The July 10 2000 Payatas Landfill Slope Failure

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ABSTRACT: *This paper presents an investigation of the slope failure in the Payatas landfill in Quezon City, Philippines. This failure, which killed at least 330 persons, occurred July 10th 2000 after two weeks of heavy rain from two typhoons. Slope stability analyses indicate that the raised leachate level, existence of landfill gas created by natural aerobic and anaerobic degradation, and a significantly over-steepened slope contributed to the slope failure. The Hydrologic Evaluation of Landfill Performance (HELP) model was used to predict the location of the leachate level in the waste at the time of the slope failure for analysis purposes. This paper presents a description of the geological and environmental conditions, identification of the critical failure surface, and slope stability analyses to better understand the failure and present recommendations for other landfills in tropical areas. In addition, this case history is used to evaluate uncertainty in parameters used in back-analysis of a landfill slope failure.*

KEYWORDS: Landfills, Failure investigations, Slope stability, Shear strength, Gas formation, Leachate, Leachate recirculation, Pore pressures.

SITE LOCATION: IJGCH-database.kmz (requires Google Earth)

INTRODUCTION

At approximately 4:30 am Manila local time (MLT) on July 10th 2000, a slope failure occurred in the Payatas Landfill in Quezon City, Philippines. The slope failure killed at least 330 people (Gaillard and Cadag, 2009) and left many homeless because of the close proximity of homes to the landfill slope. News reports indicate the landfill had been initially 18 to 40 m (60 to 108 ft) high with steep side slopes and had been subjected to torrential rain from two recent typhoons. A reconnaissance team, including the third author, traveled to the site to observe the waste slide area and collect pertinent information. Merry et al. (2005) and Kavazanjian and Merry (2005) summarize the observations and efforts of the reconnaissance team. This paper describes characteristics of the Payatas landfill, the slope failure, landfill leachate and gas evaluations, stability analyses, sensitivity analyses of the back-calculated parameters that were used to better understand the case history, and presents recommendations for analysis of landfills in tropical areas.

DEVELOPMENT OF CONDITIONS AT PAYATAS LANDFILL (CIRCA 2000)

The Payatas Landfill, located in the northeast corner of Quezon City in Philippines (see Figure 1), is shown in Figures 2 and 3. Quezon City has a population of about three million people and is the largest city in the Metro Manila area. The Payatas housing development started in the early 1970s when about 30 large upscale housing developments were proposed. For the initial development stage, the Payatas area was not supposed to be the site of a landfill because it had developed infrastructure, e.g., streets, with relatively upscale homes along them. In 1973, waste was first placed at the Payatas Landfill site as general fill for a depressed area and it remained as a small landfill for the use of the Payatas housing development. In 1988, the Smokey Mountain Landfill in Manila was closed and the rate of landfilling at the Payatas Landfill increased significantly.

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Figure 1. Location map for Payatas landfill near Manila (Britannica 2011 and Wikipedia 2011).

The metro-Manila area generated an average of 5,443 metric ton (6,000 tons) of municipal solid waste (MSW) per day during the 5 years prior to the failure. About 1,540 metric ton (1,700 tons) per day of that MSW was placed at the Payatas Landfill. In 1998, the landfill was supposed to close, but the Quezon City Government asked the Metro Manila Development Authority (MMDA) to postpone closure because of the higher cost of using the landfill in San Mateo, Rizal (~6.7 km away). According to the MMDA, the Quezon City Government continued to postpone closure of the Payatas Landfill until one month prior to the 10 July 2000 slope failure.

Description of South Landfill

This section describes observations of a smaller landfill immediately southeast of the waste failure that helps interpret the waste slide. These observations are based on both site reconnaissance and photos. Figure 4 shows the slopes at this smaller landfill, which did not fail after both typhoon events although both facilities followed the same landfill operation procedures. The following bulleted sections discuss in numerical order the labels shown in Figure 4:

1. The access road to the landfill is a street through a residential neighborhood. Most likely, waste disposal consisted of dumping at the edge of the road and subsequent pushing down the slightly sloped terrain. The waste height eventually reached the same elevation as the surrounding residential area and waste trucks drove upslope the resulting waste slope for unloading;

2. A deep, man-made trench at the slope crest was excavated to collect and channel surface precipitation. Although this trench did not result in slope failure, a similar trench was observed near the failed waste mass. The trench is believed to have acted as a water-filled tension crack, where the failure plane intersects the bottom of the crack. The tension crack depth increased with frequency of precipitation and subsequent waste erosion. Also, the trench provides a direct conduit of water to lower depths of the landfill, thereby decreasing effective stress in the waste. This landfill, like the landfill with the slope failure, did not have a soil cover on the top of the facility, which allowed for faster infiltrate of precipitation over a large area of the landfill;

3. A combination of steam and smoke is emanating from the slope in the foreground. It is common for steam or smoke to be present because of the large area of uncovered MSW that can result in spontaneous combustion. Residents living downslope of the failure heard a loud explosion before the waste failure (Merry et al., 2005). The explosion may have been facilitated by elevated gas pressures due to MSW degradation and steam or smoke due to spontaneous combustion of the exposed MSW;

4. A leachate collection trench surrounds the landfill periphery and is used to collect and remove leachate seeping from the waste pile.

Figure 2. Aerial view of 12.7-hectare Payatas Landfill with slope failure and homes visible in foreground (photo by S.M. Merry).

Figure. 3 Aerial view of waste slide from news video before regrading of slide mass (Baua, 2011).

Figure 4. Stable landfill south of Payatas Landfill with waste slide showing smoke plumes (photo by S.M. Merry).

Description of Cross Section and Cause of Failure

Figure 5 shows a close-up of the slope failure area and scarp in Payatas Landfill. Regrading of the slide mass had started shortly after the failure, which is shown in Figure 5. A leachate collection trench was excavated along the toe of the slide mass and is highlighted by the arrow and text box in Figure 5. The native brown soil underlying the waste and residential area is visible along the toe and sides of this excavated leachate collection trench. Leachate seepage existed prior to and shortly after the slope failure; however, the exact leachate level in the landfill just prior to failure is not known. A smaller landfill leachate collection trench was present along the slope toe prior to the slide and was similar to the trench shown in Figure 4. The leachate collection trench as it existed prior to the slide was covered by the slide mass and hence, is not visible (see the central portion of Figure 5). However, it was visible along the toe of the north-facing slope of the landfill east of the slide mass (see the top of Figure 5 where a small trench separates homes from the landfill toe and is highlighted by an arrow and text box). In summary, Figure 5 indicates that the leachate trench in front of the slide mass was constructed after the slide to collect leachate exiting the slide mass and a considerable volume of leachate existed in the landfill prior to failure but the exact leachate level at failure is not known.

Figure 6 shows a series of aerial views of the waste slide area that were used to understand the failure surface for use in the stability analyses. Figures 6(a) and 6(b) provide side views from the southeast and northwest direction, respectively, of the waste slide. Figure 6(c) highlights benches in the landfill slope, and Figure 6(d) delineates a mildly sloped toe buttress. Digital measurements from these photographs were used to create the cross-section shown in Figure 7. These measurements indicate toe buttress slope angles in the slide area of 1.4H:1V (~35°) to 0.84H:1V (~50°) and steeper side slopes above the buttress that range from 60° to 70° (Bautista, 2007; UNFCCC, 2007). Additionally, site reconnaissance and pictures both indicate that leachate was seeping from this toe buttress.

The waste is underlain by a native hard, brown fine-grained loam or adobe, which is a moderate plasticity silty clay (Seman and Rydergren, 1992). In this study, the failure surface was deemed not to have passed into the native brown silty clay because photographs in Figures 2, 5, and 6(c) indicate that the failure surface is relatively shallow and exits the slide mass near the toe of the MSW. This is indicative of a slab or sliver of MSW sliding off the surface of the landfill. In addition, the brown native silty clay is sloped upward (see Figure 7) near the slope toe because waste was initially placed in this depressed area.

In 1992, the landfill was considered to be nearing its final height of 18 m. An initial closure design was prepared by VBB VIAK Engineers, Stockholm, Sweden (Merry et al., 2005). Seman and Rydergren (1992) plans show the existing landfill slope toe is connected directly to an excavated leachate collection trench along the slope toe. Seamen and Rydergren (1992) describe frequent occurrences of elevated temperatures and/or gas pressures in the landfill. In addition, site reconnaissance in 2000 revealed smoke or steam escaping from the smaller landfill located south of the failed slope (see Figure 4).

