The catastrophic landfill flowslide at Hongao dumpsite on December 20, 2015 in Shenzhen, China

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Abstract A catastrophic flowslide occurred at Hongao dumpsite on Dec 20, 2015 in the Guangming New District of Shenzhen, China. The flowslide caused 73 causalities with 4 missing and damaged 33 buildings. In the absence of extreme weather condition and seismic activity, the causes of the failure were analyzed on the basis of multi-temporal remote sensing images, site investigation, in-situ tests, laboratory tests, and numerical analyses. Site investigations showed that the volume of the flowslide deposit was $2.32 \times 10^6$ m$^3$ and the volume of dumpsite filling was $6.27 \times 10^6$ m$^3$ at the time of the event, which is 3 times greater than the design capacity. The flowslide was characterized by high travel velocity and long runout distance. The displaced material was primarily silty soil in mixture of construction debris, andfilling is the most common method of MSW disposal (Brunner and Fellner, 2007), transforming an abandoned quarry to a dumpsite conserves resources by reclaiming the quarry space and provides MSW storage solution (Zou, 2016). The landfill slope stability is critical to the MSW management, and thus the selection and design of landfills require engineering assessment on both slope stability and environment impact.

1 Introduction

Owing to the population growth and industrialization, rapid increase in the accumulation rate of municipal solid waste (MSW) poses challenges in MSW management and urban planning (Huang and Cheng, 2016). As landfilling is the most common method of MSW disposal (Brunner and Fellner, 2007), transforming an abandoned quarry to a dumpsite conserves resources by reclaiming the quarry space and provides MSW storage solution (Zou, 2016). The landfill slope stability is critical to the MSW management, and thus the selection and design of landfills require engineering assessment on both slope stability and environment impact.

The porosity and water content of MSW is typically high in an unregulated landfill because of inadequate drainage system, and therefore some failures of MSWs exhibited fluid flow behave, i.e. flowslide, with extremely high mobility (e.g. Dai et al., 2016; Huang and Cheng, 2016). Landfill slope failure in MSW dumpsite can be found in previous studies, wherein 6 reported cases between 1993 and 2005 had resulted in approximately 500 deaths and significant economic loss (Blight, 2008; Blight and Fourie, 2005; Eid et al., 2000; Kjeldsen and Fischer, 1995; Kocasoy and Curi, 1995; Merry et al., 2005; Mitchell et al., 1990). The deadliest event in history killed 278 people in Manila, Philippines in 2000, and the second deadliest event buried 71 houses and killed 143 people on Feb 21, 2005 at Leuwigajah dumpsite near Bandung, Indonesia (Lavigne et al., 2014).

Shear strength of MSW is a function of various parameters, including the type and composition of the waste, disposal rate, water content, surcharge, and compaction (Eid et al., 2000; Huvaj-Sarihan and Stark, 2008). The movement of waste failure are complex and still poorly known due to the lack of field monitoring data. Previous studies have focused on empirical methods (e.g. Blight and Fourie 2005; Srour 2011), laboratory experiment, and numerical analyses. An energy balance approach was presented to estimate the maximum flow velocity of MSW slope failure (Blight and Fourie, 2005). The empirical-statistical method still has widespread use in practical applications, but the accuracy is commonly model dependent (Dai et al., 2016; Huang and Cheng, 2016). The mixture of peat, kaolinite, and quartz sand were used in laboratory tests to study the interplay between water content of filling and failure of MSW (Dai et al., 2016). With an increased water content in the MSW, the mobility increases with the maximum and final thickness of deposit decrease.
Numerical simulation has been widely used in landfill slope stability analysis (Chang, 2002, 2005; Chugh et al., 2007; Huang and Cheng, 2016), and some of the more advanced methods were recently employed, such as the smoothed Particle Hydrodynamics (SPH) method for runout distance analysis of two typical landfill flow slides occurred in Sarajevo and Bandung (Huang et al., 2013).

