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BACK-ANALYSES OF LANDFILL SLOPE FAILURES

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ABSTRACT

This paper investigates the shear strength of municipal solid waste (MSW) using back analyses of failed waste slopes. Shear strength of MSW is a function of many factors such as waste type, composition, compaction, daily cover, moisture conditions, age, decomposition, overburden pressure, etc. These factors together with non-standardized sampling methods, insufficient sample size to be representative of in-situ conditions, and limited shear displacement or axial strain imposed during the shear tests affect the test results and have created considerable scatter in reported test results. This scatter led the authors to pursue the back-analysis of failed waste slopes as a better means for estimating the shear strength of MSW. The back-analysis of failed waste slopes in the Gnojna Grora landfill in Poland, Istanbul Landfill in Turkey, Hiriya Landfill in Israel, and Payatas Landfill in Philippines are presented in this paper. Each of the landfill slope failures is reviewed and the results of the back-analyses presented. Finally, comparison of the recommended shear strength envelope of MSW and those by various researchers for the design of landfill slopes is presented.

INTRODUCTION

Shear strength of municipal solid waste (MSW) is a function of many factors such as waste type, composition, compaction, daily cover, moisture conditions, age, decomposition, overburden pressure, etc. These factors together with non-standardized sampling methods, insufficient sample size to be representative of in-situ conditions, and limited shear displacement or axial strain imposed during the laboratory shear testing have created considerable scatter in reported results. As a result, the authors utilize the back-analyses of failed waste slopes to estimate the shear strength of MSW.

BACK ANALYSES OF MSW LANDFILL FAILURES

The failed waste slopes are Gnojna Grora landfill in Poland, Istanbul Landfill in Turkey, Hiriya Landfill in Israel, and Payatas Landfill in Philippines. For each of the landfill slope failures the location of the landfill, the composition and properties of the waste, triggering factors for the instability, the location of the failure surface and leachate levels is discussed. To back-calculate the shear strength of MSW the slope stability software UTEXAS3 is used. The results of the back-analyses are summarized in Table 1.

Gnojna Grora landfill in Poland

Gnojna Grora Hill landfill, in Warsaw, Poland is described by Bouzza and Wojnarowicz (2000). The archeological work performed in one part of the landfill revealed that the landfill dates to the 14th century. It was an uncontrolled landfill where residents dumped their garbage. Therefore it is an old landfill without any liner or cover system layers.

Immediately after the reconstruction and renovation works of the old town was completed in 1965, cracks were observed in nearby buildings due to the movements in the landfill. Field investigations (the date of which are not reported) were performed, borings were drilled, pressuremeter tests conducted, and test pits excavated to determine the thickness of the waste, and some of its properties. Waste depth varied between 5 to 30 m and four types of material were mentioned. These materials are, Waste Fill (WF) near the ground surface, underlain by Upper Waste (UW), Intermediate Waste (IW), and Lower Waste (LW). Waste Fill is composed of large amounts of demolition debris together with old domestic MSW, and it is very heterogeneous. The observed slope movements were suspected to be occurring in this layer. All other MSW layers are relatively homogeneous. Slope stability analyses were performed for zone 1 of the landfill which had the steeper slopes. Because there is no information about the thickness of different waste layers and their properties in the in the aforementioned study, it is assumed that most of the waste in the landfill is Waste Fill, referred to as 'waste' or 'MSW' from here on.

The properties of the WF layer was difficult to measure by laboratory or field tests due to its heterogeneity. The unit weight of the waste material was estimated to be 17 kN/m^3 (because the waste is mixed with demolition debris) and the natural water content of the underlying waste with more percentage of MSW and less of the demolition debris is 28 to 80%.

Based on piezometer records the groundwater/leachate level was found to be 3 to 5 meters below ground surface (Fig.1). No geosynthetic liner system was installed prior to waste placement and thus the waste is in contact with native materials and groundwater. The slope did not experience a large slide but tension cracks developed in buildings on top of the landfill indicating the onset of sliding. Some of the observed building cracks may be caused by waste settlement rather than slope movement but tension cracks were observed indicating the onset of slope instability. Because the slope did experience some movement, the Factor of Safety (F.S.) was assumed to be near unity for the back-analysis. To back-calculate an effective stress friction angle, the MSW was assumed to exhibit a cohesion intercept (c') of 0 kPa. The back-calculated friction angle (ϕ') of the WF is 21º. The back calculated friction angle is reasonable considering the age of the waste. The landfill is estimated to be 300 years old. Therefore, the back-calculated shear strength of MSW would be expected to be close to the shear strength of a cohesive soil even though the WF is the newest of the waste layers. The average effective normal stress on the observed failure surface through the waste is 106 kPa and the corresponding back-calculated shear strength is 40.7 kPa.