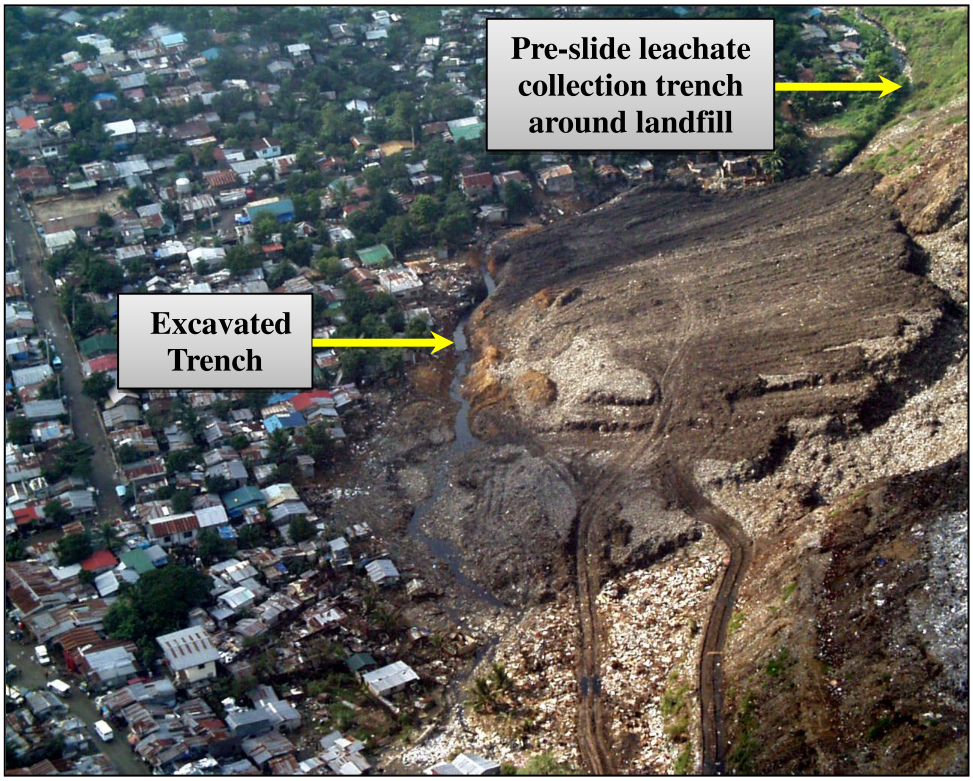

Figure 5. Slope failure showing new and existing leachate trenches along slope toe (photo by S.M. Merry).

The cause of slope failure is hypothesized to involve lowering of the effective stresses along the failure surface. Five factors may have contributed to the failure: (1) presence of leachate due to precipitation, (2) generation of landfill gas pressures in the waste, (3) low MSW density or unit weight due to limited compaction resulting in greater infiltration, (4) steep exterior slopes, and (5) low or reduced shear strengths of MSW. These five factors are investigated and discussed in this section.

- First, infiltration of precipitation was likely intensified by a lack of drainage (i.e., ponding of precipitation), the drainage trench that was excavated to direct surface runoff towards the slope crest, and the lack of cover soil at the top of the landfill. In addition, excavation of drainage trenches at the toe of the landfill reduced resisting forces available to the slope. The resulting rapid infiltration provided the source for water, which then lowers the effective stress within the waste mass. A similar phenomenon was observed with a waste slide that occurred in Java, Indonesia, where people also perished (Koelsch, 2005). Both case histories indicate landfills in tropical climates with intense rainfall can experience slope failure from elevated leachate or pore pressures.

- The second factor involves the possibility that excess gas pressures, e.g., methane, generated by MSW biodegradation within the saturated zone, further lowered the effective stress in the MSW and the corresponding shear resistance along the failure surface. This is a realistic contributing factor as the build-up of landfill gas pressure within saturated MSW provides pore pressures similar to those observed during pressurized leachate recirculation (Merry et al. 2006), which was thought to contribute to the Dona Juana landfill slope failure (GeoSyntec Consultants, 1998; Gonzalez-Garcia and Espinosa-Silva, 2003; Hendron et al., 1999). Loud cracking sounds, e.g., possibly methane combustion, were heard from near the northern flank of the landfill after the slope failure (Merry et al., 2005).


(a)
Seeping Leachate


(b)

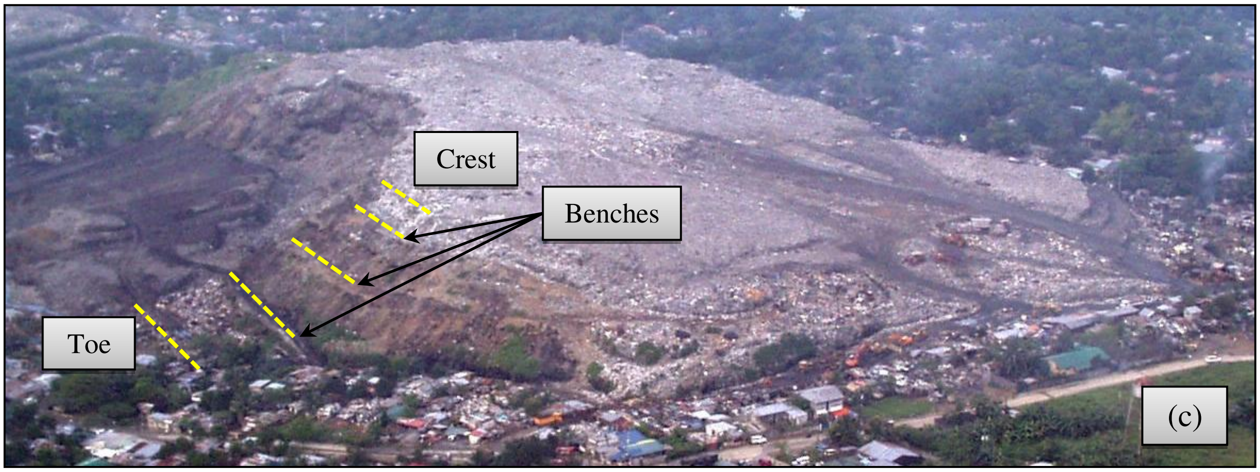

Figure 6. Slope geometry and failure surface after failure (a) evidence of seeping leachate, (b) representative slope inclination, (c) location of slope benches, (d) toe buttress (photos by S.M. Merry).

- The third contributing factor for the waste slide is the low waste density or unit weight, due to limited compaction and a high portion of light plastic materials (Koelsch, 2001). The absence of soil cover, a drainage trench excavated at the slope crest, and low MSW unit weight reduced the surface flow of rainfall and resulted in a higher infiltration rate into the waste.

- The fourth contributing factor is the pre-failure slope geometry. The steep exterior slopes ranging from 40° to 70° result in additional weight, i.e., driving force, acting on the failure surface.

- The fifth factor involves low or possibly reduced shear strength of MSW. In engineered landfills in North America, MSW shear strength is relatively high and has been documented in numerous studies (Kavazanjian et al., 1995; Van Impe, 1998; Eid et al., 2000; Stark et al., 2009; Bray et al., 2009). However, in landfills that possess minimal compaction, high moisture content, and thermal and biological decomposition, the shear strength along the failure surface may be lower than that typical of engineered landfills in North America.