A MSW landfill consists of mainly construction and demolition waste failed at the Hongao dumpsite in the Guangming New District of Shenzhen, China at approximately 11:40 AM on Dec 20, 2015. The flowslide destroyed 33 buildings and accommodations in the industrial park, and resulted in 74 causalities with 4 missing. No extreme weather conditions or seismic activity were reported at the time of the event. This article investigates the characteristics and causes of failure. Field investigation, in-situ tests, Unmanned Aerial Vehicle (UAV) stereo-measurements, and remote sensing images were used to analyze the material properties and characteristics of the failure. Laboratory tests and numerical analysis were performed to better understand the failure mechanism.

2 Data and Methodology

A series of satellite images of the site was obtained from Google Earth between Nov 2002 and Feb 2016. Two high resolution pre-/post-failure images were acquired by satellite image and aerial photo. Topography map of the pre-disposal site was provided by the local government, of which the design was submitted by a third-party consulting firm in Dec 2013. Based on the official report on the incident, the design capacity was $4 \times 10^6$ m$^3$ with 9 slope benches at an average ratio of 1:2.5. Field and UAV measurement indicate that the total volume of dumpsite filling was $6.27 \times 10^6$ m$^3$ before failure ($5.83 \times 10^6$ m$^3$ by Yin et al., (2016)). The Digital Orthoimage Map (DOM) and Digital Surface Model (DSM) were reconstructed from aerial photos, and a topographic map for the post-sliding dumpsite was generated from the DSM at the scale of 1:1000.

Structured terrains in the study site were excavated for vehicles which consisted 9 benches and 3 platforms. Topographic map for the pre-sliding dumpsite was derived by combining the original, pre-sliding, and post-sliding slope images, which also provided an estimation on the volume and depth of the pre-/post-sliding landfill.

The preliminary field investigation was conducted on Dec 23, 2015, with a follow-up investigation on Jan 21-25, 2016. Field investigations evaluated the topographical, geological, and groundwater conditions. Density, water content, and permeability of the displaced material was measured in-situ.

Undisturbed and reconstituted soil samples were obtained and used for laboratory tests, including weathered silty soil and waste fillings. Grain size distribution was conducted using wet sieve analysis. Maximum dry density and optimum water content of the displaced material were determined by standard compaction test. The shear parameters of the displaced material (c and $\phi$) were obtained by triaxial compression test. In order to back-analyze the failure of dumpsite, input parameters for the material properties and model geometry were measured on site or in the laboratory. The mobilized shear strength was back-calculated using the conventional limit equilibrium method (LEM) to demonstrate the complexity of such progressive failure involving liquefaction and post-failure landslide propagation.

3 Geological and Climatic Setting

The location of the site is shown in Fig. 1a, of which the pre-failure image was obtained by Pleiades image on Dec 18, 2015 (2 days before failure) and the post-failure aerial image was taken by UAV (3 days after failure) as shown in Fig. 1b and 1c. The dumpsite was located 23 km north of Shenzhen and 5 km south of the Guangming New District (E113°56′5″, N22°42′44″). The dumpsite was surrounded by three ridges with a free face excavated in a mild slope gradient. The bedrock of the dumpsite is mainly Cretaceous granite rock (Fig. 2a and 2b). The landfill consisted of construction waste with silty soil, clay, rock, and gravel (Fig. 2 and 3).
Fig. 1 Location of the study site and images of the pre-/post-failure dumpsite. The boundary of the flowslide is indicated in red. a Location of the study area; b Pleiades satellite image (Dec 18, 2015); c Aerial photo of the flowslide with locations of samples for laboratory test (Dec 23, 2015).

The study area belongs to the zone of subtropical monsoon climate with an average temperature of 22° and an average annual precipitation of 1500 mm concentrating between April and September (> 85 % of the annual rainfall) (Zhang et al., 2006). Based on the rainfall data from the nearby Tangjia Rainfall Station (Jan 1, 2013 - Dec 20, 2015) as shown in Fig. 4, a heavy rainfall occurred on Dec 9, 2015 with accumulated precipitation of 67.8 mm. The area of catchment (Fig. 5a) was estimated as $4.7 \times 10^5 \text{ m}^2$, which was nearly 3 times greater than the area of the dumpsite ($1.6 \times 10^5 \text{ m}^2$). Drainage system was implemented to collect surface run-off by the drainage pipes installed at a higher elevation above the dumpsite and diverted into the peripheral drainage channels. Field evidences suggested that the drainage system was abandoned, possibly due to the lack of maintenance (Fig. 5b and 5c). With the inadequate drainage, ingress of rainwater in the dumpsite was permitted (Fig. 6c), in addition to the concentration of surface run-off and groundwater into the dumpsite. The infiltration rate of the waste filling was estimated as $5.26 \times 10^{-6} \text{ cm/s}$ by field double ring infiltrometer. Low permeability and inadequate drainage resulted in high water content and high pore water pressure in the dumpsite. The groundwater in the study area were mainly bedrock fissure water and Quaternary pore water (Fig. 6).
Fig. 2 Photos of the rock and soil specimen collected from the displaced material. a Exposed granite rock near the flowslide scarp; b Close view of the granite rocks; c Displaced material in the zone of depletion; d Materials in the flowslide source area.

