Geotechnical Stability of Waste Fills: Lessons Learned and Continuing Challenges

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Abstract: Several noteworthy stability failures occurred at landfills in the United States in the 1980s and early 1990s, a timeframe coinciding with the promulgation of modern US environmental regulations. These failures were extensively studied, and lessons were learned. A state-of-practice developed to enable the design of waste fills to be stable throughout their construction, operation, and closure periods. However, a survey of landfill performance in the United States in the 2010–2019 timeframe shows that waste fill stability failures continue to occur. This paper, an expansion of the 2018 Terzaghi Lecture given by the first author, presents a brief review of several waste fill failures from the 1980s and 1990s and the lessons learned during that period. Several more recent waste fill failures are then reviewed, from which it is concluded that 20–30 years after the earlier failures, facility operators and design engineers are relearning the earlier lessons, as well as new lessons related to evolving waste streams and operating practices. The paper concludes with a discussion of the current standard-of-care for the design of US waste fills and suggests that this standard can be improved through application of the lessons described herein. **DOI: 10.1061/(ASCE)GT.1943-5606.0002291.**

Author keywords: Waste fills; Slope stability; Shear strength; Pore pressures; Progressive failure; Lessons learned; Municipal solid waste.

Introduction

The modern age of solid waste disposal facility (i.e., landfill) engineering in the United States coincides with promulgation of federal and state regulations for landfills in the timeframe of the 1980s to early 1990s. These regulations led to the widespread use of soil–geosynthetic liner and cover systems and the development of large facilities designed with relatively steep slopes owing to the substantial value of disposal airspace and the significant costs and challenges associated with siting and permitting new facilities. The federal regulations that drove these developments were first promulgated by the US Environmental Protection Agency (EPA) for hazardous waste (HW) disposal facilities in July 1982 (EPA 1982), and for municipal solid waste (MSW) disposal facilities in October 1991 (EPA 1991). Concurrently, most states received delegated authority from EPA when they promulgated their own regulations that were at least as stringent as the federal regulations.

During the 1980s and 1990s, waste fill stability failures occurred at several HW and MSW disposal facilities (Table 1). In addition to waste fill failures, there were numerous liner system and cover system veneer-type stability failures (Bonaparte et al. 2002; Gross et al. 2002). Evaluations of the causes of these waste fill and veneer failures led to lessons learned and a state-of-practice for the geotechnical analysis and design of these facilities. In fact, the first author of this paper expressed optimism during that period that the state-of-practice had advanced to the point that landfills could be safely constructed, operated, and closed with respect to both environmental protection and geotechnical stability (Bonaparte 1995).

During the timeframe 2010–2019, waste fill stability failures have continued to occur in the United States, at a frequency estimated by the authors to be several per year. The more substantial incidents known to the authors are listed in Table 2. Several are listed as "confidential" at the request of the facility owner and, thus, do not include references. The authors and their colleagues were involved in investigations into the causes of these confidential failures. There may be additional waste fill stability incidents in the United States in the considered timeframe of which the authors are not aware.

The purpose of this paper is to review three of the early (1980s–1990s) US waste fill stability failures to identify the root causes and lessons learned that contributed to the design standard-of-practice developed during that period. Fast-forwarding 20–30 years, three waste fill stability failures from the 2010–2019 timeframe are then reviewed to identify the root causes and lessons learned from these more recent events. The authors conclude the paper with observations regarding these more recent failures and recommendations for improving the current state-of-practice.

While the focus of this paper is on solid waste landfills, the lessons described are broadly applicable to all types of waste fills including mine tailings piles, phosphogypsum stacks, and coal combustion residuals (CCR) impoundments and landfills. The 2015 promulgation of the Federal CCR Rule (EPA 2015) is resulting in the closure of several hundred US CCR impoundments and landfills in the coming years. As shown by the failure of a CCR impoundment in Tennessee in 2008 (AECOM 2009), the cost and consequences associated with a large CCR impoundment failure can be significant. Similarly, the Mount Polley mine tailings impoundment failure in Canada in 2014 illustrates the significant cost and consequences of a failure at this type of facility (Morgenstern 2018).

This paper is limited in scope to US facilities and engineering practices. The authors recognize that numerous waste fill failures have occurred in developing countries that have resulted in

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Table 1. Representative US waste fill stability failures, 1984–1996

Name	Location	Date	Nature of failure	References
Global	Old Bridge Township, New Jersey	1984	Waste mass and foundation	Oweis et al. (1985)
Kettleman Hills	Kettleman City, California	1988	Waste mass and liner system interface	Seed et al. (1988, 1990), Mitchell et al. (1990a, b, 1993), Geosyntec (1991), Byrne et al. (1992), Byrne (1994), Stark and Poeppel (1994), Gilbert et al. (1998), and Filz et al. (2001)
Crossroads	Norridgewock, Maine	1989	Waste mass and foundation	Reynolds (1991), Richardson and Reynolds (1991), Luettich et al. (2015), and Reynolds (2015)
Chiquita Canyon	Los Angeles, California	1994	Liner system tearing and waste displacement due to seismic ground motions	Augello et al. (1995) and Kavazanjian (2006)
Rumpke	Coletrain Township, Ohio	1996	Waste mass and foundation	Geosyntec (1996a, b), Kenter et al. (1997), Schmucker and Hendron (1997), Eid et al. (2000), Stark et al. (2000) and Chugh et al. (2007)
Mahoning	Mahoning, Pennsylvania	1996	Waste mass and liner system interface	Stark et al. (1994)

Table 2. Representative US waste fill stability failures, 2010–2019

Name	Location	Date	Nature of failure	References
Matlock Bend	Loudon County, Tennessee	2010	Waste mass	Geosyntec (2011)
Confidential	Eastern, Mid-Atlantic	2011	Waste mass above intermediate soil cover layer	Project files containing investigative data and root cause analysis
Confidential	Southeast, Gulf Coast	2012	Waste mass and foundation	Project files containing investigative data and root cause analysis
Big Run	Ashland, Kentucky	2013	Waste mass	Gilbert (2014)
Chrin Brothers	Williams Township, Pennsylvania	2013	Waste mass and liner system interface	Stark (2016)
Tri-Cities	Petersburg, Virginia	2015	Waste mass	Virginia Waste Management Board (2015)
Confidential	Northeast, Appalachian Plateau	2017	Waste mass above intermediate cover soil layer	Project files containing investigative data and root cause analysis
Confidential	Southeast, Piedmont	2018	Waste mass downslope creep; veneer-type landfill expansion instability	Project files containing investigative data and root cause analysis
Confidential	Southeast, Coastal Plain	2018	Waste mass	Client communication
Confidential	Southeast, Piedmont	2019	Waste mss	Project files containing investigative data and root cause analysis

substantial loss of life, a consequence that has thankfully been avoided in the United States failures identified in this paper with one exception. In these developing countries, the regulations governing waste fills, the design state-of-practice, and the disposal facility operational practices are not as advanced as in the United States. In some countries, uncontrolled dumping still occurs. A number of references are available that describe international waste fill failures, including Hendron et al. (1999), Koerner and Soong (2000), Caicedo et al. (2002), Fernandez et al. (2005), Blight (2008), Huvaj-Sarihan and Stark (2008), Koerner and Wong (2011), Anthanasopoulous et al. (2013), Jafari et al. (2013), Lavigne et al. (2014), Reddy and Basha (2014), De Oliveira et al. (2015), Peng et al. (2016), Xu et al. (2017), and Morgenstern (2018).

Potential Failure Modes

The general configuration of the waste fill, liner and cover systems, leachate collection and removal system (LCRS), and other features in a typical US landfill is shown in Fig. 1. The waste fill in this type of structure is potentially susceptible to several failure modes. These include movements along slip surfaces (Fig. 1): (1) entirely within the waste fill, (2) through the waste fill and foundation, (3) through the waste fill and along liner system interfaces, and (4) entirely along liner or cover system interfaces and/or through liner or cover system soils. These potential failure modes must be considered for both static and seismic conditions (if the facility is in a seismic impact zone) and for critical interim operating (i.e., interim slope configurations) and final closure conditions. Waste fill stability is influenced by the shear strength and stress–strain characteristics of the waste fill materials, foundation materials, and liner system interfaces. It is also influenced by liquid and gas pressures and temperatures within the fill (e.g., Merry et al. 2006). The strength of the waste is dependent on composition, age, organic content, moisture content, confining stress, and other factors (e.g., Bray et al. 2008). Interface strengths are dependent on materials, moisture conditions, normal stresses, rate and magnitude of displacements, and other factors (e.g., Fox and Stark 2015). For brevity, this paper is focused only on failures that involve movement of the waste fill. Veneer-type liner system and cover system failures that involve only relatively shallow movements of liner/cover system components, also illustrated in Fig. 1, are not addressed.

Early Waste Fill Failures

From the waste fill failures that occurred in the 1980s and 1990s, three are briefly reviewed to identify the root cause(s) of the failures

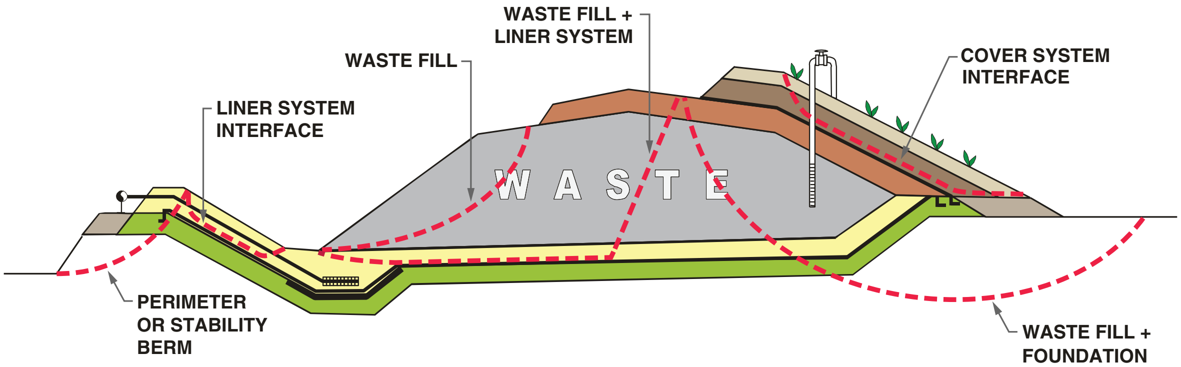

Fig. 1. (Color) Typical configuration of modern US landfill showing potential waste fill stability failure modes.

Fig. 2. (Color) Aerial view of Kettleman Hills Landfill Unit B-19, Phase 1A, showing the interim waste configuration prior to failure and the direction of waste movement (arrow). (Reproduced from Mitchell et al. 1990a, © ASCE.)

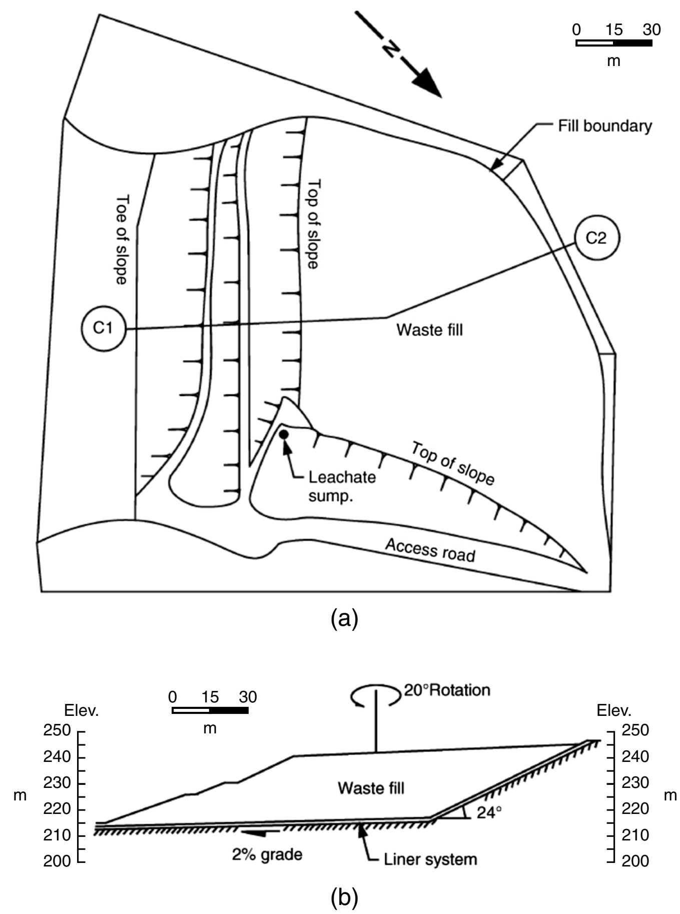

Fig. 3. (a) Preslide topography; and (b) cross section, Kettleman Hills Landfill Unit B-19, Phase 1A. (Reproduced from Mitchell et al. 1990a, © ASCE.)

and the lessons learned: (1) Kettleman Hills HW landfill failure in 1988; (2) Crossroads MSW landfill failure in 1989; and (3) Rumpke MSW landfill failure in 1996.

Kettleman Hills

The Kettleman Hills (California) HW landfill failure has been described by Seed et al. (1988, 1990), Mitchell et al. (1990a, b, 1993), Geosyntec (1991), Byrne et al. (1992), Byrne (1994), Stark and Poeppel (1994), Gilbert et al. (1998), and Filz et al. (2001).

Operation of Unit B-19, Phase 1A of the landfill began in early 1987. Placement of solid waste and cover soil proceeded at a roughly constant rate over the next year. On March 19, 1988, with the waste fill in an interim configuration with a maximum height of about 27 m and average slope face of about 3.1H:1V (horizontal: vertical), 445,000 $m^3$of waste and cover soil slid downslope and laterally about 10.7 m over a period of several hours. Fig. 2 shows an aerial view of the landfill cell prior to failure. The arrow on the figure shows the direction of subsequent waste movement. Fig. 3 presents a plan view and cross section of Phase 1A at that time. Fig. 4 presents a cross section through the double-composite liner system for the base portion (i.e., floor) of Phase 1A.