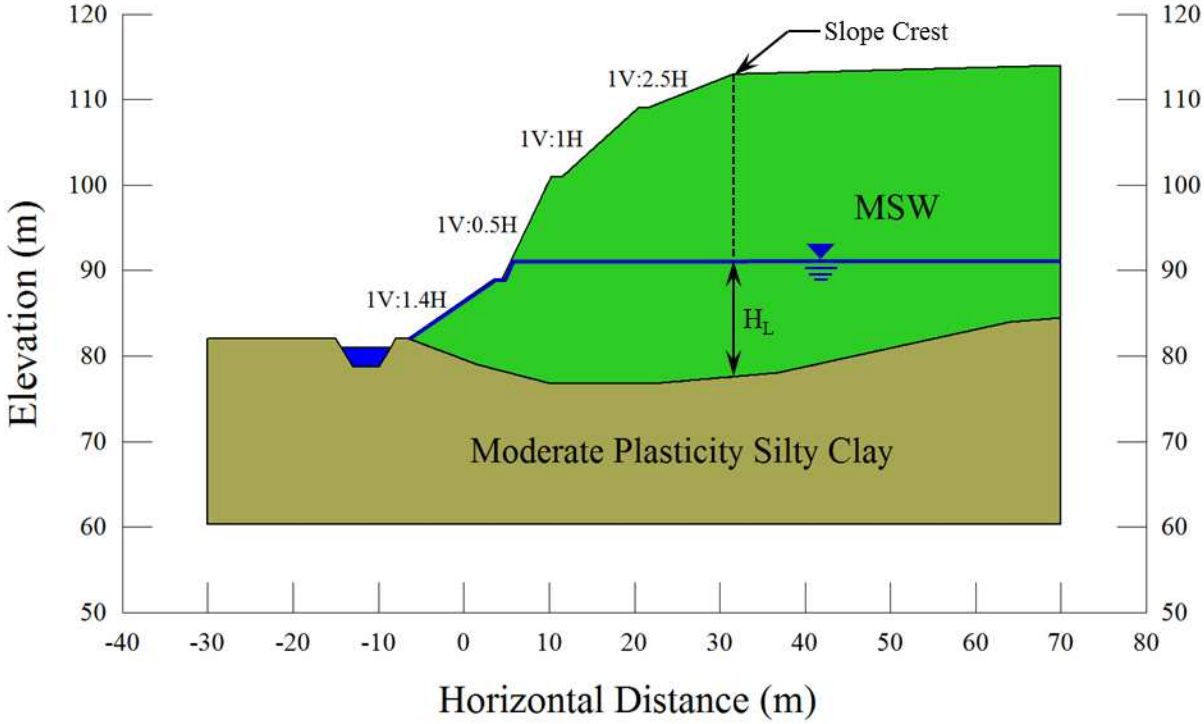

Figure 7. Two dimensional cross-section of failed waste slope.

Manila Waste Composition

Koelsch (2005) visited the Payatas landfill four weeks after the waste slide and observed that the waste composition included a high concentration of plastics and organics and an absence of certain other materials (e.g., paper, glass, metals). The waste composition in Manila is typical for an urban area in a developing and emerging economy, which results from a large consumption of plastics as package material, a lack of recycling for organic waste, and an outstanding recycling rate for reusable materials, such as aluminum, glass, and paper. As a result, scavengers live nearby collecting cans, scrap metals, bottles, and newspapers for recycling. The waste composition of the slide mass (see Figure 8) consists of large shaped plastic particles intermixed with organic matter. Also, little or no soil cover is evident from Figure 8. Table 1 presents typical waste composition from a solid waste feasibility study for metropolitan Manila (Townsend and Associates, 1989). In this case, the water content, or mass ratio of water to dry waste, is about 82.5%.

Figure 8. Composition of waste mass in slide area with no soil cover (photos by S.M. Merry).

Table 1. Waste Composition in Metro Manila (data from Townsend & Associates, 1989).

Component	Wet Basis (%)	Dry Basis (%)	Organic Composition (% dry basis)
Food wastes	11.0	3.3	3.1
Fines and inert materials	12.9	6.4	1.3
Paper and cardboard	10.2	9.2	8.7
Glass	1.9	1.9	0
Plastic and petroleum products	9.8	8.8	7.9
Yard and field wastes	33.5	6.7	6.0
Textiles	4.1	3.3	2.6
Leather and rubber	1.8	1.6	1.4
Metals	3.3	3.3	0
Wood	11.5	10.3	9.3
Total	100	54.8	40.5

MSW is a composite material having two main components, i.e., a paste fraction (soil-like mechanical behavior) and fibrous materials, which acts as reinforcement inside the MSW (Koelsch, 1995). Variability of MSW composition (paste and fibrous materials) affects the in-situ unit weight, shear strength, decomposition and gas production, and infiltration. Based on the waste composition and lack of soil cover and compaction, the unit weight is expected to be lower than engineered landfills in North America. The impact of the low unit weight on landfill stability was amplified by heavy rainfalls, which are characteristic for tropical locations. Machado et al. (2002, 2008) show the mechanical response (e.g., shear strength) of MSW materials under static loading is governed mainly by the fibrous elements and the paste, which acts as a medium to hold the fibrous part together and generate a reinforced material. The hydraulic conductivity of MSW is also governed by waste composition, effective stress, and unit weight (Beaven et al., 2008). As a result, the next sections provide recommendations for values of MSW unit weight, shear strength, and hydraulic conductivity for use in the stability and leachate analyses of the Payatas landfill, as presented herein.

Recommended MSW Unit Weight

MSW unit weight is important because variations in unit weight can affect the slope stability analyses by increasing or decreasing the normal force acting on the failure surface (Stark et al., 2009). As with soils, the unit weight is affected by compaction effort and lift thickness, depth (overburden stress), and moisture content. Unlike soils, the unit weight also varies significantly because of large variations in waste constituents (e.g. organics and fibrous materials), state of decomposition, and degree of control during placement (such as thickness of waste and daily cover, if any). It is generally believed that the initial unit weight of waste is dependent on waste composition, amount of daily cover, and degree of compaction during placement. As the waste decomposes, the unit weight becomes more dependent on depth of burial, degree of decomposition, and climate conditions. Stark et al. (1998) show that decomposed waste exhibits a unit weight close to soil, e.g., 15.7 kN/m^3 (100 pcf), and this increased unit weight should be considered in stability analyses. Although unit weight can vary significantly over short distances, average values of unit weight are usually used for analysis purposes.

Payatas landfill waste has a relatively large ratio of waste to soil and contains a large amount of plastics. In addition, the waste was not well compacted. Zekkos et al. (2006) present data on MSW unit weights based on in-situ measurements of landfills in the United States, which range from 6.5 to 16 kN/m^3. Considering the landfill disposal and climate conditions at Payatas, the average unit weight is assumed to be 10 kN/m^3 (63.7 pcf). However, sensitivity analyses were performed herein for unit weights ranging from 8 to 14 kN/m^3 (50.9 to 89.1 pcf).

Recommended MSW Hydraulic Conductivity

Many researchers have evaluated the hydraulic conductivity of waste from both laboratory samples (either recompacted or intact samples) and field tests. Reported values of MSW hydraulic conductivity vary between approximately 1x10^{-3} m/s to

$1x10^{-9}$ m/s, although typical values are in the range of 10^{-5} to 10^{-6} m/s (Beaven et al., 2008). The hydraulic conductivity of saturated waste from laboratory tests under moderate effective stresses range from 10^{-3} m/s to 10^{-6} m/s (Zhan et al., 2008; Chen and Chynoweth, 1995; Jang et al., 2002; Reddy et al., 2009). The Hydrologic Evaluation of Landfill Performance (HELP) model (Schroeder et al., 1994) uses a default value of $1x10^{-5}$ m/s for MSW.

The physical properties that influence MSW hydraulic conductivity are density, particle size, porosity, material type, degree of saturation, stage of decomposition, and depth within the landfill. Other important influences include vertical anisotropy of the waste, heterogeneity of the waste, and gas production. Beaven (2000) and Powrie et al. (2005) indicate a decrease in hydraulic conductivity with increase in effective stress, waste density, cover soil thickness (Oni and Okunade, 2009), and decrease in waste porosity. Jang et al. (2002) report decrease in hydraulic conductivity with increase in compactive effort for both solid waste and cover soil.

The landfill operations at Payatas consisted of open disposal with little compaction or soil cover, and a waste composition having a large percentage of fibers (e.g., plastics) compared to organic wastes. Therefore, it is estimated that the MSW hydraulic conductivity is in a range of 10^{-3} to 10^{-5} m/s. Figure 9 shows the back-scarp of the waste slide as well as a bulldozer regrading the slide mass. The individual fibrous materials observed hanging at the scarp are most likely the result of the waste pulling from the emdedded fibrous materials. In Figure 9, the red circle highlights leachate flowing from the waste mass. The leachate flowing from the MSW alludes to localized conduits within the waste mass that concentrate leachate flow. This observation does not provide a quantitative MSW hydraulic conductivity. Assigning a global average MSW hydraulic conductivity of 1 x 10^{-5} m/s captures these features for the purposes of the slope stability analyses. A value of 1 x 10^{-5} m/s is subseqently used in the HELP modeling of the leachate level and as an inuput parameter for estimating the landfill gas pore pressures at the time of failure.