Fig. 3 Particle size distribution of the waste filling (with sieve sizes of 20, 10, 5, 2.0, 1, 0.5, 0.25, 0.10, 0.075, 0.025, 0.01, 0.005, 0.002, and 0.001 mm)

Fig. 4 Monthly rainfall and cumulative precipitation between 2013 and Dec 2015.
Fig. 5  

a  An overview of the catchment area (Google Earth image);  
b  The surrounding peripheral drainage channel of the dumpsite was abandoned;  
c  Damaged drainage pipes were not repaired and failed to divert surface run-off into the peripheral drainage channel. Surface run-off concentrated in the waste filling.

Fig. 6  

a  Groundwater overflow in the rock fissures near the flowslide crown;  
b  Water accumulation in the flowslide crown;  
c  Water accumulation in the source area;  
d  Surface run-off and infiltration contributed to groundwater seepage.
The development of dumpsite and landform changes are presented by a series remote sensing images taken between 2002 and 2016 (Fig. 7). It is postulated that the quarry was active between 2002 and 2008. However, formation of ponds of various sizes at different spatial and temporal locations in the quarry indicated low permeability and the lack of drainage network (Fig. 7b, 7c, and 7d). The quarry was abandoned in or before 2008 as shown in Fig. 7c (Feb 20, 2008) with a small amount of waste filling in the pit. A large-scale pond was later formed due to groundwater and rainfall accumulation (Fig. 7d). The depth of the pit was over 100 m before the routine disposal activities. A service road was excavated between two rock hills exiting the quarry and formed a small-scale gully cross the pit longitudinally (Fig. 7a, 7b, 7c, and 7d). Additionally, a small-scale platform (the office area of the abandoned quarry) was excavated at the immediate downstream of the quarry exit. The gully was covered by the construction of a multi-benched retaining slope between the rock hills as waste disposal continued throughout the process (Fig. 7e). Disposal activity started possibly in 2014 (Fig. 7e and 7f), and the accumulation rate of the waste filling was considerably fast (Zou, 2016).

An unpaved road was excavated on the east of the quarry connecting the crest and toe of the hillslope as shown in Fig. 7e. A large number of trucks can be seen transporting waste filling to the dumpsite. A large amount of construction waste was dumped in the pit with 4 slope benches built at the exit of the pit (Fig. 7e). The volume of the waste increased significantly with the 1st and 2nd benches completed in late 2014, and the 3rd to 4th benches were still under construction until Jan 2015 (Fig. 7f).

The rapid accumulation and the total volume of the waste filling received some attentions before failure. An environmental assessment provided by a third-party consulting firm warned the erosion at the site and its influence on the slope stability in Jan 2015 (Zou, 2016). The disposal activity was ceased for a while as no trucks were seen in the image (Fig. 7f), and it was verified by the interview with the local inhabitants. The disposal activity was resumed in or before April 2015, and the 4th bench was completed by then (Fig. 7g). By comparing Fig. 7g and Fig. 7h, significant modifications on the landform occurred with a major increase in the volume of the waste filling as the landfill was close to the crest of the pit. The waste filling appeared flattened and disposal activity was intense as more than 20 trucks were found in Fig. 7h. A total of 8 slope benches were completed with surface drainage channels installed on the hillslope, and the 9th bench was still under construction before failure. The thickness of the waste filling was around 90 m with a volume of $6.3 \times 10^6$ m$^3$ (Fig. 8a) by extracting the difference between the pre- and post-filling DEMs.
Fig. 7 Multi-temporal remote sensing images of the study area. a 08/31/2002; b 02/20/2008; c 08/30/2010; d 11/25/2013; e 11/17/2014; f 01/23/2015; g 04/14/2015; h Pleiades (12/18/2015); f Aerial (12/23/2015). Image a to g were obtained from Google Earth. A service road was excavated at the exit of the quarry with a width of nearly 70 m (see image a, b, c, and d).
The thickness distribution of the dumpsite based on the pre-/post-disposal DEMs; b The elevation variations of the dumpsite estimated between the pre-/post-sliding DEMs.