Each of the two composite liners on the base consisted of an upper smooth high-density polyethylene (HDPE) geomembrane underlain by a compacted clay liner (CCL). The CCL was constructed of processed on-site claystone and siltstone amended with

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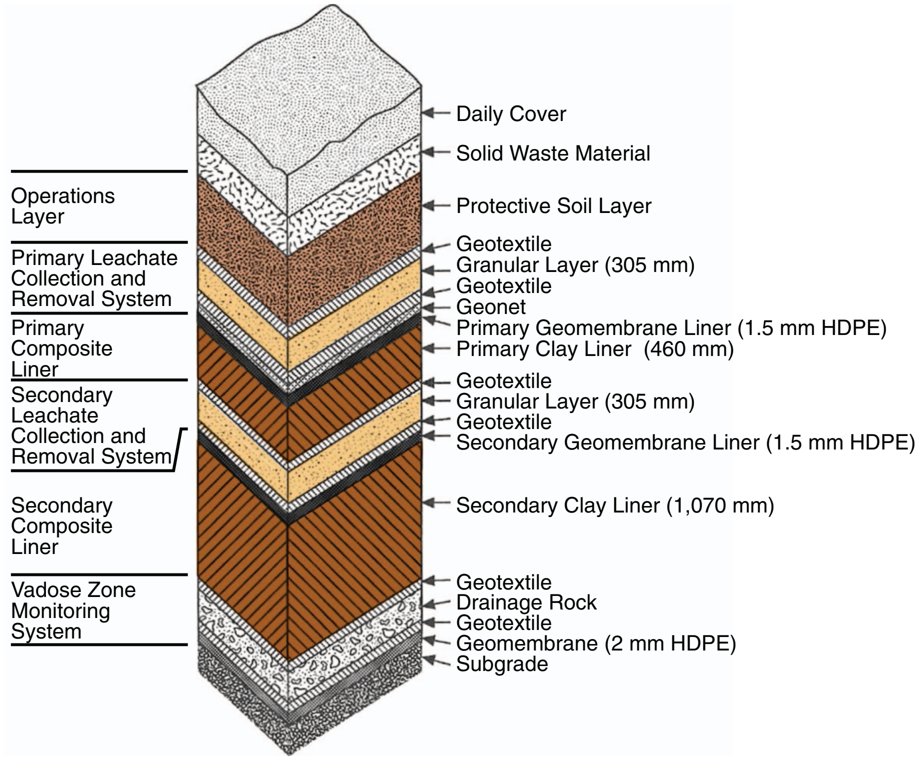


Fig. 4. (Color) Double-composite liner system for base portion of Kettleman Hills Landfill Unit B-19, Phase 1A. On the sideslope, primary CCL thinned out, a geonet was used for the primary and secondary LCRSs, and a vadose zone monitoring system was absent. (Adapted from Byrne et al. 1992)

2%–5% bentonite. The blended material classified as CH based on the Unified Soil Classification System (USCS), with a liquid limit (LL) of 60%–70% and a plasticity index (PI) of 40%–50% (Byrne et al. 1992). The primary and secondary CCLs were 0.46 and 1.07 m thick, respectively. Granular drainage layers (Fig. 4) were installed as primary and secondary LCRSs. The double-liner system on the sideslopes varied somewhat from that on the base, namely (1) the primary CCL on the landfill base thinned out on the landfill sideslopes (i.e., the secondary CCL on the base extended to the top of the slope but the primary CCL did not); (2) a geonet drainage layer was used for the primary and secondary LCRSs on the sideslopes in lieu of the granular drainage layers used on the base; and (3) a vadose zone monitoring system installed beneath the liner system on the base was absent on the sideslope. As can be seen in Figs. 2 and 3, Phase 1A was roughly bowl-shaped on three sides, with 2H:1V liner system sideslopes along the southwestern to northern sides and 3H:1V sideslopes on the northeastern side. The southeastern side of the bowl was open. The landfill was designed by a well-regarded and experienced engineering firm. Further, the liner system was installed using a detailed construction quality assurance (CQA) program implemented by another well-regarded engineering firm. In short, it appeared that both design and construction were performed thoroughly and well. So, why the failure?

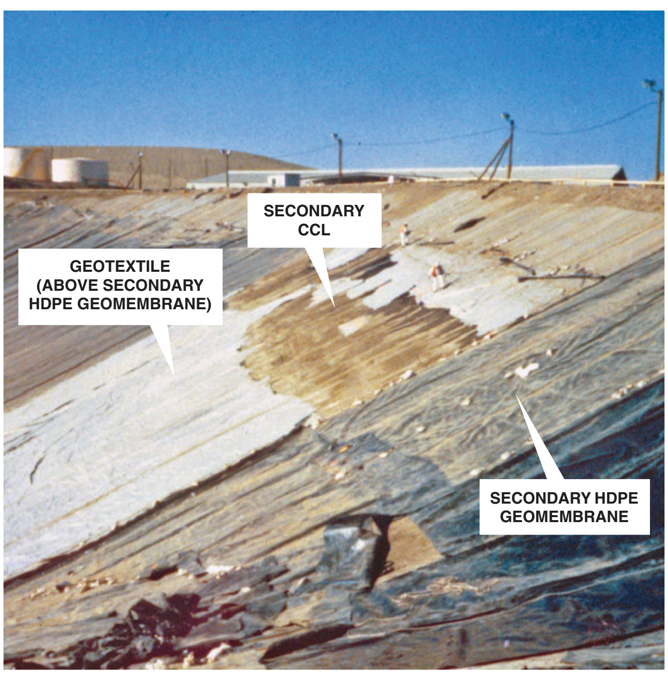


Fig. 5. (Color) Tear in geotextile layer (shown) and underlying secondary HDPE geomembrane above secondary CCL along northwest slope. (Reproduced from Byrne et al. 1992, © ASCE.)

The postfailure investigation revealed a translational sliding mechanism along the sideslope and base of the waste fill. The direction of sliding was to the southeast where the bowl was open. Sliding was observed to occur along liner system interfaces, principally the secondary geomembrane–CCL interface (Fig. 5), although shearing along the primary geomembrane and underlying geotextile interface was observed in a few sideslope areas. This latter interface only occurs along the upper part of the sideslope where the primary CCL is not present.

Shearing along the secondary geomembrane–CCL interface was found to have occurred under essentially undrained conditions owing to the slow rate of consolidation of the secondary CCL compared with the rate of cell filling (Byrne et al. 1992). Postfailure laboratory direct shear testing of the interface at the interpreted

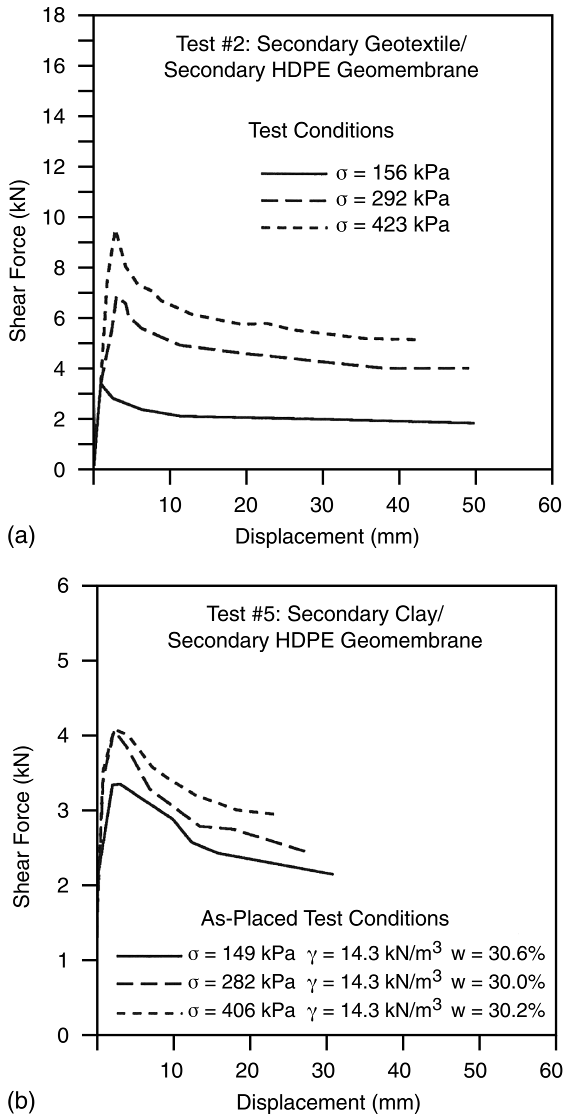


Fig. 6. Representative interface direct shear test results for Kettleman Hills Landfill Unit B-19, Phase 1A. (Reproduced from Geosyntec 1991, with permission from Waste Management, Inc.)

mean CCL in situ moisture content and dry density produced peak and residual undrained interface strengths of about 44 and 24–34 kPa, respectively. Measured peak and residual friction angles of 12°–14° and 8°, respectively, were obtained for interfaces between the smooth HDPE geomembranes and geotextiles. Peak strength conditions in the laboratory tests for both interfaces occurred at shear displacements of only 2–5 mm, with postpeak softening and substantial reductions in interface strengths at shear displacements of only 5–20 mm (Fig. 6).

Limit equilibrium analyses [both two-dimensional (2D) and three-dimensional (3D)] using combinations of residual and peak interface strength values resulted in factors of safety that could explain the failure. Subsequent finite difference and finite element analyses by Byrne (1994) and Filz et al. (2001), respectively, elucidated the mechanism for progressive shearing and strength loss at the critical interfaces that proved to be the root cause for this failure.

Lessons learned from the Kettleman Hills waste fill failure include the following:

- Liner system interfaces can exhibit pronounced shear softening at small displacements, with residual strengths much lower than peak strengths. Geomembrane–CCL interfaces may undergo undrained shear depending on the rate of waste filling and the thickness and consolidation characteristics of the CCL.
- Liner system construction and waste placement/compaction operations can induce movements that mobilize postpeak interface strength conditions. Prior to the Kettleman Hills failure, this fact was not widely recognized. Today, it is known that waste fill designs need to account for interface displacements that occur during construction of the liner system and filling of the landfill. In response to this knowledge, new materials have been developed to provide greater interface strength, e.g., textured geomembranes, as have design approaches to mitigate the problem (e.g., controlled slip interfaces above the critical liner system interface).
- Substantial differences exist in the stress–strain behavior of waste materials (usually ductile) compared with liner system interfaces (often brittle and strain softening). Thus, peak interface shearing resistances in liner systems will typically develop at much smaller displacements than those required to develop peak shearing resistances in the waste. The design of waste fills must thus consider the potential for progressive failure with only a portion of the waste's strength mobilized. After Kettleman Hills, a state-of-practice developed taking these factors into account. Examples of practice recommendations are given in Gilbert and Byrne (1995), Thiel (2001), Bonaparte et al. (2004), Stark and Choi (2004), and others.
- Waste mass stability evaluations need to address all interim waste filling configurations ("all development phases"), not just the final waste configuration. The failure at Kettleman Hills involved the waste mass when it was in an interim configuration.
- Geomembrane–CCL interface strengths are sensitive to the CCL compaction conditions. Compaction conditions that favor low permeability and intimate geomembrane–CCL contact (i.e., kneading compaction at moisture contents wet of the line of optimums, followed by smooth rolling the top surface of the CCL prior to geomembrane installation) also favor low interface shear strength and slow rates of CCL consolidation. Liner system designers must be cognizant of this fact.
- Lined waste fills may have complex geometries that can result in lower calculated 3D slope stability factors of safety than the calculated 2D factors of safety (Mitchell et al. 1993). One of the main challenges for the design engineer in using 2D analyses for lined waste fills with complex geometries is in selecting 2D cross sections representative of the 3D structure of the fill.

Crossroads

The Crossroads (Maine) landfill failure is described in Reynolds (1991, 2015), Richardson and Reynolds (1991), and Luettich et al. (2015). Construction of the landfill began in 1976, and it was operated until a waste fill and foundation failure occurred in August 1989. The landfill received primarily MSW.

The landfill site is underlain by a roughly 18-m-thick sensitive glaciomarine clay–silt deposit (Presumpscot Formation) that exhibits an overconsolidated crust (weathered, stiff, olive brown) ranging from about 1.5 to 3.6 m in thickness. The deposit transitions to a lightly overconsolidated to normally consolidated condition (unweathered, soft, gray) at a depth of about 4.5–6.0 m. LLs and PIs for the Presumpscot Formation at the Crossroads site are dependent on the silt and clay fractions in the samples and range from 25% to 40% and 10% to 20%, respectively. Regionally, the Presumpscot Formation is reported to have a sensitivity in the range of 5–10 (Andrews 1987). The landfill did not contain

Fig. 7. (Color) Aerial view of 1989 failure of Crossroads Landfill caused by west–east retrogressive foundation failure; arrow shows direction of waste movement. (Reproduced from Luettich et al. 2015, with permission of University of Maine.)

Fig. 8. (Color) Close-up view of Crossroads Landfill failure showing intact waste blocks and scarps caused by retrogressive foundation failure. (Reproduced from Luettich et al. 2015, with permission of University of Maine)

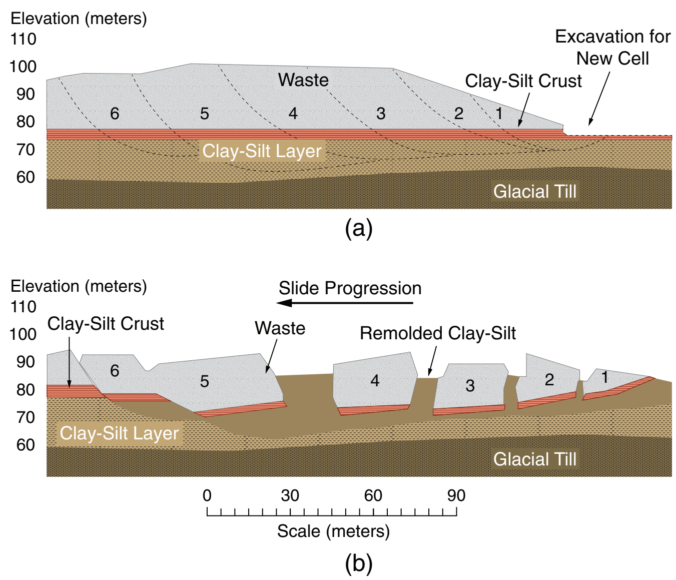


Fig. 9. (Color) Crossroads Landfill: (a) condition prior to failure showing overconsolidated clay–silt crust transitioning to underlying normally consolidated clay–silt layer and excavation of the crust at the western toe; and (b) condition after failure with intact MSW blocks and crust material displaced on the remolded clay–silt layer. (Reproduced from Reynolds 2015, with permission of University of Maine.)

any constructed liner or LCRS. Instead, the underlying glaciomarine layer served as a hydraulic barrier. The bottom of the waste was placed directly on top of the overconsolidated crust at the ground surface. By August 1989, the waste fill had reached a maximum height of about 21 m, with 3H:1V sideslopes, and covered an area of about 3.2 ha. After a period of heavy rain (125 mm over 10 days), the landfill underwent a rapid (estimated to be about 1 minute) west-to-east retrogressive failure (Fig. 7) involving approximately 500,000 m^3 of MSW and cover soil translating as much as 50 m.