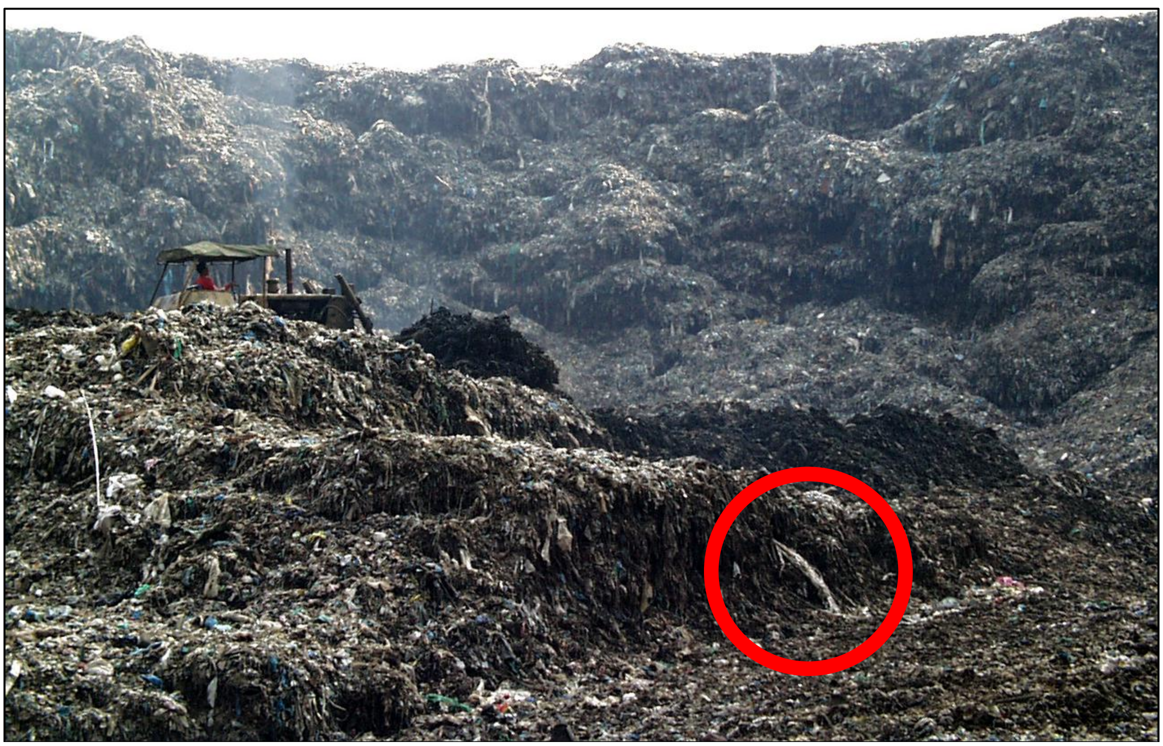

Figure 9. View of the back-scarp of waste slide and leachate flowing from the exposed MSW (photo by S.M. Merry).

Recommended MSW shear strength parameters

Shear strength testing of MSW is difficult because of the heterogeneous composition of landfill materials, difficulty in sampling, specimen preparation, shear testing, magnitude of shear displacement, range of particle size, unit weight (Zekkos, 2005; Karimpour-Fard et al., 2011), and time-dependent properties, such as, the age of the MSW and decomposition state (Nascimento, 2007), unit weight, etc. As a result, many recommendations on the shear strength of MSW have been published, e.g., Kavazanjian et al., 1995; Van Impe, 1998; Eid et al., 2000; Stark et al., 2009; Bray et al., 2009; Zekkos et al. 2010). For example, Eid et al. (2000) and Stark et al. (2009) recommend stress-dependent MSW shear strength parameters based on back-analysis of failed landfill slopes and laboratory tests. Kavazanjian et al. (1995) base their strength envelope on the lower bound of the laboratory and back-calculation of unfailed landfill slopes. Van Impe (1998) summarizes the shear strength of MSW data from laboratory tests, as well as from back analysis of case histories, suggesting a tri-linear strength envelope. Bray et al. (2009) present a shear strength envelope to characterize the static MSW shear strength using a stress-dependent Mohr-Coulomb strength criterion with c' = 15 kPa, φ_o' = 36˚, and $\Delta\varphi$ = 5˚. Table 2 summarizes the above mentioned MSW shear strength parameters.

Each of the above-referenced shear strength recommendations were developed for MSW below its field capacity. When fine-grained materials are intermixed with partially saturated MSW, negative pore pressures may exist in the partially saturated media. If the negative pore pressures are ignored, the total and effective stresses are equal and the shear strength envelopes may be directly compared. The MSW strength envelopes have generally resulted in progressively higher shear strengths, although recent work (e.g., Stark et al., 2009 and Bray et al., 2009) tends to reduce the shear strength at higher normal stresses due to the stress dependent nature of MSW. MSW strengths for effective normal stresses of 500 kPa or less are considered herein because of the shallow nature of the Payatas slope failure suggested by Figure 2.

In addition to being dependent on the effective normal stress on the failure plane, the shear strength of MSW is also shear displacement or axial strain dependent and tends to increase with increasing deformation (Eid et al., 2000). Review of existing laboratory data show that most of the laboratory shear tests investigated are not continued to sufficient displacement or strain to mobilize the peak shear strength of the MSW. As a result, Stark et al. (2009) provide MSW shear strength envelopes mobilized at horizontal displacements of 10, 30, 50, 100, and 150 mm. Table 3 shows equations for each strength envelope estimated from direct shear tests, where R^2 values are all above 0.90. Figure 10 compares the shear strength envelopes in Table 2 to envelopes recommended by others (e.g., Stark et al., 2009; Bray et al., 2009; Kavazanjian et al., 1995). As highlighted in Figure 10, all of the recommended envelopes fall between the 50 and 100 mm of shear displacement strength envelopes.

Table 2. Summary of MSW shear strength parameters.

Recommended Shear Strength Parameters		Author
$0 \le \sigma'_n \le 30\ kPa$ $30 < \sigma'_n \le 300\ kPa$	$c' = 24\ kPa \quad \phi' = 0°$ $c' = 0\ kPa \quad \phi' = 33°$	Kavazanjian et al. (1995)
$0 \le \sigma'_n \le 20\ kPa$ $20 < \sigma'_n \le 60\ kPa$ $\sigma'_n > 60\ kPa$	$c' = 20\ kPa \quad \phi' = 0°$ $c' = 0\ kPa \quad \phi' = 38°$ $c' = 20\ kPa \quad \phi' = 30°$	Van Impe (1998)
	$c' = 25\ kPa \quad \phi' = 35°$	Eid et al. (2000)
$0 \le \sigma'_n \le 200\ kPa$ $\sigma'_n > 200\ kPa$	$c' = 6\ kPa \quad \phi' = 35°$ $c' = 30\ kPa \quad \phi' = 30°$	Stark et al. (2009)
$\tau = 15kPa + \sigma'_n \tan\left(36° - 5° \log\left(\frac{\sigma'_n}{P_a}\right)\right)$		Bray et al. (2009)

In fact, for effective stresses lower than 200 kPa, recommended strength envelopes are about average of these two displacements. For horizontal displacements of 10, 30, and 50 mm, the strength envelopes are linear and can be described by Mohr-Coulomb cohesion and friction angle parameters. As the displacement increases to 100 mm, the strength envelopes are stress dependent and described by a power trend with no apparent cohesion. Based on preliminary stability analyses, the 50 mm envelope in Table 3 is assumed to represent the average mobilized MSW strength at the time of the slide. The section "Inverse-Analysis of Wasteslide" describes the rationale for using the 50 mm envelope.

Table 3. Recommended MSW Direct shear strength envelopes.