Recent research suggests that the shear strength of MSW decreases with age, i.e., decomposition (Siegel et al. 1990, Brandl 1998, Gabr et al. 2002, Reddy and Bogner 2003, Gonzalez-Garcia and Espinoza-Silva 2003, Koelsch and Ziehmann 2004). There is a continuing debate on whether both cohesion and friction angle decrease with time, or only cohesion decreases. It is therefore reasonable to assume cohesion intercept is equal to 0 for a 300 years old MSW-demolition debris mix, and back-calculate the friction angle.

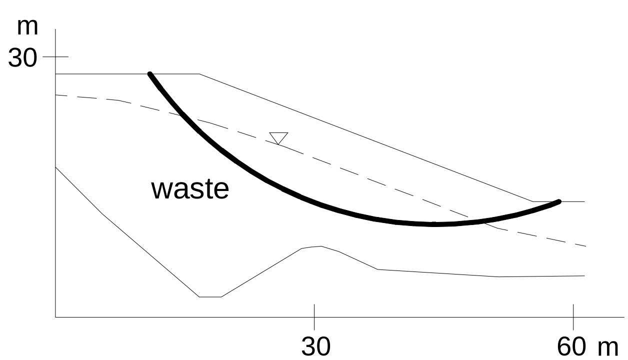

Fig.1 Approximate slope profile of Gnojna Grora landfill in Warsaw, Poland (from Bouzza and Wojnarowicz 2000)

<u>Landfill in Istanbul, Turkey</u>

This dumpsite, described by Kocasoy and Curi (1995) and Koerner and Soong (2000), is about 30 km away from the city center in Istanbul, Turkey. It was located on the upper portion of a tributary valley that discharges runoff into a local stream. The dumpsite has been in operation since 1976. Composition of the waste material, after removal of the recyclable material by scavengers, is estimated to be about 70% food remains/organics, 10% papers, 6% textile, 3% plastics, 3% metals (Kocasoy and Curi 1995). It is not known whether these values are determined by weight or by volume. MSW has high moisture content, and the subsoil is reported to be an impermeable strata. Maximum MSW slope height was about 45 m, with steep front slopes of up to 45 degrees or even more. The MSW was placed without any liner system. The waste is not compacted and is not covered with soil, except in areas where trucks were bringing the waste and dumping it. In those parts, the surface of the waste was covered with gravel and broken stones for easy traffic.

The catastrophic slope failure occurred on April 28, 1993 and resulted in 27 casualties and involved approximately 500,000 m^3 to more than 1,000,000 m^3 of waste (Fig. 2 shows a site plan view after the failure). Before the slide, 3 to 5 m of demolition debris and soil was placed on top of the uncompacted waste starting in mid-1992 to provide cover for the waste and to increase income obtained from the fees of dumping of demolition debris. Fires were known to be burning on the surface of the waste at several places during most of the year before the slide (Figs. 3 and Fig. 4 are photos from the landfill).

Streams of leachate were observed to be leaking from near the toe of the MSW slope and running down to the valley bottom. On the day of the slide a major explosion occurred due to compressed gases in the dumpsite. It is explained by Kocasoy and Curi (1995) that the explosion could not have been the main cause of movement of the waste. Heavy rains, and excessive leachate level built up within the old decomposed waste caused by water infiltrating from the adjacent surface water ponds were likely the triggering mechanism, together with recently placed demolition debris on top of the waste (Koerner and Soong 2000, Kocasoy and Curi 1995). The waste is assumed to be saturated (Koerner and Soong, 2000).