Postfailure investigations showed that the sliding surface was principally in the normally consolidated portion of the glaciomarine layer below the overconsolidated crust. As can be seen in Fig. 7, the slide morphology consisted of a series of intact blocks of waste and crust material essentially floating on the underlying remolded clay–silt layer, with the blocks moving to the west. Crevices between blocks were as much as 15 m wide and 9 m deep. Fig. 8 shows a close-up view of the waste blocks, and Fig. 9 illustrates the structure of the waste fill just prior to, and after, the failure.

In mid-1986, three years before the failure, in situ vane shear and laboratory triaxial compression testing of the foundation soils were conducted to evaluate the stability of the growing landfill. The vane shear testing, conducted using a Maine Test Boring (MTB) vane (Reynolds 2015), produced undrained shear strengths (uncorrected) in the range of 50–100 kPa for the overconsolidated crust and 20–25 kPa for the normally consolidated zone below the crust. Based on the results of the stability evaluation, a height limitation of 17 m was placed on the landfill at that time. Nonetheless, from late 1987 to early 1989, the landfill height progressively increased to 21 m. This increase was rationalized based on monitoring results for inclinometers installed around the perimeter of the landfill indicating a maximum lateral displacement rate of about 1.5 mm per month (with the origin of the movements being in the normally consolidated zone below the crust). This displacement rate was interpreted as "high, but acceptable" based on the stability of the readings and piezometers installed near the inclinometers that indicated essentially no excess pore pressure buildup.

In the spring of 1989, a few months prior to the failure, "updated" slope stability analyses were performed for the landfill, resulting in a minimum calculated factor of safety (FS) close to 1.0. This FS was reportedly obtained using a literature-derived value for the clay–silt deposit's undrained shear strength ratio (S_u/σ'_v), an estimate of the in situ effective-stress profile based on an estimated degree of foundation consolidation, and "updated" unit weight values for the MSW (described subsequently in this section). The foundation shear strengths so derived were reported to be "considerably" larger than the vane-derived shear strengths used in the 1986 slope stability evaluation. Notwithstanding the new FS analysis results, operation of the landfill continued, but actions were taken to improve stability. These included ceasing placement of waste near the crest of the landfill slope and constructing stabilizing berms along the toes of the east, north, and south landfill slopes (but not adjacent to the western toe).

Investigations conducted after the failure identified several contributory factors. Most obvious is filling of the landfill to a height and at a rate that resulted in a calculated FS value approaching 1.0. However, the agreement between the failure height of the landfill and the calculated FS appears to be somewhat a coincidence. Based on the postfailure investigation, foundation shear strengths used in the 1989 stability analyses appreciably underestimated the actual strengths (i.e., the foundation soils were up to 25%–30% stronger than the strengths used in the stability analyses). Potential contributors to this include underestimation of (1) the degree of consolidation of the foundation soil beneath the landfill, (2) the soil S_u/σ_v' ratio, and (3) the magnitude of the applied waste fill load. In subsequent studies, Reynolds (2015) showed that the laboratory-measured S_u/σ_v' ratio of the sensitive foundation soil at the site is dependent on the sampling methods used and demonstrated the increase in undrained strength due to secondary compression and drained creep. The time-dependent loading of the foundation by the weight of the landfill represents a complex, staged-construction stress path, requiring that pore pressure dissipation and soil strength gain be accurately assessed. In hindsight, there was substantial unrecognized conservatism in 1989 regarding the foundation shear strength profile.

In the 1989 slope stability analyses, the unit weight of the waste was also underestimated, which counterbalanced the underestimation of the foundation shear strengths. The original (1976) estimate for waste unit weight was 5.8 kN/m^3 based on the scant technical literature available at that time. By 1987, engineers had used the recorded gate tonnage, the consumed landfill volume, and an estimate for cover soil percentage to calculate an in-place average unit weight of 12.4 kN/m^3; this was the value used in the 1989 stability analyses that produced FS $\approx$ 1.0. Based on postfailure, large-scale field testing, the measured average in situ unit weight of the waste fill was 15.1 kN/m^3. Thus, the foundation load applied by the waste fill was about 22% larger than used in the 1989 slope stability analyses. The high unit weight of the waste at the Crossroads site was attributed to a high percentage of cover soil in the waste coupled with high moisture content due to the lack of leachate collection. The high unit weight of the waste may have also contributed to more consolidation strength gain of the foundation soil prior to the failure than recognized at the time. It is unclear based on the available information whether leachate levels in the waste fill were known in 1989 and included in the stability analyses.

The trigger for the landfill failure appears to have been a 2-m-deep excavation at the western toe of the landfill in the weeks prior to the failure. The excavation was for a new Subtitle D lined landfill cell immediately adjacent to the existing waste fill. This excavation unloaded the area adjacent to the toe of the existing fill while at the same time removing the relatively strong overconsolidated glaciomarine crust. Based on analyses conducted by Reynolds (1991), this excavation resulted in a 14% decrease in the calculated factor of safety at the toe. The excavation destabilized the western landfill toe, triggering an initial localized failure that quickly retrogressed across the site as a series of distinct waste blocks sitting on a base of remolded clay–silt material (Fig. 9). Had the excavation not been made, and had an earthen buttress been installed in 1989 along the western toe (as it had around the other three sides of the landfill), the failure might have been avoided. With waste disposal soon moving to a new lined cell, filling of the original cell would have ceased and the foundation soil beneath the cell would have continued to undergo primary and secondary compression and gain strength.

Lessons learned from the Crossroads waste fill failure include the following:

- Waste unit weight is a critical design parameter as waste self-weight is the principal source of foundation loading beneath most waste fill structures.
- An accurate understanding of the liquid levels and pore pressure conditions in both the waste fill and underlying foundation is critical to the satisfactory assessment of waste fill and foundation stability.
- Each significant construction and/or operational change in the field should be evaluated prior to implementing change; in this case, excavation at the western toe unloaded the sensitive glaciomarine clay–silt deposit and reduced the calculated FS of the landfill toe by about 14%.
- For sites where foundation stability is dependent on the progress of soil consolidation and strength gain under the time-dependent loading of the waste fill, this progress must be accurately defined for each stage of waste filling; an adequate field instrumentation program is essential to confirming foundation settlements and pore pressures; and, depending on the rate of filling, engineering measures (e.g., underdrains, vertical drains) may be needed to accelerate this process. Postfailure geotechnical investigations demonstrated the challenges and importance of obtaining high-quality in situ data and samples from this site. Postfailure investigations for the Crossroads landfill showed measured vane strengths for the clay–silt deposit were about 15% higher with a Geonor H-10 Vane Borer (with lightweight actuator rods) compared with the MTB vane used in 1986. Reynolds (2015) attributed the difference in results to greater soil disturbance with the heavier MTB vane. Similarly, sampling techniques were improved in subsequent projects at the site from cased jetted borings and Shelby tube sampling to rotary borings using drilling muds and hydraulically actuated fixed piston samplers. Laboratory shear strength measurements after the failure focused on the use of direct simple shear (DSS) testing. A staged DSS test methodology was developed to evaluate undrained shear strength gains in the clay–silt deposit due to drained shearing (creep) under the incremental loading of the waste fill over multiple years (Reynolds 2015).

Rumpke

The Rumpke (Ohio) MSW waste fill failure has been analyzed and discussed by Geosyntec (1996a, b), Kenter et al. (1997), Schmucker and Hendron (1997), Eid et al. (2000), Stark et al. (2000), and Chugh et al. (2007). The failure occurred in March 1996. It involved an 8-ha portion of the landfill, with the movement of roughly 1.15 million m^3 of MSW and cover soil up to about 270 m into a deep excavation to the immediate north of the landfill (Figs. 10 and 11). The purpose of the excavation was to create an area for a new Subtitle D lined landfill expansion. Although the excavation itself did not contribute directly to the initiation of failure, it allowed the failing waste to accelerate down the 3H:1V excavation slope and then across the excavation floor.

The Rumpke landfill was started in the 1940s by pushing dumped waste into a ravine on the property. The waste was placed directly onto the ground where a brown clayey colluvial and residual soil layer (hereafter referred to as native brown soil) formed a 2- to 5-m-thick mantle over shale and limestone bedrock. During the roughly 50-year landfill operating life, waste was filled

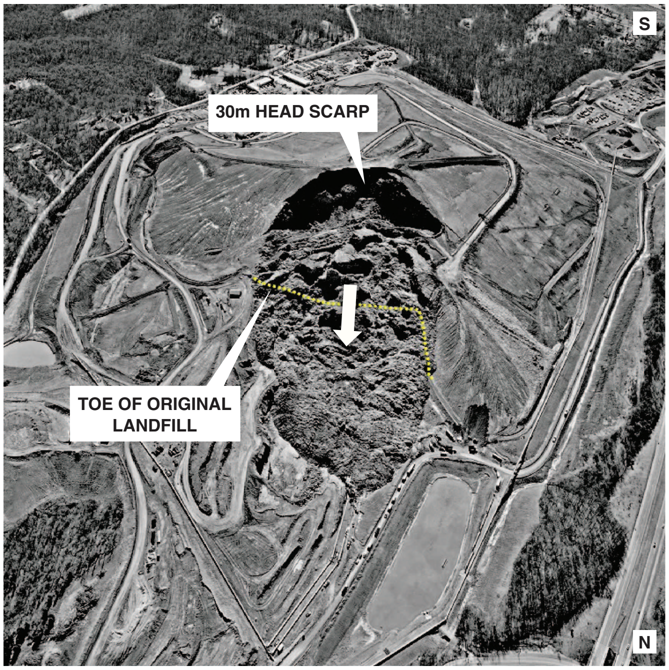


Fig. 10. (Color) Aerial postfailure view of Rumpke Landfill landslide; arrow shows direction of waste movement. (Reproduced from Stark et al. 2000, © ASCE.)

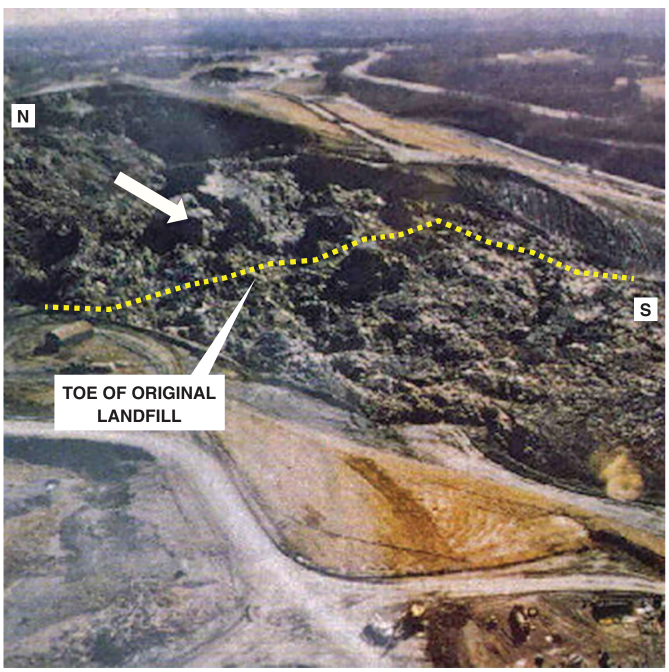


Fig. 11. (Color) Aerial close-up view of Rumpke Landfill landslide; arrow shows direction of waste movement. (Adapted from Eid et al. 2000.)

to a state-permitted maximum elevation of about 325 m above mean sea level (msl) and with a state-permitted maximum slope of 3H:1V. As reported by Stark et al. (2000), owing to delays in preparing the lateral expansion, the landfill operator continued to place waste onto the landfill to heights 10–12 m above the state-permitted maximum elevation and to an average northern slope inclination of 2.6H:1V. Fig. 12, modified from Stark et al. (2000), shows a representative cross section through the Rumpke landfill at the time of failure. The landfill did not have an LCRS; piezometric levels in the fill were estimated to be as high as 15 m above the bottom of the landfill at the location of the head scarp when it failed (Fig. 12).

A postfailure investigation (Geosyntec 1996a, b) revealed that the failure surface extended at a near vertical angle from the landfill crest, through the waste, to the native brown soil where it followed the bedding of the soil layer (a few degrees, on average) until daylighting at the landfill toe. The slide occurred quickly, reportedly over a period of about five minutes. A week prior to the slide, tension cracks were observed at the top of the landfill, at the location of the subsequent head scarp. The cracks slowly widened throughout the week and toe bulging was observed. The morning of the slide, the landfill began to experience observable movement and increasing leachate seepage at the toe. At the same time, the tension cracks at the landfill crest were observed to be growing in length and width. After several hours of gradually increasing creep rates and a total movement at the landfill toe of about 4 m, the landfill failed. The failure started at the toe and then retrogressed rapidly to the crest of the waste fill. After the failure, the near-vertical head scarp was almost 30 m in height (Fig. 10).

As part of the postfailure investigation, several borings were advanced at the site, several inclinometers were installed, the properties of the brown native soil were characterized, and slope stability analyses were conducted. The native brown soil was found to have LLs and PIs in the range of 40%–75% and 25%–50%, respectively, with clay fractions in the range of 25%–60% and USCS classifications of CL and CH. In situ moisture contents were measured to be in the range of 20%–35%. A few drained torsional ring shear tests were independently conducted by Geosyntec (1996b) and Eid et al. (2000). Drained residual friction angles from the tests varied based on the sample properties and exhibited reasonable agreement with the correlations to LL, clay fraction, and effective normal stress proposed by Stark and Eid (1994) and Stark and Hussain (2013). The more plastic, higher–clay fraction samples exhibited drained residual friction angles in the range of 10°–13°. The drained, fully softened friction angles for these samples were 23°–24°. Laboratory test results on undisturbed samples showed the native brown soil exhibiting brittle stress–strain characteristics, with peak shearing resistances developed after only a few millimeters of displacement.

As part of the forensic investigation, slope stability analyses were conducted by both Geosyntec (1996b) and Stark et al. (2000) to back-calculate mobilized strengths (reported as secant residual friction angles) along the failure surface at the initiation of failure. Geosyntec obtained back-calculated values ranging from 10° to 15° for several 2D cross sections. Stark et al. reported a back-calculated value of 13.5° based on 3D analyses.