Δx (mm)	Shear strength envelope	R^2
10	$\tau = 0.7 + \sigma_v' \tan(15.1°)$	0.92
30	$\tau = 14.1 + \sigma_v' \tan(22.3°)$	0.90
50	$\tau = 20.8 + \sigma_v' \tan(24.7°)$	0.93
100	$\tau = 4.08{\sigma_v'}^{0.714}$	0.91
150	$\tau = 2.35{\sigma_v'}^{0.846}$	0.99

STABILITY ANALYSES

The cross section in Figures 11(a) and 11(b) show the estimated circular and planar failure surface, respectively, based on available photographs (see Figures 2, 3, 5, and 6) and site reconnaissance. The cross sections show the MSW is approximately 31 m thick and is underlain by the brown native clayey silt that slope downward towards the center of the landfill. A leachate trench is shown at the toe of the slope to model the pre-failure slope condition. These cross-sections were used in the inverse analysis described below.

Slope Stability Analyses

XSTABL (Sharma 1996) and SLOPE/W (GeoStudio, 2007) were used to perform the slope stability analyses described below. XSTABL and SLOPE/W allow a search for the critical failure surface and specification of a particular observed failure surface. The Morgernstern and Price stability method (Morgernstern and Price, 1965) was used to conduct two-dimensional analyses. This procedure satisfies static equilibrium completely and is applicable to most slope geometries and soil profiles.

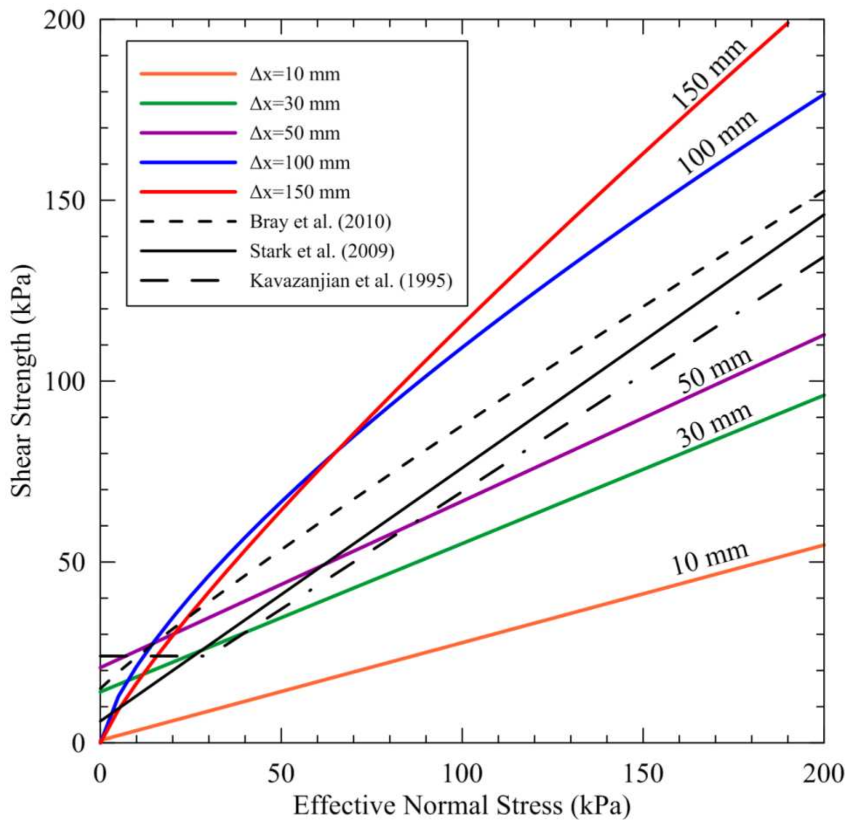

Figure 10. MSW shear strength envelopes.

Merry et al. (2005) report there was considerable leachate seeping from the waste into the excavated trench at the toe of the slope after the slide. Therefore, inverse analyses presented herein investigated the leachate level necessary to reduce the factor of safety (FS) to unity. The piezometric surface is assumed horizontal because some soil cover is present on side slopes near the slope toe, which reduced the potential for leachate outbreaks above the toe. The leachate level, H_L, defined in Figure 7 was back-calculated using MSW unit weight (8 to 14 kN/m^3 (50.9 to 89.1 pcf) with an average of 10 kN/m^3 (63.7 pcf)) and shear strength parameters (see Table 3) discussed above. The leachate level at the time of failure (H_L) is assumed horizontal, where H_L is measured from elevation 78 m and horizontal distance of 31 m. The elevation and horizontal distance correspond to top of native foundation soil and slope crest (see Figure 7). The resulting H_L is compared to a subsequent HELP model analysis to determine the accuracy of the back-analysis.

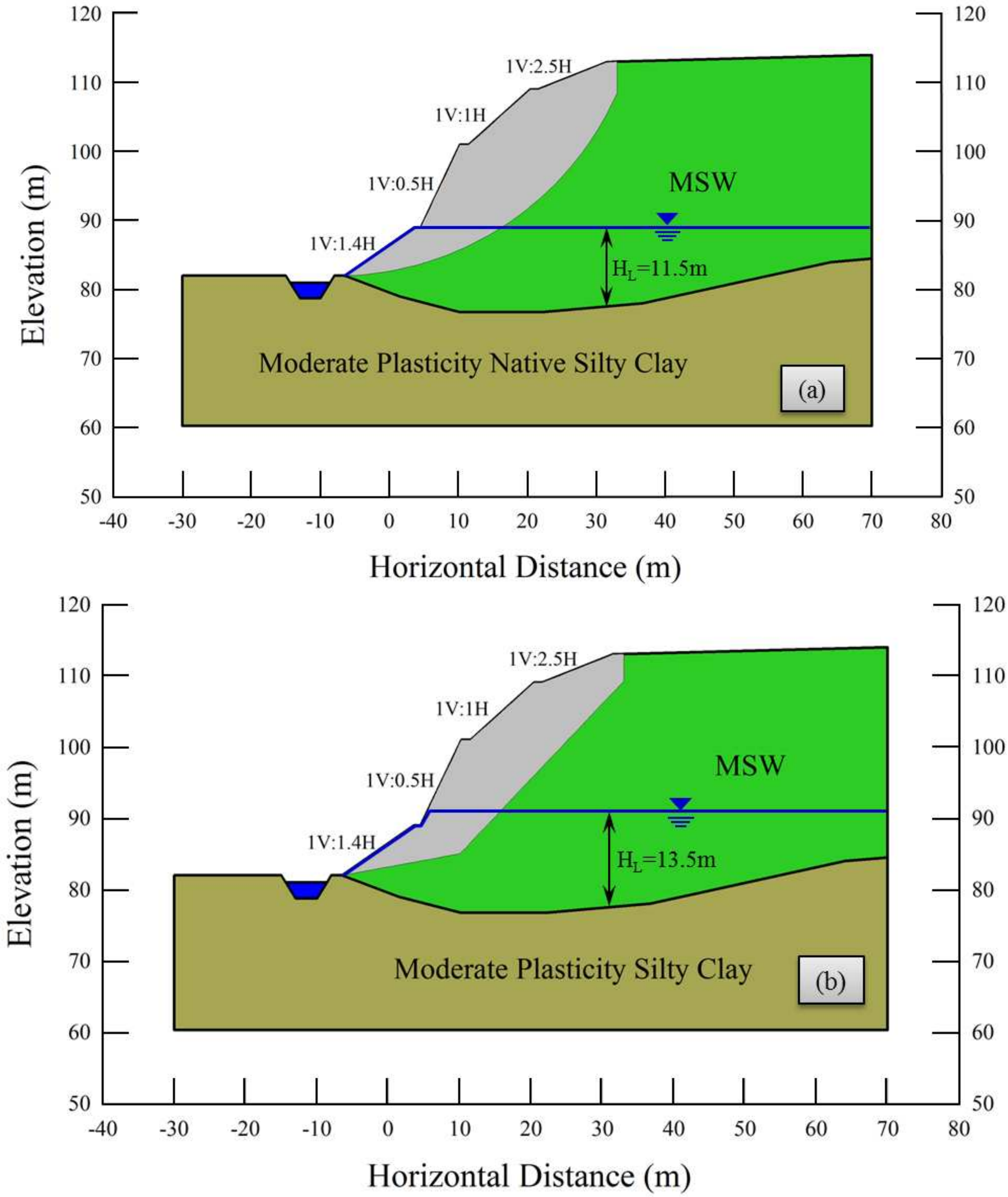

Figure 11. Cross- sections showing location of (a) circular and (b) planar failure surfaces.