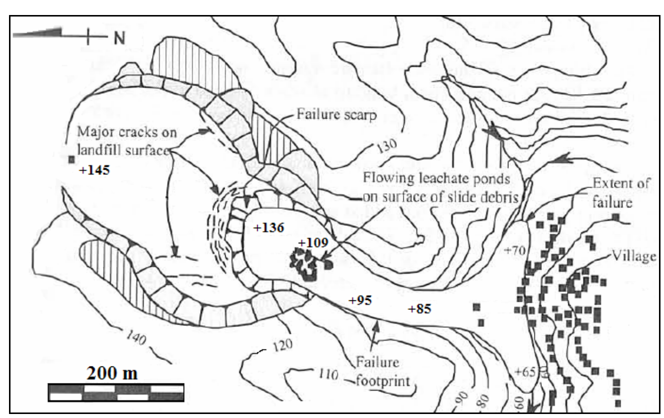

Fig. 2. Site plan view after slope failure (modified from Koerner and Soong 2000)

Below the waste mass is impermeable rock (see Fig. 5). Thus the failure surfaces analyzed pass only through the waste mass. A typical MSW unit weight of 11 kN/m^3 is assumed because no further information is available. The demolition debris was assumed to have a unit weight of about 19 kN/m^3. A small portion of the failure surface passed through the demolition debris at the top of the MSW, therefore the shear strength of the debris is assumed to be the same as the MSW for the purposes of back-calculation.

Fig. 3. Front slope of the MSW before slope failure showing surface fires and ssteep slope of the MSW (Kocasoy and Curi 1995).

Fig. 4. Tension cracks on top of landfill and steep backscarp in MSW after slope failure (Kocasoy and Curi 1995)

The approximate cross section of the landfill and the leachate level is shown in Fig. 5. The location of the actual failure surface in the field is not known. Search for the critical failure surface passing through the slope toe was performed and it is failure surface B in Fig. 5. This search resulted in a failure surface closer to the one described by Kocasoy and Curi (1995) than Koerner and Soong (2000). The figure in Kocasoy and Curi (1995) study didn't include a scale therefore the failure surface couldn't be used for this study. It is assumed that their failure surface is the most reliable because it is based on based on their observations at the site and data they obtained from the municipality of Istanbul. A noncircular failure surface is assumed by Koerner and Soong (2000) and shown as failure surface A in Fig. 5. Water level from Koerner and Soong (2000) is at about 21 m below the top of the waste. Fires and explosion are not modeled in the slope stability analyses.

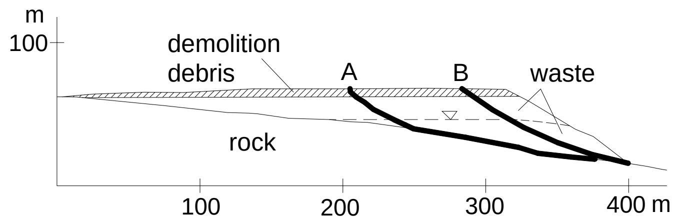

Fig. 5. Approximate cross-section of landfill and estimated slip surfaces (modified from Koerner and Soong 2000)

Because the observed failure surface at the time of failure is not known, both failure surfaces (A and B) in Fig. 5 are considered in the analyses. Weighted average values of the back-calculated average effective normal and shear stress pairs along the base of the failure surface are (σ'_n, τ) = (170, 62 kPa) and (76, 68 kPa) for failure surfaces of A and B respectively. A circular failure surface (B) passing through the slope toe results in lower factors of safety than the noncircular failure surface (A) if one uses the same shear strength parameters for both cases. If a higher leachate level (leachate level at about 10 m below the top of the waste surface, not shown in Fig. 5) would be assumed (Kocasoy and Curi 1995, Koerner and Soong 2000), the corresponding backcalculated pair is (54, 77 kPa). High leachate levels are believed to be more representative of the field conditions, therefore an average value of the two circular analyses (σ'_n, τ) = (65, 72.5 kPa) is listed in Table 1.

<u>Hiriya Landfill in Israel</u>

The Hiriya waste dump is located just east of Tel-Aviv, Israel, in an open area at the convergence of the Shappirim river (to the south) and the Ayalon river (to the north). In some areas the edge of the waste is less than a few meters from the rivers. The river channels pose a threat to landfill stability under flood condition, and from normal erosion. The dump has been used for the disposal of the municipal solid waste for the greater Tel-Aviv area for decades, as well as the dumping of building waste and some industrial waste. The 'mountain' has grown to tremendous proportions with the footprint of the waste covering 40 hectares, and containing more than 16 million m^3 of waste. The landfill reaches a height of 60 m above the surrounding level ground, with the slopes having slope angles of 45 degrees or more. This landfill was in use from 1952-1998 and was Israel's largest landfill. The landfill does not have an engineered bottom liner, final cover, or leachate and gas control systems (Isenberg al. 2004).