The root causes of the Rumpke waste fill failure are assessed as (1) lack of recognition prior to the failure of the brittle, strain softening behavior of the native brown soil and its low residual shear strength (as low as 10°–13°); (2) lack of recognition of the buildup of leachate levels in the waste fill that reduced the effective stresses in the fill and at the native brown soil interface; and (3) the filling of the landfill beyond its permit limits, which increased the shear stresses applied to the native clay material. In addition, there are several other factors that may have contributed to triggering of the slide. These are (1) the excavation of the toe of the waste fill slope on the north side, adjacent to the expansion excavation, to provide an access road around the southern edge of the excavation; and (2) a prolonged period of frigid weather prior to the failure, which may have caused leachate draining from the landfill near

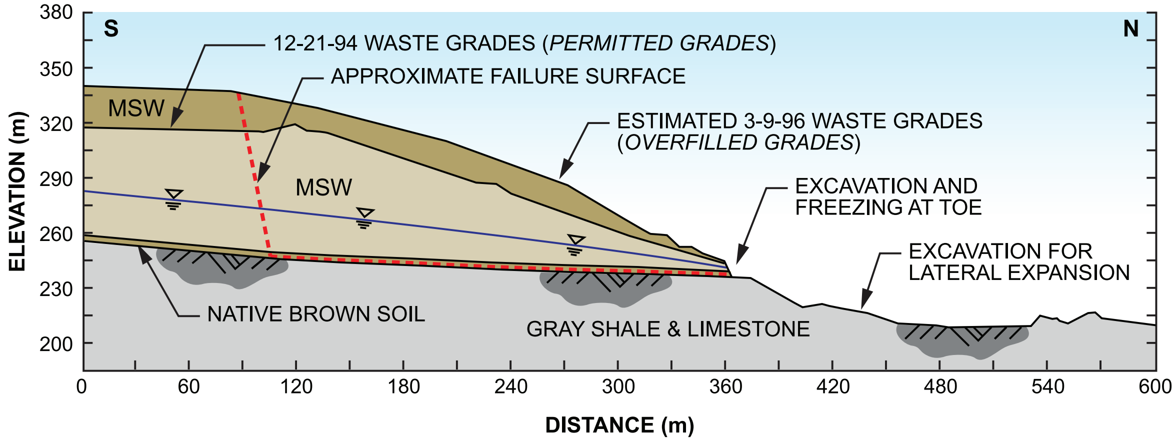


Fig. 12. (Color) Idealized, representative cross section through Rumpke Landfill showing prefailure geometry, extent of overfilling, and location of failure surface. (Adapted from Stark et al. 2000.)

the toe to freeze, possibly creating an ice dam that allowed localized leachate head buildup at the toe.

Based on the postfailure investigations and analyses that were conducted, the following lessons were derived from the Rumpke waste fill failure:

- Foundation conditions for old unlined waste fills must be thoroughly understood if additional filling, excavation, or expansion of the fill is planned.
- Strain incompatibility between MSW and foundation soil (in this case the native brown soil) can lead to uneven development of shear resistances in these materials, with the soil developing its peak strength and then undergoing strain softening prior to development of an appreciable fraction of the strength of the waste fill. This incompatibility should be addressed in design through an evaluation of the potential for progressive failure and the use of postpeak soil strengths and/or factored strengths for the waste fill.
- Leachate buildup in old unlined waste fills can reduce slope stability factors of safety and contribute to the development of unstable slope conditions.
- Surface cracking and toe bulging in waste fills may provide early indications of marginally stable slope conditions. Site operations personnel should be trained to recognize these indicators and promptly alert site management should they be observed. Monitoring programs, using instruments such as slope inclinometers, survey markers, optical fibers, and other means, can also provide early indications of marginal stability slope conditions.
- Operational activities (e.g., toe excavation and freezing conditions) can serve as triggers to slides at waste fills that are already only marginally stable.

Based on the waste fill failures described in this paper, plus others not reviewed here, many lessons had been learned by the mid-1990s and they contributed to the development of a state-of- practice with respect to the evaluation and design of waste disposal facilities for geotechnical stability. The lessons most relevant to the scope of the paper are given in Table 3.

Recent Waste Fill Failures (2010–2019)

As noted in the introduction, waste fill failures have occurred in the United States during the 2010–2019 timeframe. Table 2 presents a partial list of these recent MSW waste fill failures. Three of these failures are reviewed here to assess their root cause(s) and the lessons that can be learned from them.

Confidential Eastern MSW Landfill

This facility is in the mid-Atlantic US Coastal Plain physiographic province at a location that receives average annual precipitation of about 1,200 mm. It is a large facility that started operations in 1994. At the time of failure, the landfill was about 58 m tall, with 3H:1V sideslopes incorporating drainage benches at about 12- to 24-m vertical intervals. The landfill is underlain by a composite liner system with a functioning LCRS. Prior to the failure, the landfill received MSW, dewatered sewage sludge, and other materials. The landfill operator also practiced leachate recirculation for many years, and the facility's flat top deck and stormwater management practices resulted in significant stormwater infiltration into the fill. These conditions, taken together, resulted in a wet, partially saturated waste fill with elevated piezometric levels.

To obtain additional disposal capacity at the site, a permitted lateral expansion was developed by constructing a 30-m-wide outward extension to the original landfill base liner system and then filling the expansion area as a roughly 30-m-wide prism of waste placed against the 3H:1V sideslope of the original landfill [Fig. 13(a)]. As indicated on the figure, the original sideslope was covered by a low-permeability intermediate cover soil layer that was left in place when the expansion was developed. This cover soil impeded vertical percolation of leachate from the expansion area to the LCRS at the bottom of the original landfill. At the same time, the expansion design did not include a new drainage layer installed on top of the intermediate cover soil layer of the original landfill to allow leachate drainage in the expansion area. The base liner system for both the original landfill and expansion consisted of a single composite liner (0.6-m-thick CCL overlain by a 1.5-mm-thick textured HDPE geomembrane) overlain by a triplanar geocomposite drainage layer and then a 0.3-m-thick sand layer. Above the liner system, a "recirculation liner system" was installed that consisted of a 0.75-mm-thick textured HDPE geomembrane overlain by a 0.3-m-thick soil layer and a 0.6-m-thick layer of tire chips. This design was intended to maintain a leachate head of less than 0.3 m in the LCRS (as required by regulation) while allowing the buildup of larger leachate heads on top of the recirculation liner system. Of note, no part of the liner system was involved in the failure.

Table 3. Lessons learned by mid-1990s regarding geotechnical stability of waste fills

Category	Lesson learned
Site investigations	Geotechnical site investigations are as important to the design of waste fills as they are to any other type of critical geotechnical infrastructure.
Waste material characterization	Waste materials have geotechnical properties that must be reliably evaluated or conservatively estimated for design.
Liquid and gas pressures	Moisture conditions and liquid and gas pressures within the waste fill must be reliably evaluated or conservatively estimated for design.
Geosynthetic interfaces	Geosynthetic–geosynthetic and soil–geosynthetic interfaces can be weak and sensitive to moisture and placement conditions.
Geosynthetic interface testing	Project-specific soil and geosynthetic interface testing should be conducted if there is any doubt about the adequacy of available lab databases to represent the project-specific conditions.
Mobilized strength compatibility	Mobilized strength compatibility needs to be carefully considered with respect to waste (often ductile), geosynthetic interfaces (often brittle and strain softening), and foundations (can be sensitive, brittle, slickensided, strain softening, undrained, and/or liquefiable).
Progressive failure	Strain-softening and progressive failure mechanisms must often be considered in waste fill stability analyses owing to material stress–strain incompatibilities.
Soft soil sites	Time-dependent staged loading effects on foundation pore pressure generation and dissipation, and the associated foundation strength gain, will need to be evaluated at most soft soil sites.
Interim waste configurations	Most waste fills will go through numerous interim waste configurations prior to final closure; these configurations should be addressed in both the design drawings and operations plans. This requires in-depth communications and agreement between design engineers and owners/operators on the details of these configurations.
Operating phases	Over the life of a facility, operating conditions in the field may deviate from the original design—these deviations need to be identified by the owner/operator before they have the potential to impact waste fill stability. This requires the continuing involvement of a design engineer throughout the facility life.
Surcharging old waste fills	Placement of new waste on top of old unlined landfills must be approached cautiously; liquid conditions in the original fill must be understood as do the foundation conditions beneath the fill.
Field indicators of distress	Surface cracking and toe bulging of waste fill slopes may be indicators of marginally stable slope conditions.

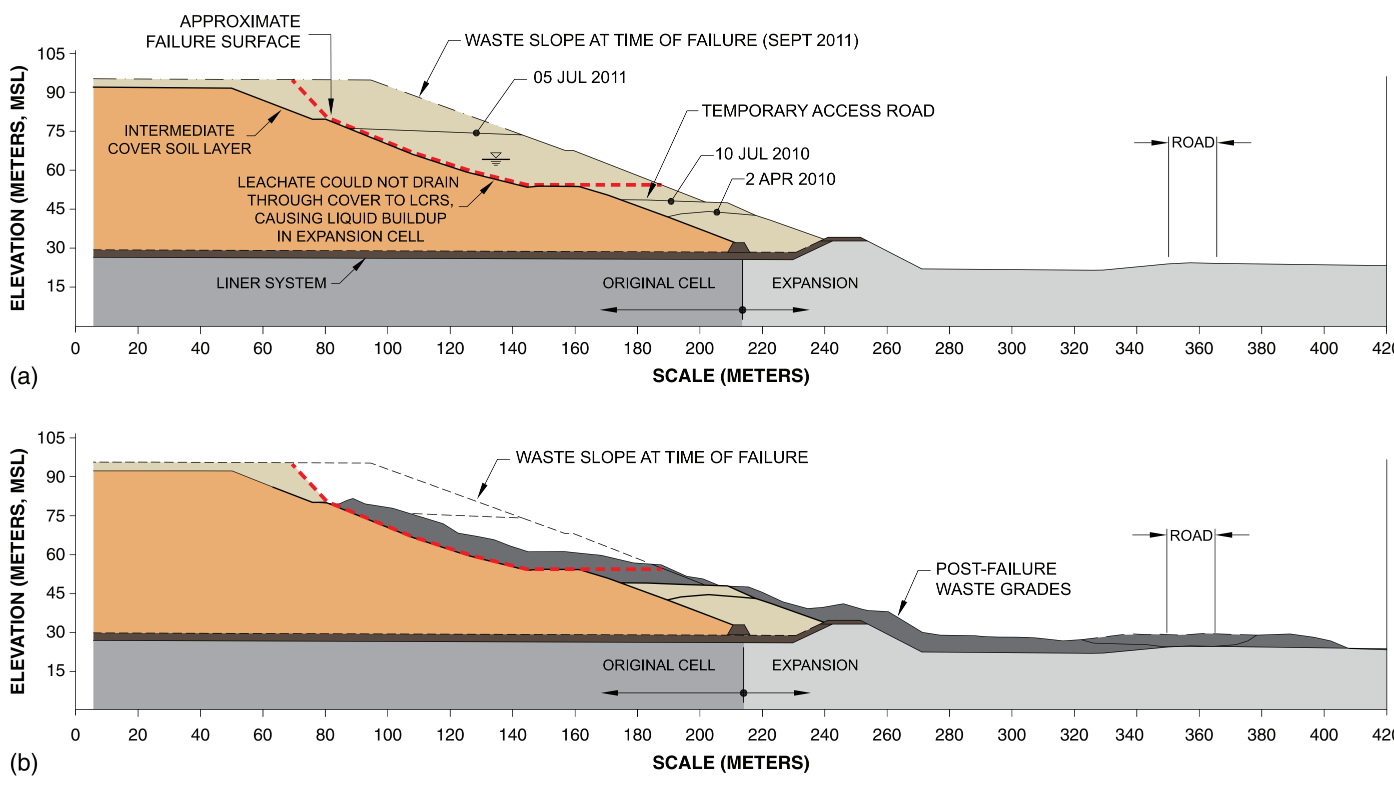


Fig. 13. (Color) (a) Prefailure; and (b) postfailure conditions for representative cross section through confidential eastern US MSW landfill. (Reproduced with permission.)

The waste fill failure occurred in September 2011, after the site received nearly 400 mm of rainfall in the prior month, including rainfall from Hurricane Irene and Tropical Storm Lee. In the weeks prior to the failure, waste filling was occurring near the top of the expansion area. The slide involved approximately 140,000 m^3 of waste and cover soil that flowed in a matter of minutes up to 150 m beyond the landfill boundary [Figs. 13(b) and 14].

After the failure, the root cause(s) were investigated, and recommendations developed for repairs and operational changes at the site. The investigation showed that the failure surface started about

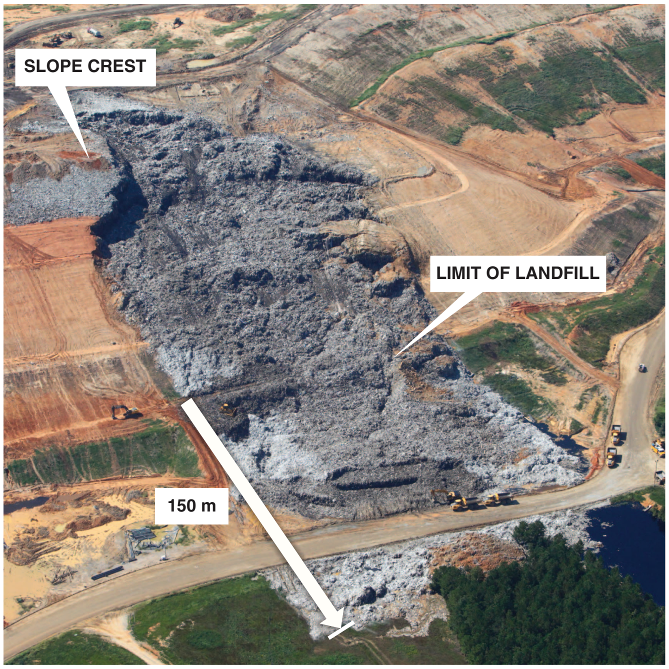


Fig. 14. (Color) Aerial view of landslide at confidential eastern US MSW landfill showing 150-m waste flow beyond landfill limit (from project files). (Reproduced with permission.)

Fig. 15. (Color) Aerial view of exposed intermediate cover soil layer in slide area at confidential eastern US MSW landfill (from project files). (Reproduced with permission.)

30 m behind the crest of the expansion area, extended through the waste at a steep angle, then followed the interface between the expansion area waste and underlying intermediate cover soil layer to about the midheight of the slope where it exited the slope horizontally through the expansion area waste, well above the top elevation of the expansion area's compacted-soil perimeter berm [Figs. 13(b) and 15].