The failure surfaces shown in Figure 11 were estimated from photographs in Figures 2, 3, 5, and 6 and consist of a shallow circular and shallow planar failure surface through only the MSW. From Figure 3, the authors surmise that the failure surface is shallow in nature, and the waste mass slid down the side slope as a mass, i.e., a slab or sliver slide. Subsequently, ledges developed along the main scarp formed from vertical blocks toppling over or sliding down (see Figure 3). Therefore, it was concluded that the failure surface passed from the crest of the slope through the MSW, and exited at the slope toe without entering the underlying brown native clayey silt.

Although a circular failure surface indicates a homogenous material, a planar failure surface starting at the toe and daylighting at the top of the landfill was also analyzed. Because the exact location of the failure surface at the crest is not known, the scarp of the waste slide was used to estimate the location of the failure surface at the top of slope that yielded the lowest factor of safety. Furthermore, after the waste slide, the waste was regraded (see Figures 2 and 5) such that the actual slide mass geometry does not represent the possible progression of failure. A helicopter video from a news report (http://www.abs-cbnnews.com/video/nation/metro-manila/07/10/11/payatas-tragedy-commemorated) shows an aerial view of the landfill prior to regrading, which was also helpful in estimating the location and geometry or the failure surface. From this video, the failure progresses from the main slide mass and regressing from the initial head scarp to a final scarp that is about 5 m behind the slope crest. Then, an ensuing block of waste toppled from the scarp creating the final geometry shown in Figure 2. Merry et al. (2005) report that a steep (vertical at the top) head scarp remained above the area where heavy machinery had reworked the waste mass.

Inverse-Analysis of Wasteslide

Before proceeding with the inverse analysis, preliminary stability analyses of the cross-section shown in Figure 11(a) were performed using the observed circular failure surface and each of the strength envelopes provided in Table 3. The analyses yielded the approximate FS for each MSW strength envelope. A piezometric or phreatic surface was not used in these analyses, so these results were used to estimate the level of shear displacement involved and mobilized strength envelope. The resulting values of FS indicated that horizontal shear displacements of 10 and 30 mm under-predicted the mobilized shear strength (see Table 4). The authors reviewed published MSW shear strength envelopes for modern engineered landfills in North America. Three published shear envelopes (Bray et al., 2009; Stark et al., 2009; Kavazanjian et al., 1995) along with displacement based direct shear envelopes are shown in Figure 10. Based on preliminary stability analyses, the 50 mm envelope in Table 3 was selected because it gave a reasonable factor of safety for conditions before the failure. Due to lack of compaction, biodegradation, waste picking, and possible thermal degradation, the MSW shear strength at Payatas (50 mm envelope from Table 3) is expected to be lower than modern engineered landfills in North America.

In the inverse analysis described below, the piezometric surface required to reduce the FS to unity is about 11.5 m and 13.5 m for circular and planar failure surfaces, respectively, using the Δx=50 mm MSW shear strength in Table 3. A leachate level (H_L) of 11.5 m is greater than usually associated with engineered landfills but it is deemed plausible for the Payatas landfill because no soil cover was present, two recent typhoons impacted the site, limited waste compaction or grading occurred, a precipitation collection trench was excavated at the top of slope, no leachate collection system was installed, and the facility is located in a depressed area. In addition, ponded water/leachate was observed at the top of the landfill during the site reconnaissance which occurred several weeks after the failure (Merry et al., 2005). Finally, Figure 6(a) shows leachate present at the first bench of the landfill several weeks after the failure.

Table 4. Preliminary factor of safety of circular failure surface with no leachate.

Δx (mm)	Shear strength envelope	FS
10	$\tau = 0.7 + \sigma_v' \tan(15.1°)$	0.44
30	$\tau = 14.1 + \sigma_v' \tan(22.3°)$	0.93
50	$\tau = 20.8 + \sigma_v' \tan(24.7°)$	1.16
100	$\tau = 4.08{\sigma_v'}^{0.714}$	1.82
150	$\tau = 2.35{\sigma_v'}^{0.846}$	1.86

Landfill Gas Pressures

It is hypothesized that landfill gas and leachate pressure reduced the effective stress along the failure surface. Landfill gas, which is dominated by methane and carbon dioxide, is formed as a natural part of anaerobic degradation. In landfills that are sufficiently dry, landfill gas migrates readily through the pore space and is released into the atmosphere with little build-up of pressure in the waste. However, as saturation levels in the waste rise, the permeability of the waste to landfill gas migration decreases. Merry et al. (2006) use a one-dimensional (1-D) column of saturated media where degradable waste exists only in the lowest portion of the column to model this situation. This model shows that below the level of saturation, the steady-state excess gas pressure increases linearly similar to a hydrostatic condition. The combination of hydrostatic leachate pressure and gas-induced pore pressure may be accounted for using an equivalent fluid unit weight for the pore fluid that is artificially higher than water. A subsequent parametric study of the input parameters shows that the equivalent fluid unit weight of the pore fluid is dependent on waste hydraulic conductivity (Merry et al., 2006). The corresponding MSW hydraulic conductivity for this case is assumed to be greater than $1\text{x}10^{-5}$ m/s. Figure 12 shows the relationship between waste hydraulic conductivity and equivalent fluid unit weight of the pore fluid and is used to estimate an equivalent fluid unit weight. The input parameters for Figure 12 are provided in Table 5. The gas generation rate (Q_t) is estimated by Eq. (1):

$$Q_t = \sum_{i=1}^{n} 2 \cdot k \cdot L_o \cdot M_i \cdot (e^{-k \cdot t}) \quad (n \le t_a) \tag{1}$$

where Q_t is the expected gas generation rate in the t^{th} year (m^3/yr), k is the methane generation rate constant (yr^{-1}), L_o is the methane generation potential m^3/Mg, M_i is the mass of solid waste filled in the ith year (Mg), t_i is the age of the waste mass in the t^{th} year (yr), and t_a is the total years until the slope failure (yr). The representative values for L_o and k provided by Burklin and Lloyd (2009) for the Phillipines are 60 m^3/Mg and 0.18 yr^{-1}, respectively. From 1988 to 2000, an average 1,500 metric tons of MSW/day was disposed in the Payatas Landfill and corresponds to Q_t equal to $5.27\text{x}10^7$ m^3/yr. The gas generation rate, Q_t, is incorporated into the gas pressure model by introducing the term β_{gas}, or ratio of gas generation rate to MSW weight disposed in one year (Merry and Kavazanjian, 2005; Merry et al., 2006). β_{gas} is reported in units of m^3/ kg waste/ yr. Assuming $4.8\text{x}10^8$ kg MSW is disposed each year, β_{gas} is estimated to be 0.112 m^3/ kg waste/ yr.

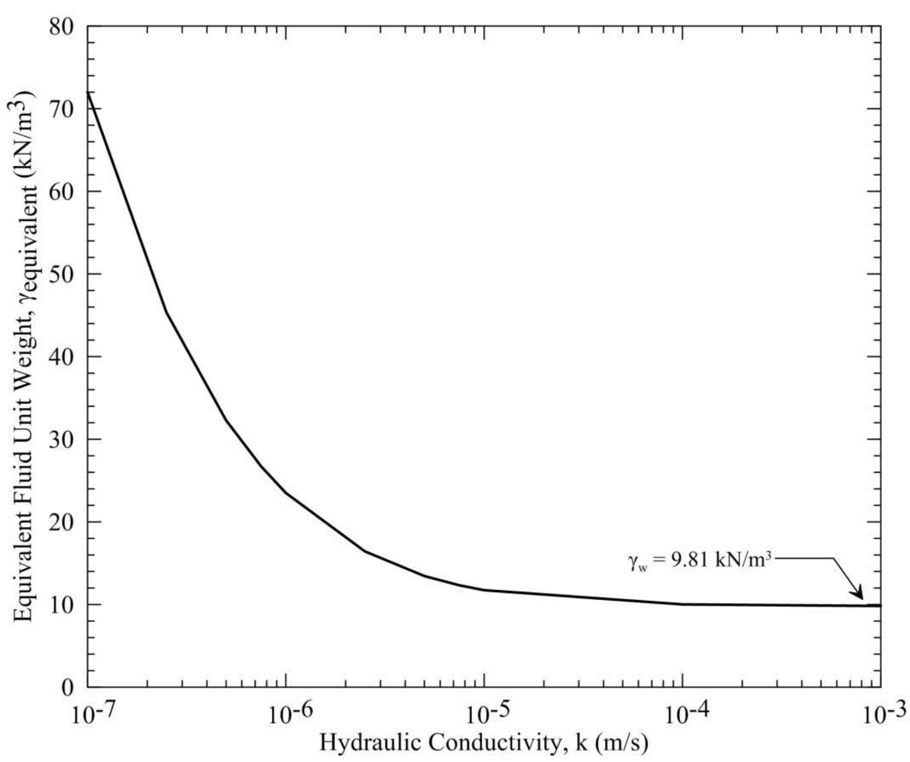

Figure 12. Dependence of equivalent fluid unit weight, $\gamma_{equivalent}$, on MSW saturated hydraulic conductivity, $K_{waste,sat}$