Side slopes of Hiriya landfill range from 1.3H:1V to 1.6H:1V with an average 1.44H:1V. The slopes are covered with a thin irregular soil cover, with waste exposed in some areas. Slope heights range from 43 to 64 meters, averaging 56 meters as measured above surrounding level terrain. As a result of the steep slopes, the lack of vegetation, drainage and erosion controls, the landfill has experienced small and large instability problems. In the winter of 1997-98 a major slope failure (Figures 6 and 7) occurred in the northern face of the dump, following a period of heavy rain, and the Ayalon River was blocked for some days.

Fig. 6. 1997 Hiriya landfill slope failure (Isenberg, 2003)

Fig.7. Top of Hiriya landfill after slope movement (Isenberg 2003)

In the year before the failure, the top of the landfill was covered with a layer of earth, consisting of varying quantities of clay, sandy clay and clayey sand, so as to reduce rainwater infiltration into the waste. More than 20 boreholes, 10-47 meters deep, were drilled, for gas generation and monitoring leachate levels (Klein, 2003). Drilling was terminated slightly below the saturated leachate level. The borings encountered MSW mixed with soil. At shallow depths the waste was partially decomposed and the degree of decomposition and moisture content increased with depth. The moisture content (on a dry weight basis) varied from 13% to 67%, averaging 38% (Isenberg et al. 2004). Temperatures of the waste measured during drilling were 40 to 60ºC. Leachate levels were encountered at depths of 7 to 23 meters below the plateau surface. Leachate seepage was apparent from the numerous small individual seeps located near the toe of the landfill.

Figure 8 shows the cross section of Hiriya landfill, and the results of the back-analysis are presented in Table 1.

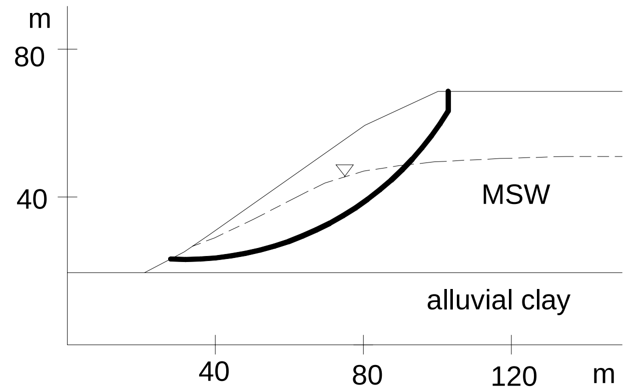

Fig. 8. Cross section of the Hiriya landfill (modified from Isenberg 2003)

<u>Payatas Landfill in Philippines</u>

The Payatas landfillis located in the northeast of Metro-Manila within the boundaries of Quezon City in Philippines. It has been in operation since 1973 and about 1500 tonnes of MSW are placed since 1996. Following placement, waste is pushed over the brink of the top slope so that it makes a steeper slope which creates more space for further waste on the top. The height of the landfill was about 30 m before the failure (Fig.9).

Fig. 9. Payatas landfill slope failure in 2000. (Kolsch and Ziehmann, 2004)

The slope failure occurred on July 10, 2000. About 1.2 million m^3 of MSW slid and caused more than 250 fatalities. Waste and debris had covered an area of 30,000 m^2 in front of the toe of the slope (Kolsch and Ziehmann 2004). The landfill did not have a liner system. The waste material contained high proportion of plastics and organics, and less of metals, papers, and glass due to recycling by scavengers. Landfill had large waste to soil ratio. These factors and little or no compaction resulted in a low density waste (Merry et al. 2005).

The exact mechanism of failure is not clear, but several factors probably contributed to the failure. These include heavy rains (68 cm rainfall caused by two typhoons that occurred in the area in the two weeks prior to the failure, see Fig.10) leading to the likely saturation of the entire waste mass; building up of waste that caused the side slopes to be steeper than recommended (1.5H:1V at the time of failure); ponding of water on the top of the slope; the construction of drainage trenches at the top of the slope to drain this ponding water; construction of a 2 to 3 m deep drainage ditch at the toe of the slope; and the potential build-up of landfill gas (Merry et al. 2005).