On-site observations the day after the slide revealed leachate pools in the exposed waste, pockets of nonmixed soft sludge, gas escaping from the exposed waste (gas hissing sounds were heard), and evidence (e.g., matted vegetation) of leachate overland flow beyond the 150-m waste runout. As part of the investigation, 20 piezocone (CPTu) soundings were advanced outside, but in the vicinity of, the failed area approximately one month after the failure, in slopes with the same geometry and waste characteristics as the slope that had failed. Two partial CPTu logs are shown in Fig. 16. The logs present tip resistance, pore pressure response, and soil behavioral index (SBT) based on Robertson et al. (1986). CPTu 5 is from a midslope location close to the elevation of the toe of the slide. CPTu SC23 was advanced at a location about 30 m behind the head scarp of the slide on the landfill top deck.

The CPTu soundings showed that piezometric levels were elevated throughout the area. Fig. 17 shows the piezometric surface (in blue) obtained from areas adjacent to the failed area projected onto a cross section of the expansion area prior to failure. Slope stability analyses (2D) were conducted using the interpreted piezometric levels and the observed location of the failure surface to back-calculate an average mobilized MSW shear strength at the time of failure. The analyses were performed using Spencer's method (1967) as coded in the computer program SLIDE version 6.

The analyses resulted in an average drained waste friction angle of 26° (with a cohesion of 4.8 kPa). In addition, several samples of waste were obtained from the failure zone and shipped to Arizona State University (ASU) for strength evaluation using ASU's 0.46-m diameter direct shear box. The samples were provided in 19-L buckets and prepared for testing by reducing particle size to less than 75 mm, placing the material into the direct shear apparatus by hand in 150-mm-thick lifts, and then lightly tamping each lift. The testing was performed at the "as-received" moisture content of the waste. The composition of the samples was estimated to be about 75% MSW and 25% sludge by wet weight. Direct shear testing resulted in drained secant friction angles of about 24° and 20° at normal stresses of 69 and 138 kPa, respectively, at shear displacements of about 40–50 mm. These laboratory strengths are considered conservative (i.e., lower than the field strengths) owing to both sample disturbance and the orientation of the slip surface in the laboratory tests (horizontal) compared with the field (Fig. 17). Notwithstanding this conservatism, the results of the back analysis and direct shear tests, taken together, suggest that the shear strength of the waste in the expansion area was lower than the strength one might normally expect for compacted, sludge-free MSW at relatively low normal stress levels [e.g., strength relationships such as those proposed by Kavazanjian et al. (1995), Bray et al. (2008), and Stark et al. (2009)]. The relatively low strength of the waste (compared with typical MSW) and the elevated liquid and gas pressures in the waste were the root causes of this waste fill failure. The elevated liquid levels were the result of the operational practices at the facility described previously and the presence of the low-permeability intermediate cover soil layer that impeded vertical percolation of the leachate from the landfill expansion to the LCRS at the bottom of the original landfill. The elevated gas pressures were the result of watering in of many gas wells, accelerated biodegradation of the waste due to the presence of sewage sludge and excess moisture, and the physical conditions in the fill that prevented operation of the active gas management system in the area for five months prior to the failure.

Lessons learned from this waste fill failure include the following:

- Leachate recirculation and stormwater infiltration, if not adequately controlled and managed, can lead to the buildup

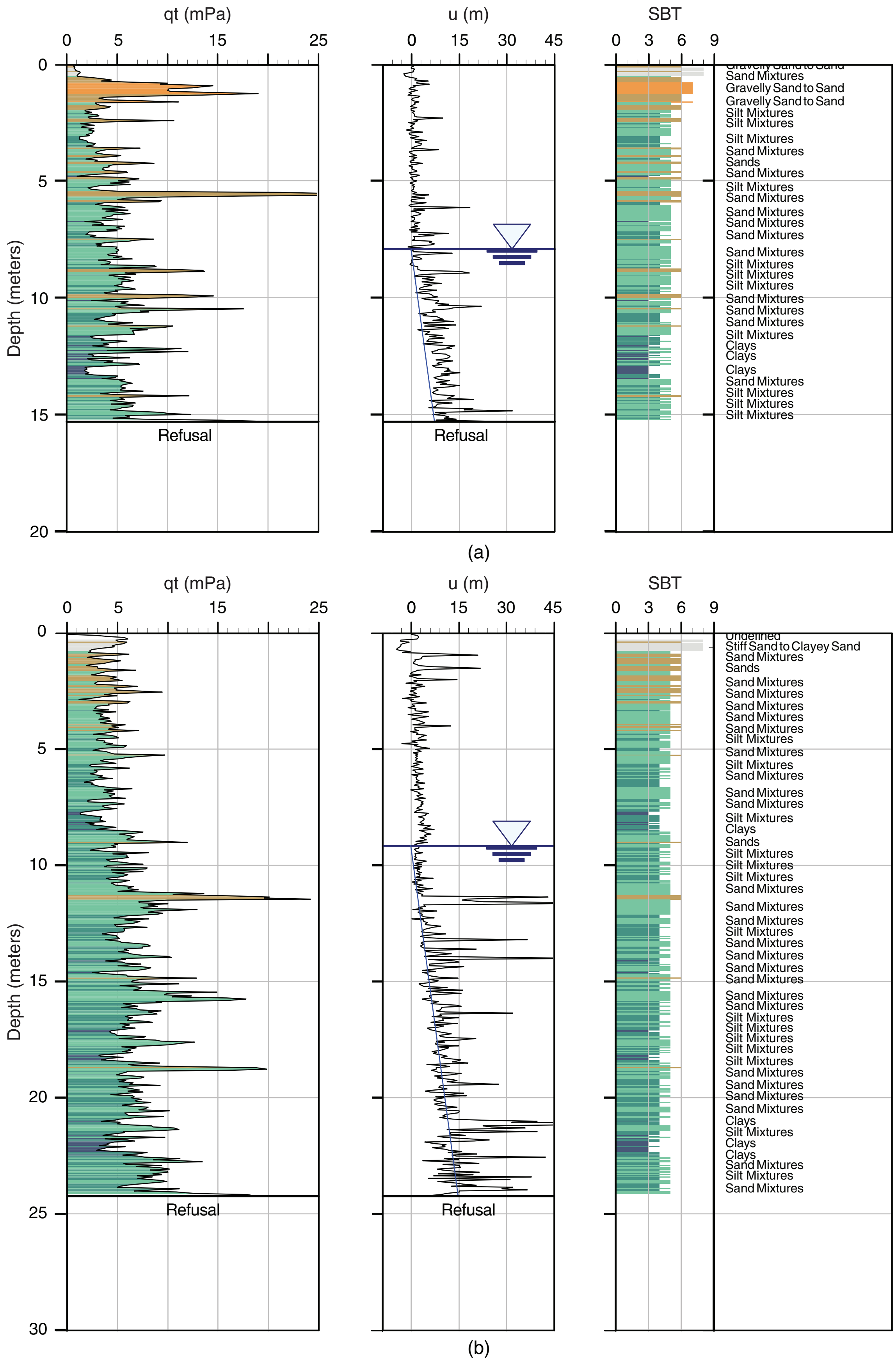


Fig. 16. (Color) Representative CPTu logs for areas adjacent to the slide zone: (a) midslope location; and (b) head scarp regions (from project files). (Reproduced with permission.)

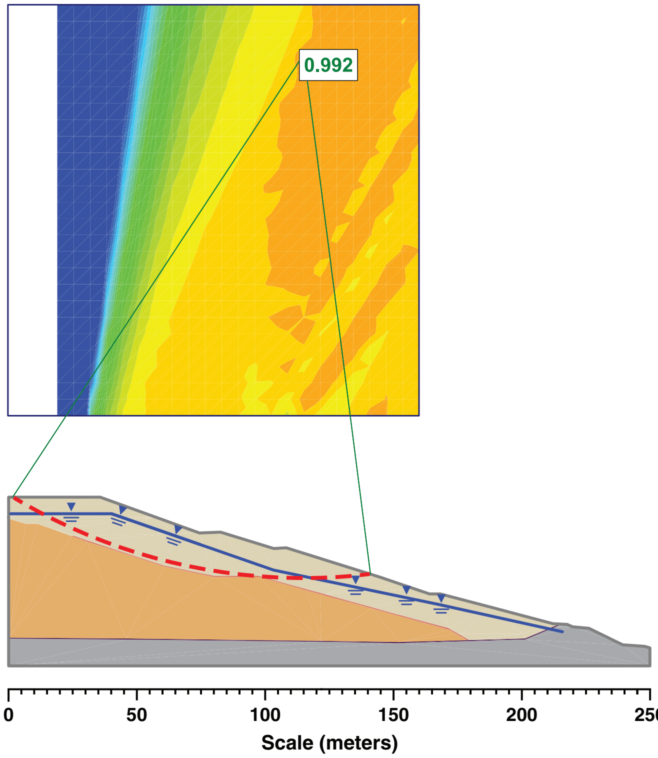


Fig. 17. (Color) Slope stability back analysis to investigate average operative shear strength of in-place MSW–sludge mixture at the time of failure (from project files). (Reproduced with permission.)

of elevated liquid levels and pore pressures in the waste. If leachate recirculation is to be practiced, it should be moderated and liquid buildup in the fill should be monitored. Enhanced internal drainage systems may be needed in some cases to achieve adequate drainage.

- The effects of sludge on the strength (decreases), permeability (decreases), and degree of saturation (increases) of the waste fill must be accounted for in design.
- Waste fill expansions that involve the placement of new waste over old need to account for the interface conditions. In this case, a low-permeability intermediate cover layer impeded leachate percolation from the expansion area to the LCRS at the bottom of the original landfill. Either the intermediate cover needed to be removed or breached, or a new LCRS placed on top of it.
- Gas collection efficiency can be greatly reduced in excessively wet landfills, both through operational problems such as the flooding of gas wells, and by the reduction of MSW gas permeability at increasing levels of waste saturation (e.g., Beaven et al. 2008; Stoltz et al. 2010).

Confidential Southern MSW Landfill

This facility is in the Coastal Plain physiographic province of the US Gulf Coast region. Average annual precipitation at the site is 1,400 mm. The site subsurface stratigraphy consists of a thin surface layer of tan silty clay and clayey silt overlying a thick deposit of overconsolidated high-plasticity clay. The upper weathered zone of the deposit is described as tan-yellow, stiff to very stiff, with numerous fractures and slickensides. It extends to an average depth below ground surface of about 10 m. Very stiff to hard, blue-gray unweathered clay underlies the weathered zone to a depth of about 120–150 m. The clay is calcareous and contains shell fragments, pyrite, and occasional slickensides. Soil index property testing of the clay resulted in an LL range of 90%–105%, PI range of 65%–80%, clay fraction range of 70%–80%, and in situ moisture content range of 35%–40%. The soil has a USCS classification of CH. The laboratory-measured vertical hydraulic conductivity of the unweathered blue clay is in the range of 1×10^{-11} m/s. Measured values for the weathered zone are in the range of 10^{-9}–10^{-10} m/s. Interpreted preconsolidation pressures for the weathered and unweathered material based on 1D laboratory consolidation testing are on the order of 150 and 1,500 kPa, respectively.

The landfill cell of interest is approximately 3.6 ha in size and is in a borrow soil area that was excavated in early 1996. Original ground surface elevations prior to borrow area development were about 119–122 m msl. The excavated bottom of the cell was at 102–105 m msl, so the excavation depth was in the range of 14–18 m. The excavation had 3H:1V slopes on the east, south, and north sides. On the west side, the excavation extended beyond the limits of the cell being discussed here, for future cell construction. At the west boundary of the cell, an intercell separator berm was constructed in the north–south direction. The berm was 3 m high and 11 m wide at the crest, with 3H:1V sideslopes. The bottom of the cell excavation was sloped at a 2% grade to a sump on the south side of the cell.

The liner system for the cell was not constructed until spring 2007, more than a decade after borrow area excavation. During that decade, precipitation fell into the open excavation and stormwater would at times pond in it. The liner system for the cell base and east and north backslopes consisted of a 0.6-m-thick native clay CCL installed directly on intact or recompacted native clay material. The CCL was overlain by a woven geotextile and 0.45-m-thick sand LCRS. This liner system was permitted as an alternative to the prescriptive composite liner system in EPA (1991) MSW regulations based on the thickness and low permeability of the native clay material. The liner system on the south backslope had a 1.5-mm-thick textured HDPE geomembrane replacing the CCL. Slope stability analyses conducted as part of the cell design utilized an effective-stress friction angle and cohesion for the native clay soil of 18° and 3 kPa, respectively.

Waste placement in the cell began around June 2007, and most waste was deposited within 18 months, by January 2009, with limited continuing filling in the cell until early 2010. The maximum waste elevation reached 137 m, resulting in an outboard waste slope on the western side of the cell about 29 m in height with a 4H:1V inclination (Fig. 18). The toe of this western slope was at an intercell berm (Fig. 19). The filled cell received an intermediate soil cover.

In October 2011, more than three months prior to the failure, a north–south oriented crack developed along the southeastern crest of the waste slope, along the alignment of a stormwater conveyance channel. The landfill operator sealed the crack by backfilling with cover soil; however, the crack periodically reopened, typically after rainfall events. Each time, the operator resealed the crack using cover soil.

In February 2012, roughly three years after the cell had been substantially filled and after several days of heavy rain, the operator observed that the crack had reopened and propagated northward along the entire eastern slope crest (Figs. 18 and 19). Concurrently, the main body of the waste fill (i.e., approximately 550,000 m^3) was observed to be slowly moving westward. Over the course of less than a week, the fill translated approximately 8 m to the west, resulting in an approximately 300-m-long, 20-m-wide (at the crest), and 10-m-deep graben on the cell's east side, as seen

Fig. 18. (Color) Slope failure at confidential southern US MSW landfill; translational movement was from east to west (photo from project files, taken on February 7, 2012). (Reproduced with permission.)

