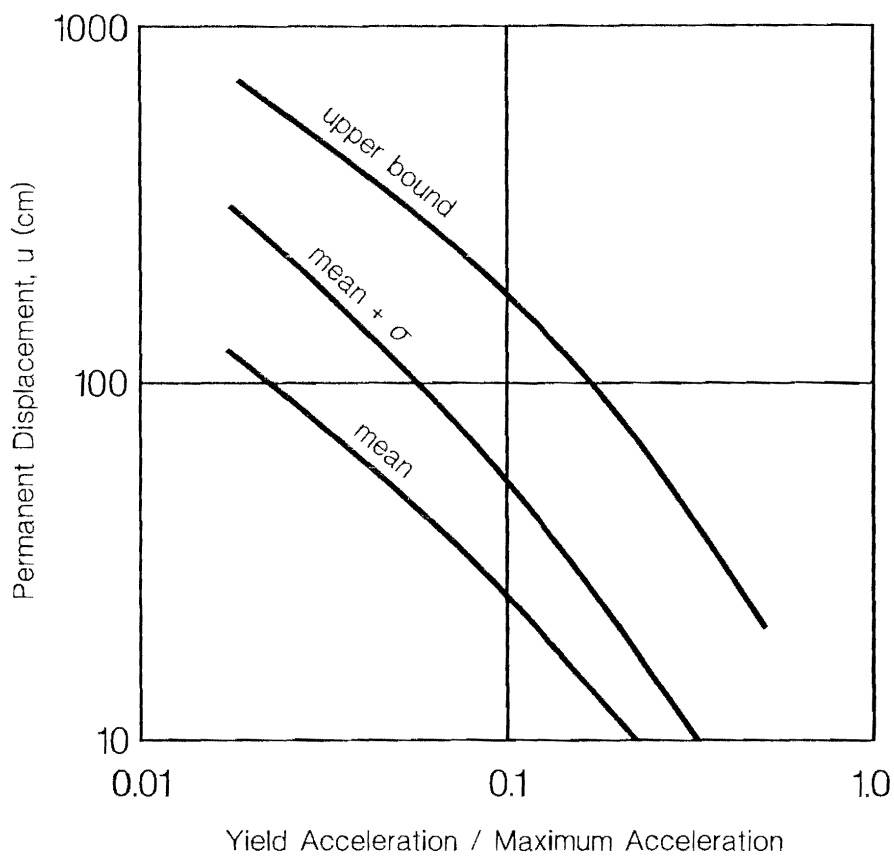

Figure 8. Permanent seismic deformation chart by Hynes and Franklin (1984)

The value of Kmax corresponds to the maximum value of the average horizontal acceleration for a potential slide mass. Since the upper and lower failure surfaces are located at the bedrock/soil or solid waste interface and no liner system was installed, the maximum horizontal bedrock acceleration was used as Kmax. If the failure surface passed through the

waste, the value of Kmax would have to correspond to the maximum average acceleration of the slide mass extending to a specified depth in the waste. This acceleration could be estimated using empirical correlations or calculated using a seismic response analysis.

In an effort to compare with another simplified permanent deformation analysis technique the Hynes and Franklin (1984) permanent deformation chart (see Figure 8 which is located on the previous page due to formatting constraints) was also used. Because of uncertainties in using this simplified chart and limitations of deformation analysis, the mean + σ relationship is frequently used. As previously mentioned, the ratio of yield/maximum accelerations for the upper and lower failure surfaces were calculated to be 0.81 and 0.5, respectively. It can be seen from Figure 8 that these ratios do not lie on the mean + σ relationship. As a result, this chart could not be used to estimate the permanent deformation. In addition, the effect of earthquake magnitude is not incorporated in this chart. In summary, the Makdisi and Seed (1978) permanent deformation chart is preferred because the effect of earthquake magnitude can be incorporated and the ratio of yield/maximum accelerations is plotted using an arithmetic scale instead of a logarithmic scale.

Conclusions

This case history illustrates the importance of the shear behavior of materials underlying the waste, e.g., native soil and/or geosynthetics, on slope stability. These native materials and/or geosynthetics should be sampled and tested to evaluate their shear strength and the resulting impact on interim and final slope stability. Based on published information and the analyses conducted for this case history, it is recommended that a drained residual shear strength be used to characterize the shear strength of colluvial soils underlying existing waste. Of course, the colluvium directly underneath the waste should be tested to estimate the drained residual strength because of the natural variability that occurs in colluvium across a site. Since colluvial deposits usually contain a thin, weak, highly plastic layer of cohesive soil, the average liquid limit and/or clay-size fraction for the deposit should not be used to estimate the drained residual friction angle from an empirical correlation.

The next main lesson involves the importance of toe excavations and the stability of interim slopes. As mentioned previously, the critical time for the stability of a slope is while the toe is excavated or daylighted. Clearly the time that the toe of the slope is exposed or daylighted must be minimized. Careful planning should be conducted to ensure that the slope is exposed for a minimum amount of time. If there are construction delays, waste may need to be diverted to another facility to ensure that the adjacent slope is not overbuilt. Engineers should investigate the stability of interim slopes, even though it may not required by regulations, to ensure slope stability during construction.

It is suggested that an interim slope exhibit a static factor of safety greater than 1.3 because of design and calculation uncertainties, a slope can be overbuilt without being detected, and most importantly people are usually required to work below the exposed slope. This also illustrates the need for close oversight by regulatory personnel to check the inclination and height of slopes during routine inspections. The slope also may warrant instrumentation to monitor slope movements while construction personnel are working below the slope. Prior to installation of the instrumentation, procedures for monitoring, quickly interpreting the results,

determining the criteria for imminent failure, and evacuating the excavation should be developed.

Acknowledgments

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