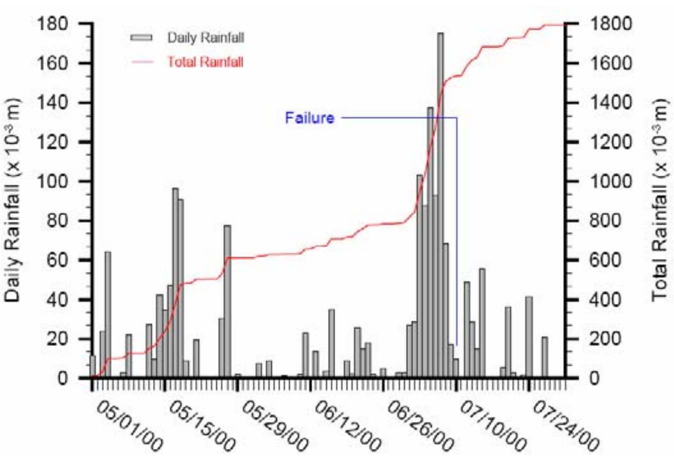

Fig.10. Precipitation record at Quezon City Weather Station in Philippines during May thru July 2000 (Merry et al. 2005).

Merry et al. (2005) assume that the failure surface passes through the waste and the underlying natural clay (failure surface A in Fig. 11). In this study, using the failure surface A in Fig. 11, and using the unit weights of MSW and clay substrata, and shear strength of clay substrata given by Merry et al. (2005), a cohesion of 20 kPa, and a friction angle of 32 degrees are back-calculated for the MSW. The back-analysis is correct if the location of the failure surface used in the analysis represents the observed failure surface in the field, and the shear strength of the underlying clay strata is known. The weighted average effective normal stress along the portion of the failure surface that passes through the waste is 62 kPa and the corresponding shear strength is 59 kPa. Although Merry et al. (2005) did not back-calculate the shear strength of the MSW, they suggest using a cohesion of 19 kPa and a friction angle of 28 degrees based on Geosyntec (1998) and Kavazanjian (2001). These parameters lead to an average effective normal stress along the failure surface through the MSW of 62 kPa and a shear stress of about 52 kPa, which is in agreement with the back-calculated shear strength parameters in this study.

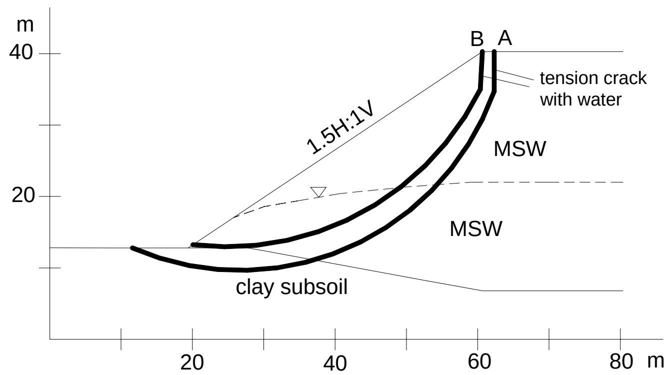

Fig. 11 Cross section of Payatas landfill slope failure (modified from Merry et al. 2005)

To account for the excess pore pressure that may have been generated by the buildup of landfill gas within the saturated waste, Merry et al. 2005 suggest using a unit weight of fluid that is greater than that of water. For a given shear strength of the waste, as the unit weight of the fluid, $\gamma_{fluid,equivalent}$, is raised, the Factor of Safety decreases. In the back-analysis performed in this study, pore pressures that might have increased due to the gas generated in the landfill were not considered. If these increased pore pressures could have been estimated and included in the analyses, a higher back-calculated shear strength for the waste would have been obtained.