Using Table 5, the equivalent fluid unit weight of the pore fluid, $\gamma_{equivalent}$, that accounts for both hydrostatic water pressure and gas pore pressure is 11.7 kN/m^3. Due to the inclusion of gas pressure in the inverse analysis, the piezometric surface that gives a FS of unity is lowered by 1 m for both the circular and planar failure surfaces in Figure 11.

Table 5. Input parameters for landfill gas pressure analysis.

Parameter	Value
γ_{MSW}	10 kN/m^3
β_{gas}	0.112 m^3/ kg waste/ yr
k_{waste}	1 x 10^{-5} m/s
Temperature	20°C
Leachate Depth	11.5 m
$\gamma_{fluid\ equivalent}$	11.7 kN/m^3

HELP Model Estimated Leachate Level

The leachate level at time of failure was estimated with an inverse analysis using estimated MSW shear strength (see Table 3 and ΔX=50 mm), average unit weight of 10 kN/m^3 (63.7 pcf)), and the presence of gas pressure within the water (11.7 kN/m^3). To obtain another estimate of leachate level within the landfill at the time of failure, the HELP model Version 3.07 (Schroeder et al., 1994) was used. The HELP model is a quasi two-dimensional (2-D) hydrologic model because several 1-D processes are combined. In the vertical direction, precipitation, infiltration, evapotranspiration, and percolation are combined with lateral flow (surface runoff and lateral drainage) to estimate leachate generation and level. The HELP model uses weather, soil, landfill design data, and solution techniques that account for the effects of surface storage, runoff, infiltration, evapotranspiration, vegetative growth, soil moisture storage, and unsaturated vertical drainage to estimate leachate generation.

The 1-D column representing the Payatas Landfill in the slope failure area consists of four significant layers from top to bottom: a 0.15 m-thick layer of ponded water, 30 m-thick layer of MSW, 1 m thick MSW drainage layer, and a 25.4 m thick layer of brown native plastic clayey silt. A drainage layer is included to account for leachate discharge from the exposed landfill slopes. The hydraulic conductivity assigned to this layer is equal to that assigned MSW. The 1-D HELP column excludes the depressed area shown in Figure 7 because this zone is already saturated from prior precipitation infiltration. Pertinent HELP input parameters are summarized in Table 6.

Merry et al. (2005) conclude that waste was pushed to the edge of the landfill crest creating a depressed area on top of the landfill. This led to accumulation of a small amount, approximately 0.15 m thick layer, of ponded water on top of the landfill, which would have increased the infiltration rate into the MSW. HELP does not allow direct modeling of ponded water on the top of a landfill so the 0.15 m thick layer of ponded water was modeled as a layer of soil having a high porosity (n = 0.99), high hydraulic conductivity (k = 1 m/s), a low field capacity (f_c = 0.01), and a leaf area index (LAI) of 0 to represent a lack of vegetation. Because ponded water results from a lack of runoff, a runoff curve number of zero is also specified. This resulted in rainfall collecting in this uppermost layer with no runoff or transpiration by plants (although evaporation is permitted) and subsequently percolating vertically into the underlying waste.

Table 6. Input parameters for HELP model.

	Ponded water (layer 1)	MSW (layer 2)	Drainage (layer 3)	Subgrade (layer 4)
Layer thickness (in/m)	6/0.152	1181/30	39.4/1	1000/25.4
Porosity, n	0.99	0.67	0.67	0.471
Field capacity	0.01	0.292	0.292	0.411
Saturated hydraulic conductivity (m/s)	0.1	1x10^{-5}	1x10^{-5}	5x10^{-7}

The precipitation database in the HELP model does not provide historical precipitation data for Quezon City, so recorded rainfall data from 1 May through 31 July 2000, was obtained from the Quezon City weather station and imported into HELP. This provides a reasonable simulation of the two months of precipitation immediately preceding the slope failure. Replicating approximate initial moisture conditions before May 1st 2000 involved executing the HELP model for several years prior to the failure. As a result, weather and climate conditions similar to Quezon City were required because this data is not available from the Quezon City weather station. Quezon City, located at 14° latitude, is closer to the equator than the United States and hence, has different climate conditions. Quezon City weather is dominated by high humidity, precipitation, and warm conditions year round. Table 7 presents a summary of the key weather conditions for Quezon City. The HELP program can also generate daily precipitation data using a synthetic weather generator. By entering the normal mean monthly precipitation values of Manila, Philippines shown in Figure 13, HELP generated synthetic precipitation records for 10 years preceding the failure. This synthetic record was used until May 1st of the 10th year, so that the water content and saturation levels reached a reasonable steady state value before applying the recorded data from the Quezon City weather station. From May 1st to July 31st, recorded precipitation values from the Quezon City station (Canty and Associates LLC, 2004) were input to HELP.

Table 7. Summary of key weather conditions Quezon City.

	Quezon City
Maximum yearly precipitation (m)	3.03
Average yearly precipitation (m)	2.41
Average high temperature (°C)	30.0
Average temperature (°C)	25.6
Average low temperature (°C)	20.6
Average relative humidity (%) morning/evening	87/70
Average dew point (°C)	22.8
Average wind speed (km/hr)	11

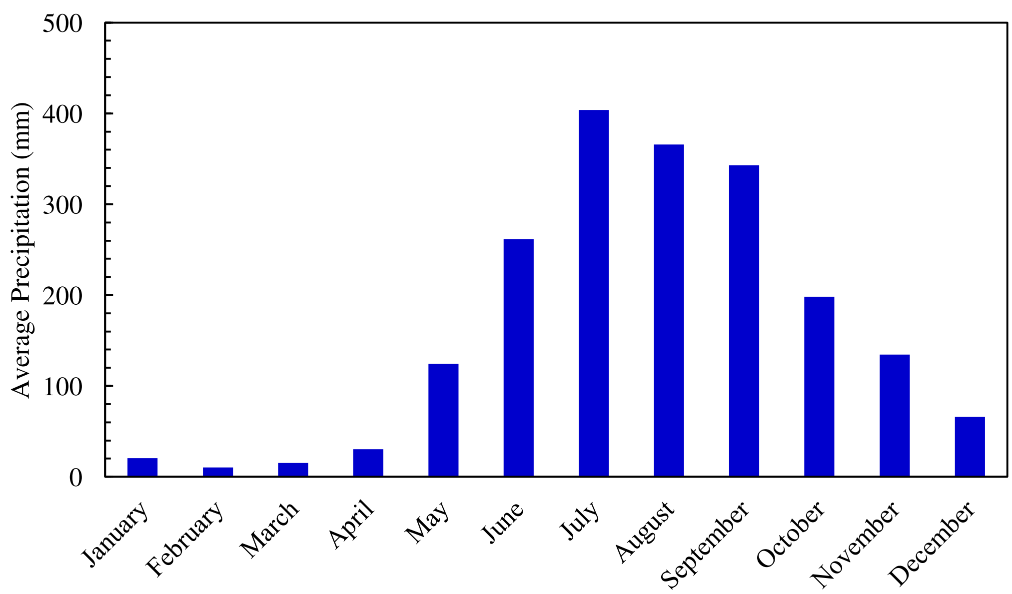

Figure 13. Average monthly precipitation for Manila, Philippines from Quezon City weather station *(data from* www.weather.com*).*

The results of the HELP model predict a leachate level of 12.9 m, including the depressed area, on the brown native plastic clayey silt subgrade on the day of the slope failure. This value is within the range of leachate level (11.5 m and 13.5 m) back-calculated using the circular and planar failure surfaces shown in Figure 11(a) and 11(b), respectively. HELP model assumptions, e.g., low estimates of evapotranspiration rates, greater lateral drainage occurring in the field than estimated, and only gravitational forces being responsible for moving pore water through the waste, i.e., ignoring capillary forces, can lead to a conservative, i.e., high, estimates of piezometric level. Another difficulty arises when modeling facilities without a proper drainage system. Although a drainage layer is not present at Payatas, one is necessary in the HELP model or leachate will gradually increase until the facility is full, i.e., the bathtub effect. This differs from field conditions where the leachate can seep from the base of the facility. In addition, the drainage layer input parameters, e.g., hydraulic conductivity, thickness, drainage distance, must also be estimated based on unknown MSW properties, which can lead to conservative, i.e., high piezometric levels.