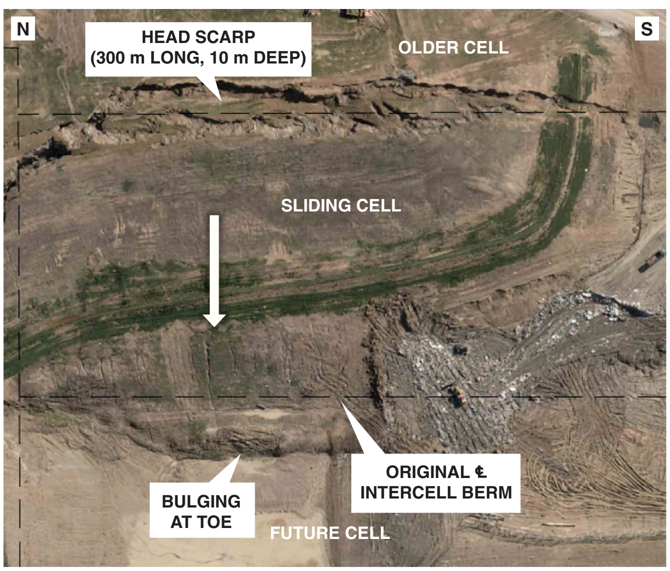


Fig. 19. (Color) Aerial view of confidential southern US MSW landfill showing several features of the waste fill slide; construction equipment at intercell berm provides sense of scale (photo from project files, taken on February 9, 2012). (Reproduced with permission.)

in the figures. Bulging was observed at the toe of the slope. The intercell berm on the west side of the cell translated to the west with the waste mass with the exit point for the slip surface being near the western toe of the berm.

Observations of the surface of the waste fill after the slide did not indicate rotation, tension/compression cracking, or significant distortion of the main portion of the slide mass. These observations support a translational sliding mechanism along a relatively shallow subhorizontal surface. After the failure, minor, episodic creep

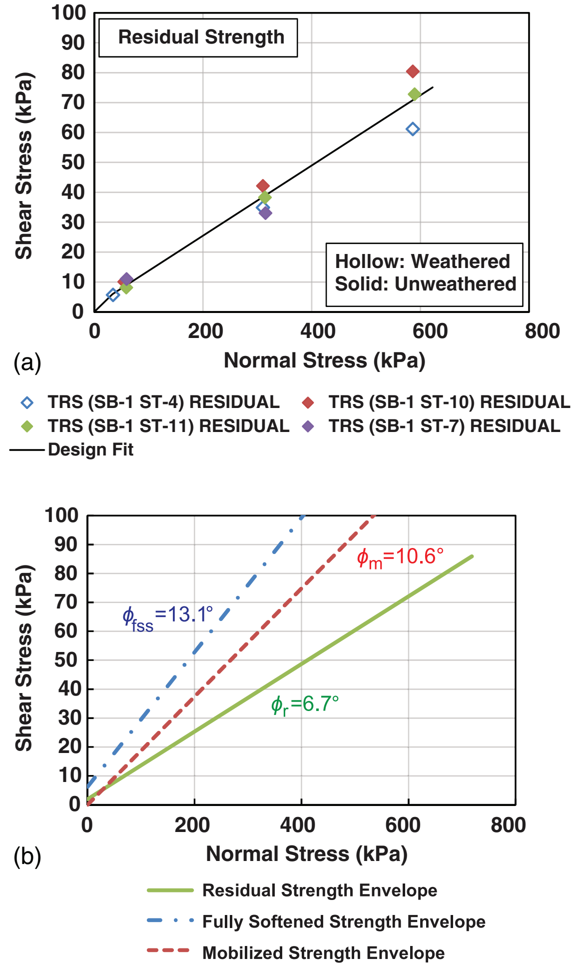
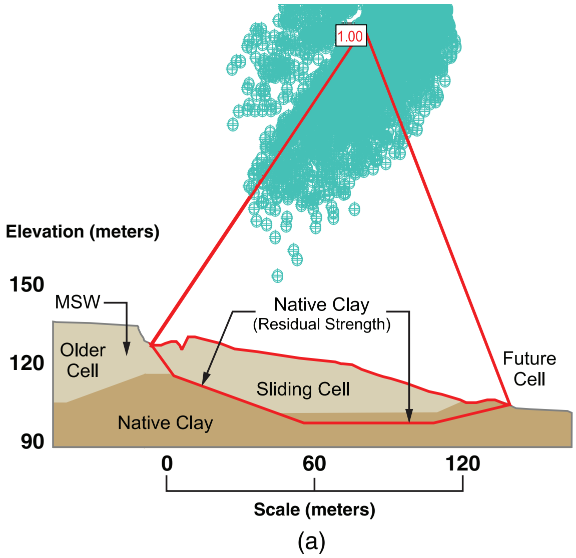


Fig. 20. (Color) Native clay strengths at confidential southern US MSW landfill: (a) residual strengths obtained from torsional ring shear (TRS) testing; and (b) residual (ϕ_r) and fully softened (ϕ_{fss}) shear strength envelopes from testing along with mobilized shear strength (ϕ_o) at incipient failure obtained from back analyses (from project files). (Reproduced with permission.)

movements occurred for several months at which point they stopped. In that timeframe, the operator constructed a soil buttress adjacent to the western intercell berm and waste was also placed at the toe to further buttress the waste fill.

An investigation was undertaken into the root cause(s) of the failure and recommendations were developed for the design and operation of future cells. Twenty CPTu soundings were advanced along with five soil borings. Thirty-four thin-walled Shelby tube samples were recovered from the boreholes. In addition, 14 vibrating wire piezometers were installed in waste at elevations a few feet above the CCL or in the native clay at elevations below the CCL. All but one of the piezometers in the native clay were installed close to but outside of the landfill footprint. The piezometers were installed in August 2012, roughly six months after the failure. The laboratory program included index, torsional ring shear, direct shear, and consolidated undrained triaxial compression testing. Torsional ring shear tests were conducted on remolded samples of the unweathered and weathered native clay yielding secant residual friction angles in the range of 6°–11°. An effective-stress residual strength envelope represented by a friction angle of 6.7° and a cohesion of 2 kPa was defined for use in subsequent slope stability analyses. Torsional and



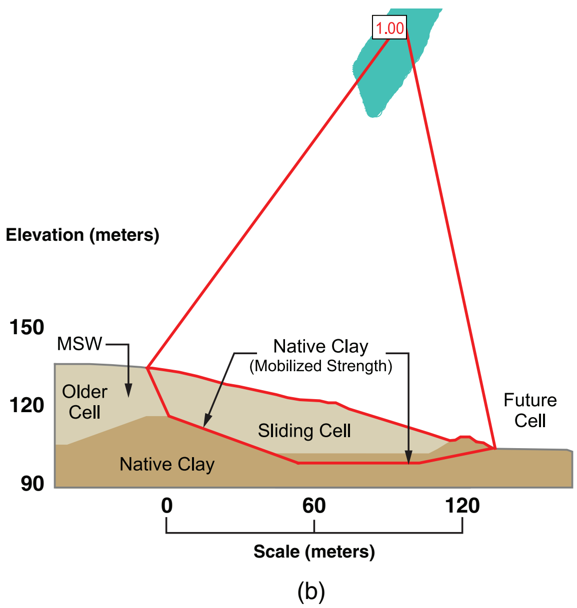


Fig. 21. (Color) Results of back analyses: (a) analysis using postfailure slope geometry and residual strength to obtain average operative pore pressure coefficient (r_u) along slip surface; and (b) analysis using prefailure geometry and average operative r_u to estimate the average mobilized shear strength at incipient failure (from project files). (Reproduced with permission.)

direct shear test results yielded an effective-stress fully softened strength envelope represented by a friction angle of 13.1° and a cohesion of 6 kPa (Fig. 20). Individual test results when interpreted as secant friction angles showed reasonable agreement with the correlations of Stark and Hussain (2013).

Pore pressures derived from measurements by the one piezometer installed within the cell footprint in native clay at a depth 1.2 m below the CCL bottom indicated essentially hydrostatic conditions between the piezometer elevation and the overlying sand LCRS above the CCL. Recall that the liner system across the bottom of the cell did not include a geomembrane and the native clay below the CCL had a permeability as low as or lower than the CCL. Thus, the LCRS served as a drainage boundary for the CCL and underlying native clay. Consolidation analyses conducted by the authors indicate that primary compression was 90% or more complete at a depth of 1–2 m below the bottom of the CCL at the time of failure.

No soil samples were obtained, and no slope inclinometers were installed, in the failure footprint below the liner system, so direct evidence of the depth of the sliding surface is not available. However, the depth was inferred to be less than a few meters below the bottom of the CCL based on field observations, the slide morphology (translational), the rationale that the native soil in proximity to the bottom of the cell excavation had undergone the most swelling and softening during the 10 years over which the excavation was open, and the results of parametric slope stability analyses. On the cell's eastern backslope, the peak and large-displacement shear strengths of the interface between the CCL and woven geotextile were found to be similar to the fully softened and residual shear strengths of the native clay, respectively. It is believed the slip surface in this area was at or near this interface.

To further evaluate the conditions at failure, 2D limit equilibrium analyses were performed, again using Spencer's method as coded in the computer program SLIDE version 5. Given the uncertainty in pore pressures on the slip surface at the time of failure, a two-part stability evaluation was conducted. An initial analysis was performed using the postfailure geometry of the waste fill (i.e., after 8 m of movement) and residual strengths along the base and eastern backslope of the cell [Fig. 21(a)]. An average pore pressure coefficient of 0.14 was back-calculated for FS = 1.0. The hydraulic head associated with this result is slightly higher than hydrostatic based on the LCRS providing a drainage boundary. A second analysis was then conducted using the prefailure geometry of the waste fill and the pore pressure coefficient of 0.14 from the previous analysis [Fig. 21(b)]. An average mobilized effective-stress secant friction angle of 10.6° was calculated for this condition (Fig. 20) suggesting that prior to the failure, the available shearing resistance along the slip surface had decreased from the fully softened value to a lower one as a result of shear displacements. When the accumulated displacements reduced the available shear resistance to a level corresponding to FS = 1.0, the slope failed. Note that if the prefailure analysis is repeated using the fully softened shear strength and a pore pressure coefficient of 0.14, FS = 1.26 is obtained.

The root cause for this waste fill failure was the lack of recognition of the effects of cell excavation, and the ensuing decade of soil rebound and water ingress, on the shear strength of the high-plasticity native clay. Excavation of the cell and unloading of the native clay induced negative pore pressures in the foundation. Availability of water over the ensuing decade allowed for infiltration by suction and along desiccation cracks and slickensides, resulting in softening and restructuring of the clay, with shear strengths trending towards the fully softened value. Filling of the cell with waste then generated excess pore pressures, relatively high shear stresses and low FS, resulting in shear displacements in the native clay that caused its shear strength to decrease. Although excess pore pressures were dissipating over time, continuing shear displacements further reduced the available soil shearing resistance, ultimately resulting in failure.

Lessons learned from this waste fill failure include the following:

- Engineers must have a thorough understanding of fundamental soil behavior, pore pressure response, and slope stability principles, including their application to the analysis and design of waste fills.
- When engaged to design a waste fill, engineers need to be careful to focus not only on the design of the waste fill itself and its

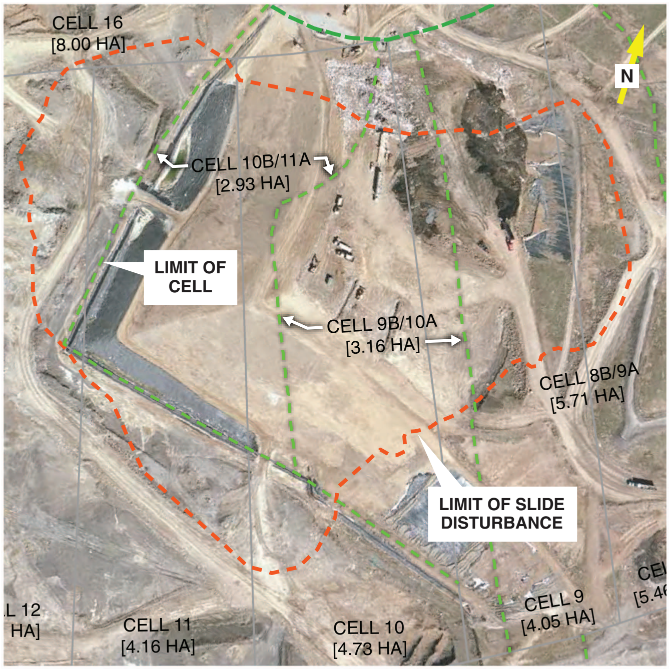


Fig. 22. (Color) Close-up view of confidential northeast US MSW landfill Cell 10B/11A prior to failure (April 2016) showing wedge-shaped slope face; note, light gray lines represent "as-designed" cell boundaries and green dashed lines represent "as-constructed" cell boundaries (from project files). (Reproduced with permission from Geosyntec Consultants, Inc.)

Fig. 23. (Color) Postfailure condition of confidential northeast US MSW landfill Cell 10B/11A, looking south; green truck cab in upper right portion of photo provides sense of scale (photo from project files, taken March 20, 2017). (Image by David J. Bonnett, reproduced with permission from Geosyntec Consultants, Inc.)

environmental protection systems, they must also carefully consider the potential for global stability issues in the context of site-specific geological, hydrogeological, and geotechnical conditions.

- In this specific case, the effects of a decade of unloading, water access, and softening on the shear strength of the native clay soil, and the associated potential for progressive failure, needed to be better understood.

Confidential Northeast MSW Landfill

This landfill site is in the Appalachian Plateau physiographic province in the US Northeast at a location receiving average annual precipitation of about 1,200 mm. The landfill area that failed in February 2017 was a new lined cell (Cell 10B/11A) that started operations in January 2015, two years prior to the failure. A double-liner system was installed across the base of the cell and the 3H:1V perimeter berm sideslopes. It consisted of a 1.5-mm-thick HDPE secondary geomembrane liner overlain by a geocomposite secondary LCRS, in turn overlain by a composite primary liner (0.6-m-thick CCL overlain by a 1.5-mm-thick HDPE geomembrane), geotextile protection layer, and 0.3-m-thick granular primary LCRS. The intermediate cover slope of the previous cell (Cell 9B/10A), which formed the backslope of the cell of interest, was inclined at about 2.5H:1V. The new cell had a wedge-shaped perimeter, with northern and eastern legs coming together to form a roughly 90° angle at the point of the wedge. As the cell was filled, this resulted in westward and southward facing exterior cell slopes (Fig. 22). Waste was placed in lifts across the base of Cell 10B/11A and up against the intermediate cover slope of the previous cell.

Materials deposited in the cell included MSW and construction and demolition (C&D) waste, high moisture content municipal sewage sludge (biosolids) and oil and gas exploration and production wastes (including drill cuttings), and lesser amounts of other materials. The postfailure forensic investigation (discussed in the following text) found pockets of the high moisture content waste in the failed waste fill. This material was described as low shear strength waste (LSSW) by the investigators and later as high-moisture waste (HMW) by Bareither et al. (2020). With only a few exceptions, retrieved MSW samples had moisture contents in the range of 20%–100%, whereas the moisture contents for LSSW samples were in the range of 50%–240%. Of note, the intermediate cover soil layer of Cell 9B/10A consisted of drill cuttings mixed with lime. This created a relatively hard, smooth, and impervious surface separating the cell where the failure initiated (Cell 10B/11A) from the previous cell (Cells 9B/10A) where the failure retrogressed. The facility operator scarified the surface of the cover layer prior to placing waste against it but did not breach or bench into it.