Table 1. Landfill case histories analyzed

Landfill	γ , γ_{sat} (kN/m^3)	Max. height (m)	σ'_n * (kPa)	τ (kPa)
Gnojna Grora, Warsaw (W)	17	26	106	40.7
Istanbul, Turkey (I)	11	45	65	72.5
Hiriya, Israel (H)	8 , 9	60	32	46
Payatas, Philippines (P)	10 , 14	33	62 (or 45)	59 (or 35)

* σ'_n = weighted average of the effective normal stresses acting on the failure surface through the waste

The picture of the observed failure in the field is shown in Fig. 9. Although the exact location of the failure surface is not known, looking at the near vertical scarp, the thickness and movement of the displaced waste mass, and no sign of bulging at the toe to indicate a deeper seated failure surface through the underlying clay, the failure surface in the field seems to be shallower and mostly through the waste than failure surface A in Fig. 9. Therefore a critical circular failure surface was found using UTEXAS3 (shown as failure surface B in Fig. 11). This failure surface, together with unit weights of MSW from Merry et al. (2005), were used in a back-analysis. The weighted average effective normal stress along the failure surface is 45 kPa and the corresponding shear strength is 35 kPa.

Figure 12 shows a summary of back-calculated MSW shear strengths given in Table 1, together with other four landfill slope failures back-analyzed and reported by Stark et al. (2000) and Eid et al. (2000). These previously analyzed cases are located in New Jersey (NJ), Maine (M), Cincinnati (C) and Eastern Ohio (EO) landfills. Also in Fig. 12 various published shear strength envelopes for MSW are presented. A recommended strength envelope is developed by Stark et al. (2007) based on an extensive literature review on the laboratory and field measurements of shear strength of MSW, which are shown as open circles in Fig. 12. The cases analyzed in the current study have low effective normal stress range. It would be interesting to look at cases where the effective normal stress on the failure surface along the MSW is large. This would aid in determining the MSW shear strength at large effective stresses. The interest in mega-landfills with effective stresses in the waste up to more than 1,000 kPa is necessitating MSW shear strength parameters at high effective stresses.

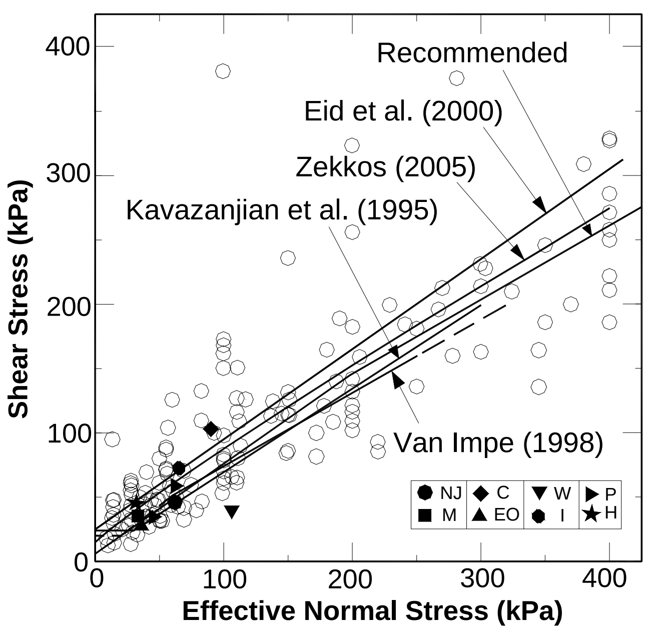

Fig.12. Summary of back-calculated MSW shear strength shown with four other landfill slope failures, and published failure envelopes (modified from Stark et al. 2007)

CONCLUSIONS

Back-analyses of landfill slope failures are important and useful for understanding the drained shear strength of MSW.

Location of the landfill, age and composition of the waste, shear strength properties, slope geometry, properties of the substrata, location of the slip surface, pore pressure conditions should be defined in reporting a landfill case history to increase its value to the profession.

Parameters (existing height and slope of the landfill, observed slip surface, and whether it failed through the waste only or it passed through the waste and other materials, pore pressure conditions in the landfill, unit weight, landfill covered or not, intermediate soil cover layers are used or not, triggering factors etc.) should be defined as accurately as possible to conduct a good back-analysis. Therefore it is crucial to perform an extensive field reconnaissance survey immediately after the failure, including recording eyewitness accounts, and records of recent history of fill placement, rainfall etc.

More case history analyses and laboratory and field testing of shear strength is needed on fresh waste and degraded (old) waste to better understand its mechanical behavior and shear strength. In the interim, a bilinear shear strength envelope is recommended based on Stark et al. (2007) and this study. The recommended bilinear envelope is, c' = 6 kPa and $\phi' = 35^o$ degrees for effective normal stresses less than 200 kPa, and c' = 30 kPa and $\phi' = 30^o$ degrees for effective normal stresses more than 200 kPa.

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