Sensitivity Analyses

Using site reconnaissance information, pictures, and reports of the wasteslide, an inverse analysis was conducted to estimate the piezometric level at the time of failure. Figures 14(a), 14(b), 14(c), and 14(d) show the sensitivity of the factor of safety for a range of MSW shear strength parameters, MSW unit weight, and leachate level. For each sensitivity parameter, the factor of safety was computed using the circular failure surface in Figure 11(a). In Figure 14, the trend line slope is used to capture the sensitivity of FS to each parameter. For example, Figure 14(a) shows that FS decreases by 0.031 for a 1 m increase in leachate level. Using the trend line slopes for each parameter, the contributing factors leading to failure can be compared. MSW unit weight can increase when rain infiltrates into the waste mass and increases moisture content. The sensitivity of MSW unit weight is a nonlinear relationship shown in Figure 14(b). Figures 14(c) and 14(d) indicate that reducing MSW cohesion and friction, i.e., slopes equal to 0.022 and 0.025, respectively, contributes less to failure than increasing the leachate height (slope=0.031). Thus, Figure 14 suggests that leachate height has the greatest influence to reduce the FS.

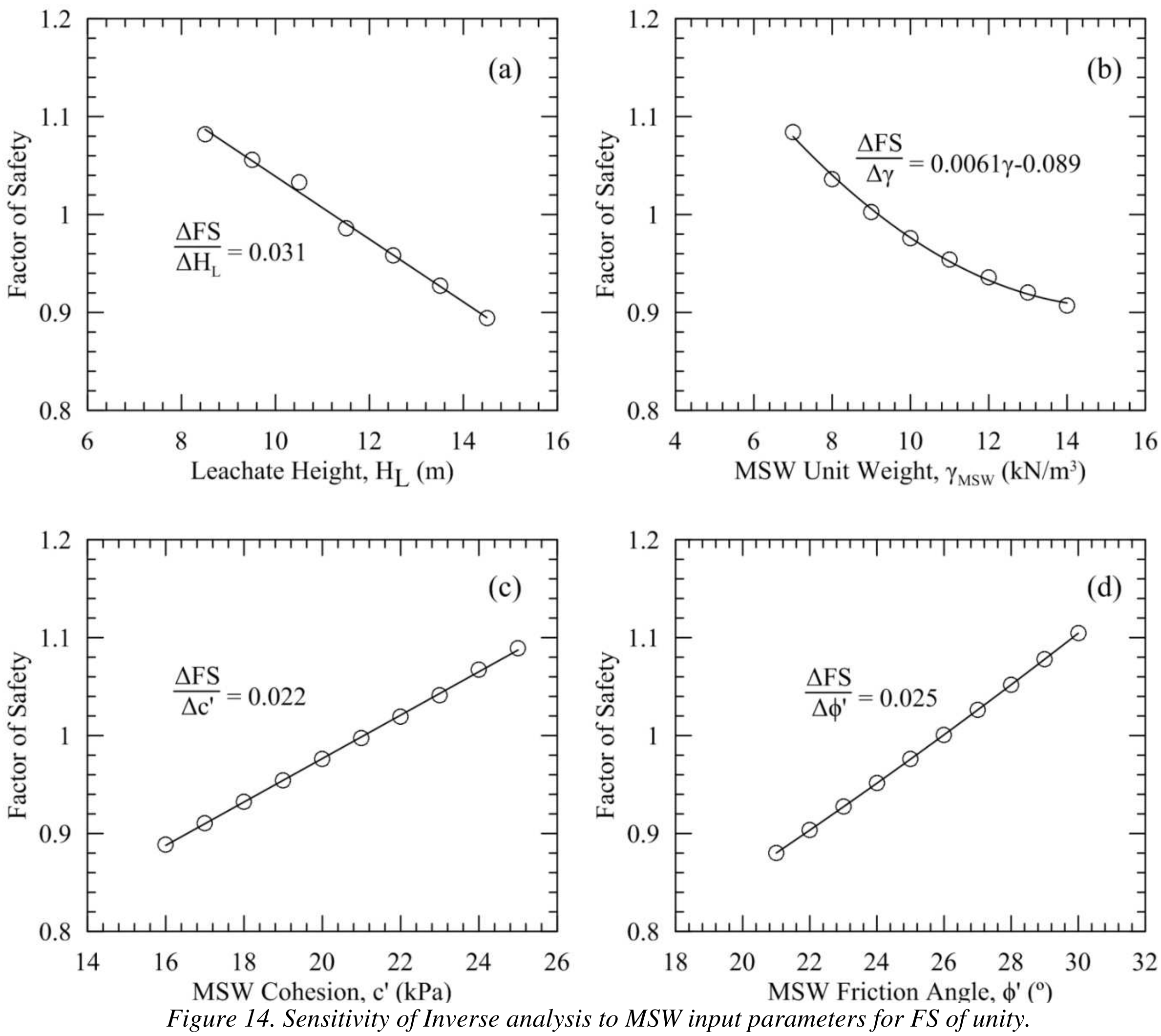

Figure 14. Sensitivity of Inverse analysis to MSW input parameters for FS of unity.

CONCLUSIONS

Results from analyses completed using the HELP model and 2-D limit equilibrium stability analyses were used to develop the following conclusions from this important landfill slope failure:

1. This case study describes a slope failure after heavy rainstorms in a tropical area. A similar waste slide occurred in Java, Indonesia, where people also perished (Koelsch, 2005). Both case histories demonstrate that landfills in tropical climates with intense rainfall can experience slope failure from elevated leachate or pore pressures. As a result, research is needed to identify pore pressure effects in wet landfills, including those that are located in tropical areas and are exposed to intense rainfalls, as well as those that are managed with leachate recirculation.

2. In particular, rainfall from two recent typhoons and landfill gas from biodegradation caused increased pore pressures within the landfill and resulted in a decrease in effective stress and stability. The landfill gas pressure was modeled using an equivalent fluid unit weight that reflects the leachate and gas pressure. The value of equivalent fluid unit weight estimated from inverse stability analyses is about 11.7 kN/m^3 for the Payatas Landfill.

3. Due to the absence of laboratory or field testing on MSW unit weight, hydraulic conductivity, and shear strength, waste composition and landfill operation and disposal procedures were used to estimate reasonable values for the inverse stability analyses presented herein. Sensitivity analyses show that the MSW shear strength properties contribute more towards failure than unit weight.

4. The Payatas case history demonstrates that landfill operation and disposal procedures can reduce the stability of open dump facilities. The sensitivity analyses indicate that leachate rise can contribute more to a shallow slope failure than MSW properties. This is important because many modern engineered landfills practice leachate recirculation, i.e., the addition of leachate into the landfill using methods such as spray-and-drip irrigation, ponds, vertical wells, and horizontal trenches and blankets. Design engineers for landfills practicing liquids addition are faced with balancing the use of pressurized liquid addition for moisture distribution with the need to minimize leachate seepage and slope stability problems. Therefore, characterizing and monitoring the pore water pressure regime is necessary to ensure stability of leachate recirculation and bioreactor landfills in North American and non-engineered landfills elsewhere.

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