At the time of failure, the 3H:1V outboard waste slopes had reached a height of about 50 m, with slope benches at about 20- to 25-m vertical intervals. At that time, the active working face was on a plateau in an area set back roughly 70–80 m from the outboard slope face. The failure involved a 6-ha portion of the landfill, with sliding occurring in both the westward and southward directions over a period of about 10 min, displacing approximately 170,000 m^3 of waste and cover soil (Figs. 23 and 24). The slide material liquefied, moving up to 75 m past the landfill perimeter berm, and covering an area outside the berm of about 2 ha. Neither the landfill foundation nor liner system were involved in the failure.

In the weeks leading up to the failure, landfill personnel observed surface cracking and anomalies within the active area. Gas was heard (hissing) escaping from some of the cracks. The personnel documented the depths and widths of the cracks, filled

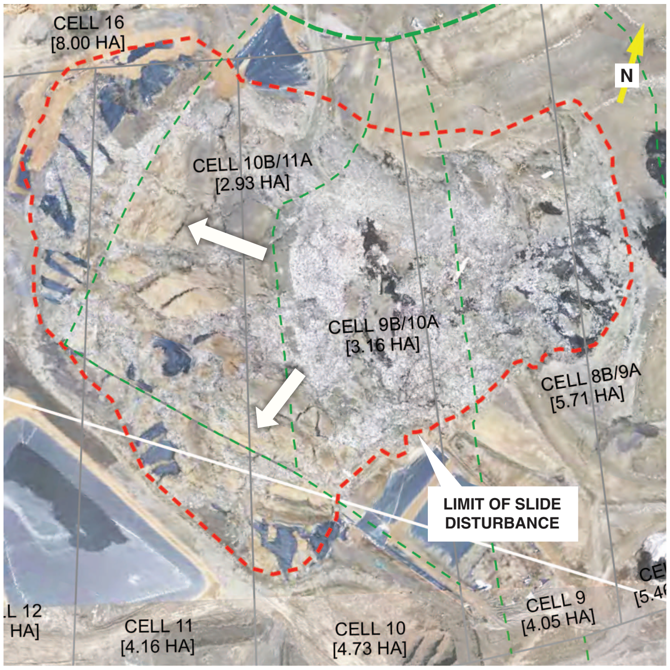


Fig. 24. (Color) Aerial view of confidential northeast US MSW landfill showing waste fill slide area and cell configurations (from project files). (Reproduced with permission from Geosyntec Consultants, Inc.)

them in, and monitored them in accordance with site procedures. Areas of waste subsidence and heaving were observed in the active area of the cell; at the time, these were not interpreted by site personnel as a sign of developing slope instability, perhaps because the area in which the subsiding and heaving were occurring was well back from the slope face. In response to site observations, the landfill operator hired a contractor to install landfill gas extraction wells in the cell to relieve gas pressure. The contractor's efforts to drill the wells to depth were unsuccessful in some cases due to shallow refusal and squeezing of sludge into the well bores. In the days prior to the failure, the landfill operator also monitored the movement of several survey pins, 10 of which were in the area that failed. In the few days prior to the failure, horizontal movements were in the range of 1.5–4.0 m, with vertical movements in the range of 0.1–1.4 m. In the hours immediately preceding the failure, survey data indicated pin movements were accelerating.

At incipient failure, a site employee observed bulging approximately one quarter of the way up the south-facing slope. The bulge burst, releasing leachate and triggering a retrogressive failure back into the waste mass. At about the same moment, a second bulge on the west-facing slope of the cell also burst, releasing leachate and initiating waste flow in a similar manner. Blocks of intact waste were observed to have ridden on the liquefied portions of waste that traveled in the southerly and westerly directions.

On behalf of the facility owner, the authors' firm, partnered with Professor Craig Benson of the University of Virginia, conducted a field and laboratory investigation of the root cause(s) of the slope failure. Nine bucket auger borings and nine sonic borings were advanced starting in June 2017, approximately four months after the failure. Bulk waste samples were obtained from both boring types. Ninety CPTu soundings were conducted at 46 locations in June, with 15 additional soundings in September. In addition, 29 piezometers were installed along with 11 inclinometers around the perimeter of the failed area. Two of the piezometers and three shape accel arrays (SAAs) were installed within the slide area to a depth terminating above the elevation of the liner system. The laboratory testing program involved evaluation of waste sample moisture and organic matter contents, volatile solids, shear strength, and hydraulic conductivity. The program included composited samples each of MSW and LSSW, plus mixtures of the composited MSW and LSSW samples. Results for some the laboratory analyses are presented in Bareither et al. (2020). Fig. 25 presents two partial CPTu sounding logs from within the footprint of the slide mass. CPTu 20B is from near the center of the slide. The presence of significant amounts of LSSW at this location is reflected in the very low tip resistance between about 3 and 8 m depth and at 10 m depth. The LSSW SBT classifies this as sensitive fines (Robertson et al. 1986). CPT 23B is from an area closer to the top of the slide. The relative absence of LSSW in this sounding is apparent.

The postfailure investigation included an evaluation of the design details and operating conditions that might have contributed to the failure. Specifically, as with the earlier eastern US case study, the placement of an intermediate cover soil layer was found to have impeded vertical percolation of liquids through it, resulting in perched liquid buildup in the waste fill. This cover soil layer was also a barrier to gas flow to gas extraction wells that were installed in the previous cell. A review of the records for Cell 10B/11A indicated that a second low-permeability surface was created in the cell when an earthen segregation layer was installed over an area where special industrial waste had been deposited that had the potential to generate hydrogen sulfide gas. The segregation layer is described as relatively impervious and as having inhibited leachate drainage and gas movement in the cell. The presence of LSSW was another factor leading to low permeability and high levels of saturation within the area of the landfill that failed. Taken together, these factors led to a buildup of liquid and gas pressures in the fill. The buildup of liquid is evidenced by the fact that the day of the failure, the landfill leachate generation rate tripled from the previous day's rate, and it remained at more than twice the previous rate for the following six days. This leachate was perched in the fill and only able to drain after the failure disrupted the low-permeability layers.

The investigation revealed that the landfill operator had established a 30-m setback from the south- and west-facing exterior slopes for LSSW disposal. The goal of the setback was to reduce the potential for this wet, low strength material to create leachate surface seeps or adversely affect slope stability. This can be an effective operating strategy in many cases, but in this case, it had the unintended consequence of concentrating the LSSW in a relatively small portion of the interior of the cell. The relatively small disposal area was in part a consequence of the wedge-shaped cell face, which resulted in a progressively smaller disposal footprint with increasing slope elevation. In addition, much of the non-LSSW waste was used up in constructing the 30-m wide, LSSW-free wedge. The forensic investigation yielded zones of high LSSW content that could be correlated back to the interior, upper portions of the failed cell.

The effects of high LSSW content on waste shear strength are observed in the results of direct shear tests performed on samples of waste material from the failed waste cell containing a range of LSSW contents. The testing was performed at Colorado State University (CSU) using a shear apparatus with an internal diameter of 280 mm. Fig. 26 provides photographs of the MSW and LSSW samples at the CSU laboratory. Test samples were shipped from the site to CSU in sealed buckets, composited, blended using specific ratios on a total mass basis at as-received moisture contents, placed in the test apparatus in lifts and lightly tamped, compressed under the applicable vertical compressive stress for 10 min, and

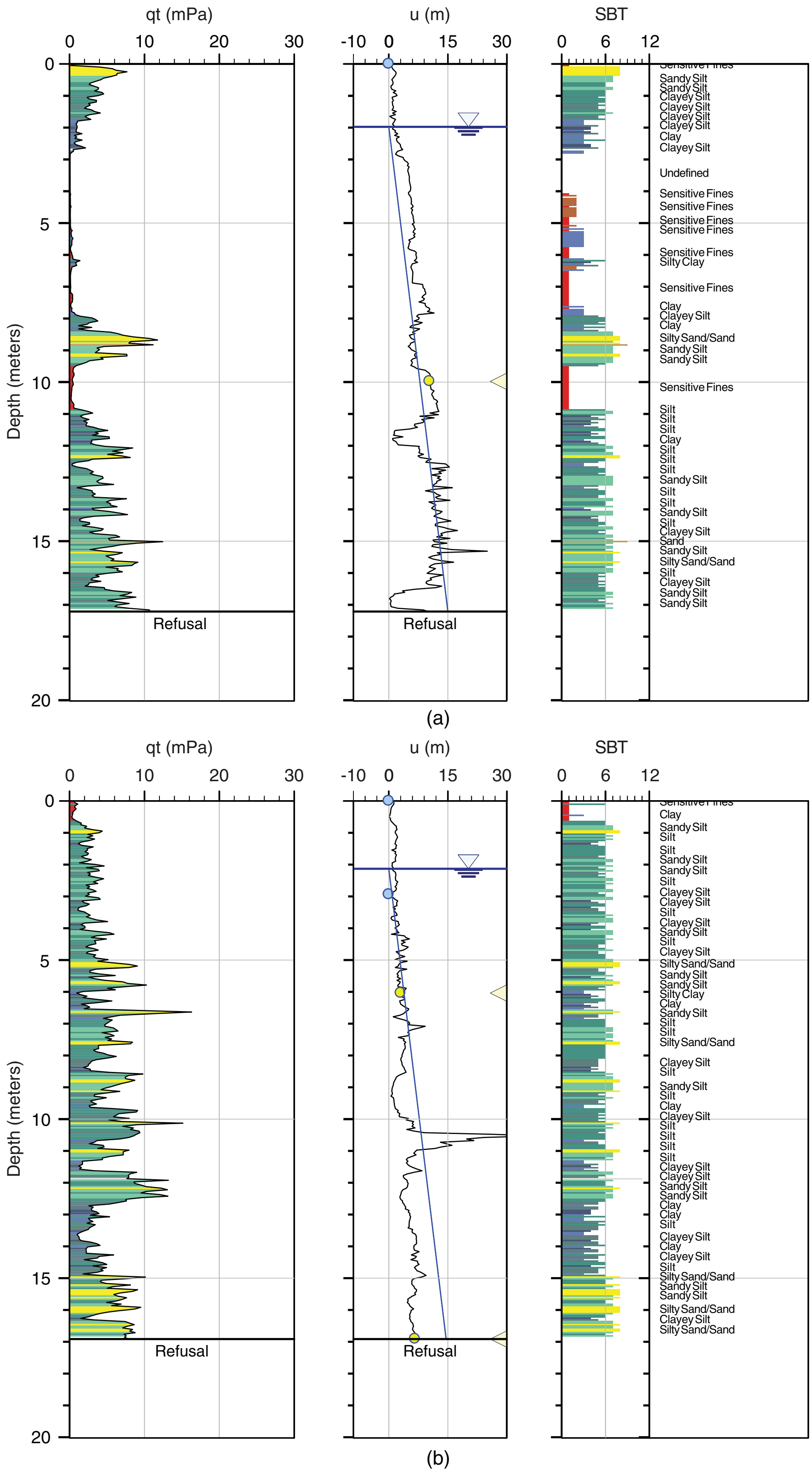


Fig. 25. (Color) Representative CPTu logs for areas within the slide zone: (a) midslide showing substantial amounts of LLSW; and (b) top of slide showing relative absence of LSSW (from project files). (Reproduced with permission from Geosyntec Consultants, Inc.)

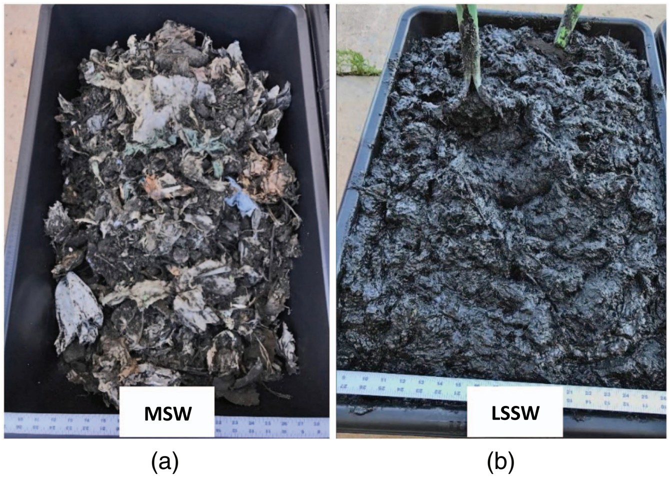


Fig. 26. (Color) Sampled waste materials from Greentree Landfill: (a) degraded MSW; and (b) LSSW. (Images by Craig Benson, with permission.)

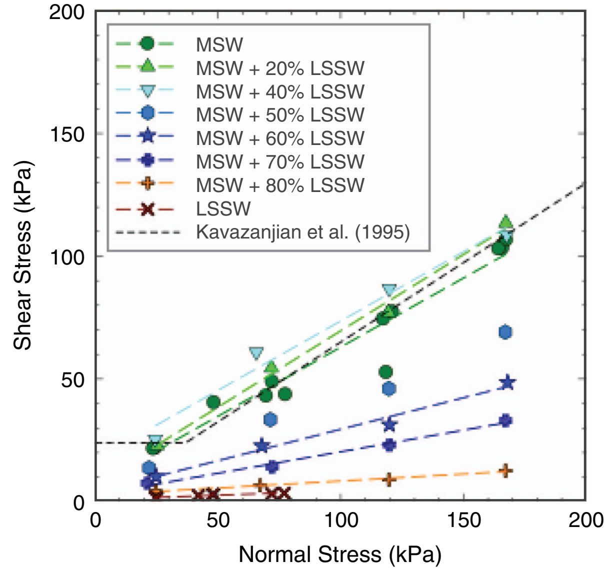


Fig. 27. (Color) Direct shear test results for MSW, LSSW, and mixtures (wet weight) of MSW and LSSW; testing conducted at Colorado State University. (Reproduced with permission from Geosyntec Consultants, Inc.)

then sheared following the procedures described in Bareither et al. (2012). The short compression time (10 min in this case) is not uncommon when testing typical MSW. In this case, it was found that at LSSW contents less than about 50%, primary compression of the samples was complete prior to shearing and the conditions during shear can be considered drained. At higher LSSW contents, however, the samples were still consolidating at the start of shearing and the tests are considered as only partially drained and potentially undrained for the samples at very high LSSW content. Shear strength test results are shown in Fig. 27. The results for LSSW contents below 50% reflect a drained response of the MSW/LSSW mixture while the results at higher moisture contents reflect partially drained to undrained strength conditions. As can be seen in the figure, for LSSW contents up to about 40%, the measured shear strengths of the blended samples are about what would be expected for MSW alone, as reflected in the MSW shear strength relationship proposed by Kavazanjian et al. (1995) that is also shown on the figure. At higher LSSW contents, the shear strengths of the blended samples drop markedly. When the samples consisted mostly of LSSW, the measured shear strengths were very low. It is not clear from the test results how much of the strength reduction at high LSSW contents is due to the properties of the LSSW versus the samples having a partially drained to undrained response in the tests. It is clear, however, that the LSSW is a low shear strength material compared with typical MSW. Moreover, the authors of this paper note that in their experience, reductions in shear strength can sometimes occur at sludge (e.g., LSSW) contents in the range of 25%–30%. Note too that several additional tests were performed on samples that were soaked prior to shearing; relatively small reductions in strength were observed for soaked samples at LSSW contents of 40% or less, with little to no difference in strengths for higher LSSW contents.

Slope stability analyses were conducted to evaluate the effects of waste strength and landfill liquid levels on the FS of the failed slope. For the analyses, the shape (translational) and location of the slip surface was approximated based on the slide exit location at the slope face, postfailure observations of a zone of waste about 12-m below ground surface described as "dark, with little to no shear strength," and measurements from an inclinometer installed near the back edge of the landside that clearly identified the depth of sliding at that location. The analysis results showed that a range of combinations of MSW/LSSW waste shear strength values (in the interior of the cell behind the 30-m setback), MSW strengths in the 30-m setback based on Kavazanjian et al. (1995), and estimated liquid and/or gas heads in the cell would have resulted in FS = 1.0. Clearly, the root causes of this failure were (1) the low shear strengths of portions of the waste fill due to the concentrated disposal of LSSW in the interior of the cell behind the 30-m setback, (2) the buildup of liquid and gas pressure in the cell due to the low permeability of the waste mass resulting from its high LSSW content, (3) the barriers to liquid and gas movement created by the intermediate cover soil layer and segregation layer, and (4) the insufficient number and of depth of gas extraction wells to effectively remove landfill gas from the cell. Based on observations prior to and after the failure, elevated gas pressures existed in the interior of the cell prior to the failure.

Lessons learned from this waste fill failure include the following:

- Special (i.e., non-MSW) wastes can create operational problems such as reducing the shear strength of the waste fill, reducing its permeability, increasing its degree of saturation, accelerating its rate of biodegradation, and possibly increasing its temperature.
- Special wastes, if placed at too high a mass fraction and if not thoroughly mixed with MSW or other stronger material, can create weak zones that have the potential to adversely affect waste fill stability.
- Low-permeability zones in the waste (from special wastes, intermediate cover soil layers, odor control layers, etc.) can trap liquids and gases, potentially leading to elevated fluid pressures. In this case, the intermediate cover soil layer needed to be breached or removed, or a new LCRS placed on top of it. The effects of the segregation layer also need to be recognized along with the low-permeability characteristics of the LSSW.
- Areas of high special waste content can impede both installation and operation of active gas extraction wells, as was observed in this case study.

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Table 4. Lessons learned in 2010–2019 timeframe regarding geotechnical stability of waste fills

Category	Lesson learned
Communication among parties	Failure of construction and operating personnel to communicate field deviations from the design back to the design engineer can lead to problems.
Properties of special wastes	It is essential that the properties (i.e., total unit weight, permeability, moisture content, compressibility, and shear strength) of special wastes and special waste/MSW mixtures be reliably evaluated or conservatively estimated for design. Note that such evaluations need to consider that material properties are moisture-, stress-, and degradation-dependent.
Leachate recirculation	Leachate recirculation to accelerate waste biodegradation, if not properly managed and controlled, can create conditions adversely affecting waste fill stability. These include buildup of liquid heads in the waste fill, saturation of the waste, watering in of gas wells, reduction in waste permeability, buildup of gas pressures, and increasing landfill temperatures.
Challenges with codisposal of special wastes	Codisposal of special wastes with MSW, in the absence of adequate engineering and operational controls, can lead to geotechnical stability problems.
Vertical and lateral expansions	The cost and permitting challenges in developing new greenfield waste sites is leading to the increasing use of vertical and lateral expansions at existing waste sites. Vertical expansion configurations and materials, in the absence of adequate engineering and operational controls, can contribute to waste fill stability issues.
Challenges associated with waste saturation	Waste fill gas permeability and gas well collection efficiency are both substantially diminished in very wet waste fills. These landfills have an increased potential to experience elevated temperatures and the issues associated with them.
Personnel awareness	Surface subsidence and movements, excessive gas releases or leachate slope seepage, difficulty advancing gas wells due to borehole collapse, slope bulging, and slope creep, if observed, should be promptly evaluated as possible indicators of deteriorating conditions in the waste fill.

- Measures to mitigate conditions in the waste fill need to be carefully thought through to identify any unintended consequences of the measures. In this case, the 30-m setback requirement for LSSW placement led to weak zones (high LSSW content) in the interior of the cell, and the placement of the low-permeability segregation layer impeded leachate and gas movements within the cell.

Looking at the waste fill failures that have occurred in the timeframe 2010–2019, challenges remain with respect to reliably achieving waste fill stability in certain situations. Some of these challenges relate to the failure of design engineers and facility owner/operators to remember the lessons that were learned 20–30 years ago, while at the same time there are new lessons to be learned related to evolving waste streams and operating practices. Several specific examples of these newer lessons are given in Table 4. Table 5 presents several recommendations to address these newer lessons learned.

Standard-of-Care for Waste Fills Stablility Assessments

The standard-of-care exercised by engineers working in a specific engineering discipline can be defined as (e.g., Lucia 2012) "that level of skill and competence ordinarily and contemporaneously demonstrated by professionals of the same discipline practicing in the same locale and faced with similar facts and circumstances." Fig. 28 from Silva et al. (2008) has been adapted herein to consider the standard-of-care for slope stability design of waste fills in the United States and to compare that standard with those for earth dams and other geotechnical structures.

Fig. 28 shows four relationships developed by Silva et al. between annual probability of failure on the vertical axis and factor of safety on the horizontal axis. The relationships were developed based on geotechnical stability evaluations involving earth dams, natural and cut slopes, and earth retaining structures. The four relationships correspond to differing "levels of engineering," defined by Silva et al. as

1. "Category I–facilities designed, built, and operated with state-of-the-practice engineering. Generally, these facilities have high failure consequences;
2. Category II–facilities designed, built, and operated using standard engineering practice; many ordinary facilities fall into this category;
3. Category III–facilities without site-specific design and substandard construction or operation; temporary facilities and those with low failure consequences often fall into this category; and
4. Category IV–facilities with little or no engineering."

Note that these definitions tie the level of engineering to the consequences of failure, i.e., the implicit engineering standard-of-care in these relationships is dependent on the consequences of failure. The larger the consequences, the better the level of engineering required to meet the standard-of-care. Silva et al. provide annual failure probabilities using their chart of 1×10^{-6} and 1×10^{-4} for earth dams designed, constructed, and operated to Category I and Category II levels of engineering, respectively, and an FS of 1.5.

Most modern US MSW landfills are designed to a minimum static factor of safety of 1.5. The authors have calculated an average annual probability of failure for these facilities based on their knowledge of the frequency of actual failures in the 2010–2019 timeframe and the number of operating MSW landfills in the United States. The results of this calculation yield an average annual failure probability on the order of 1×10^{-3}. Inspection of Fig. 28 shows that the average US MSW landfill has a 10 times higher annual failure probability than an earth dam designed to a Category II level of engineering. This result also suggests that the design of MSW landfills in the United States falls between the Category II and Category III levels of engineering. The authors have added range bars to the MSW landfill point in Fig. 28 to recognize that some facilities will have a higher failure probability than the average (i.e., lower level of engineering), but probably not too much higher owing to regulatory requirements and enforcement, and some will have failure probabilities as low as or lower than the average earth dam owing to the quality of their design, construction, and operation. The authors also show, based on their judgment, a very low failure probability for US Department of Energy low-level radioactive waste (LLRW) disposal facilities that they and their colleagues have designed. Very high standards apply to the design and construction of these facilities owing to their long

Table 5. Recommendations to address lessons learned in 2010–2019 timeframe

Lesson	Recommendations
Communication among parties	• Owners/operators should continue the involvement of the design engineer throughout the operating life of their facilities. • Design engineers should be proactive in educating owners/operators of the problems that can occur as a result of engineer disengagement and the benefits of keeping the engineer engaged. • Field deviations from the design should be communicated by the owner/operator and checked by the design engineer prior to being implemented.
Properties of special wastes	• Project-specific waste testing is recommended for waste fills that will receive appreciable quantities of special wastes. • Disposal limits for special wastes (e.g., maximum special waste percentage by wet weight) should be established as should acceptable operational practices for special waste placement (e.g., setback from the slope face, placement in lifts separated by lifts not containing special wastes, solidification requirements). • The potential for special wastes to degrade via exothermic pathways should be assessed to evaluate the potential for these wastes to contribute to elevated waste fill temperatures.
Leachate recirculation	• Recirculation rates need to be moderated so as not to saturate the waste and generate elevated piezometric levels. • Landfill internal drainage features may need to be enhanced when recirculation is used. • The placement of low-permeability daily/intermediate cover soil layers or waste layers should be evaluated carefully to assure they will not impede the flow of leachate within the waste fill. • Recirculation rates should be based on a water balance analysis of the waste fill: during operations, water input and outputs from the fill should be monitored and the water balance kept up to date.
Challenges with codisposal of special wastes	• Special waste acceptance plans (SWAPs) should be developed for each special waste stream. • SWAPs should address potential impacts of the special wastes on leachate and gas generation rates, waste properties, slope stability, and operations. • SWAPs should set limits on the acceptable ratio of special wastes to MSW/C&D wastes and provide requirements as appropriate for pretreatment, mixing, and placement. • Owners/operators should ensure that field personnel follow the SWAPs and these personnel should remain vigilant. • Unintended consequences of special operating procedures should be carefully considered.
Vertical and lateral expansions	• The intermediate cover interface between the vertical expansion and original waste fill should be carefully engineered for stability and permeability. • In some cases, the cover should be removed, or at least breached. • In other cases, a new LCRS should be installed on top of the cover. • The effects of a vertical expansion on both the geotechnical stability of the original waste fill and the rate at which gases and liquids in the original fill flow to drainage or extraction points should be carefully considered; gas and liquid flow in the original waste fill may decrease owing to the weight of the vertical expansion and compression of the original fill; this compression will increase the density and degree of saturation of the original fill while reducing its permeability.
Challenges associated with saturated waste	• Waste fills that generate appreciable amounts of decomposition gas (e.g., MSW landfills) should be maintained at saturation levels low enough to allow adequate rates of gas flow to extraction wells and venting layers (e.g., degree of liquid saturation less than about 50%–60% for MSW landfills). • When needed, internal drainage features can be installed in the waste fill (e.g., chimney drains, tiered vertical gas wells, horizontal drainage trenches) to improve liquid drainage from the fill, thereby limiting moisture buildup in it. • Waste fill operators should be alert to decreasing gas production rates and the watering in of gas wells as indicators of high levels of waste saturation. Early action can be helpful to mitigating developing issues before they become serious.
Personnel awareness	• Engineers designing solid waste fills need to understand the types of wastes that will be placed in the waste fill over its operating life, the characteristics and physical properties of those wastes, how they degrade, and how the waste fill facility will be operated (e.g., leachate recirculation, operator filling practices); this requires that the engineer inquire with the facility owner/operator regarding all these topics. • Personnel working at landfills already undergo significant training to perform their jobs; it is recommended that owner/operators now make sure the training includes early identification of potential warning signs of deteriorating slope conditions and the necessity of reporting those conditions promptly to the owner/operator and engineer.

design life (at least 200 years) and substantial consequences of failure.

In comparing the consequences of failure of a Category II earth dam to a US MSW landfill (between Category II and Category III), both have the potential for human injury or death, with larger potential human consequences more typical of an earth dam failure. In terms of the financial and environmental consequences of failure, they can be substantial for both earth dams and MSW landfills. For example, based on the authors' experience, the consequences of a US waste fill failure in terms of economic loss can be very substantially more than $25 million. The consequences of failures that have occurred in developing countries in the decade 2010–2019 far exceed those of US failures and there is an urgent need to improve engineering, construction, and operating practices in these countries.

The takeaway question from Fig. 28 is whether the current standard-of-care for design, construction, and operation of US waste fills is adequate given the potential consequences of failure, or whether the profession should attempt to elevate the standard through consistent application of at least Category II design,

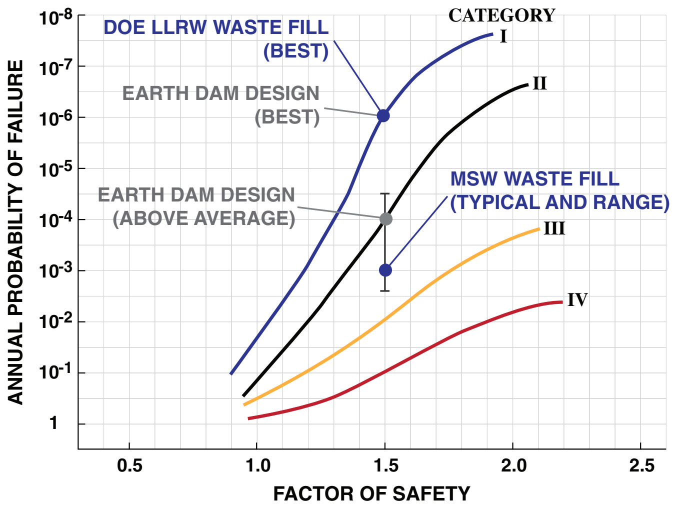

Fig. 28. (Color) Annual probability of failure versus factor of safety for categories of engineering: earth dam failure probabilities shown on plot are from Silva et al. (2008); waste fill failure probabilities are from the authors.

construction, and operations processes. The authors advocate for this and note that it would require design engineers and facility owners, who in the authors' experience are all committed to the development of environmentally protective and structurally safe facilities, to follow the lessons described in this paper.

Data Availability Statement

Some or all data, models, or code generated or used during the study are proprietary or confidential in nature and may only be provided with restrictions from the corresponding author. These include existing data for the three confidential case studies described in the paper.

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