5 Flowslide Characteristics

The flowslide area can be divided into the source area and the flow-accumulation area as shown in Fig. 9. The source area was the abandoned quarry pit. The mass slid in the direction of 340° with the maximum traveling distance of 1203 m. The maximum thickness of the remaining material in the source area was 41.51 m with an average of 20.5 m (Fig. 9 and Fig. 10). The maximum deposit thickness in the flow-accumulation area was 18.2 m with an average of 8.21 m (Fig. 8b). The geometry of a flowslide can be expressed in length (L), height (H), width (W), and area (S) (Legros, 2002; Scheidegger, 1973). The geometric parameters are indicated in the simplified flowslide geometry in Fig. 11 with values tabulated in Table 1, Table 2 and Table 3.
Fig. 9 The topographic map of the flowslide.

Fig. 10 The geological and topographical cross section (profile line 1-1’).
Fig. 11 Simplified illustration of the flowslide geometry. L: runout distance; H: elevation; W₁: scarp width; W₂: max width of the source area; W₃: frontal width of the source area; W₄: width of the shear crack; W₅: max width of the flow-accumulation area; L₁: horizontal length of the scarp; L₂: horizontal length of the mild slope; L₃: horizontal length of the steep slope; L₄: horizontal length of the flow-accumulation area; H₁: height of the scarp; H₂: height of the mild slope; H₃: height of the steep slope; H₄: height of the flow-accumulation area; H₅: thickness of the toe of the flowslide; Φ₁: extension angle; Φ₂: slope gradient of the scarp; Φ₃: mild slope gradient; Φ₄: slope gradient of the source area; Φ₅: steep slope gradient; Φ₆: slope gradient of the flow-accumulation area; Φ₇: slope gradient of the foundation area

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<th>Parameter</th>
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<th>H (m)</th>
<th>W₁ (m)</th>
<th>W₂ (m)</th>
<th>W₃ (m)</th>
<th>W₄ (m)</th>
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Table 2 Geometric parameters of the flowslide (Angle)

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<th>Φ₂ (°)</th>
<th>Φ₃ (°)</th>
<th>Φ₄ (°)</th>
<th>Φ₅ (°)</th>
<th>Φ₆ (°)</th>
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Table 3 Geometric parameters of the flowslide (Area and Thickness)

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<th>Area B</th>
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<tr>
<td>V (m³)</td>
<td>3.95×10⁶ (remaining)</td>
<td>2.34×10⁶</td>
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<td>Tₘₐₓ (m)</td>
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<td>18.82</td>
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<tr>
<td>Tₐᵥₑ (m)</td>
<td>20.49</td>
<td>8.21</td>
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5.1 Source Area

The source area of the flowslide is the quarry pit with a length \( L_{1+2+3} \) of 540.30 m and a width \( W_2 \) of 399.32 m. The area of the source area \( S_A \) is \( 11.35 \times 10^4 \) m\(^2\) and the height \( H_{1+2+3} \) was 68.5 m with an apparent dip \( (\Phi_4) \) of 7.62° (Fig. 12 and Table 1). The volume of the mobilized material from the source area was \( 2.32 \times 10^6 \) m\(^3\) and the remaining volume was \( 3.95 \times 10^6 \) m\(^3\). The maximum thickness of the source area \( (T_{A,\text{max}}) \) was 41.5 m with an average thickness \( (T_{A,\text{ave}}) \) of 20.5 m. The west of the main scarp has steep slope gradient with a mild gradient on the east. The height of the steep scarp was 25-47 m, while the height of the mild scarp was 10-20 m. The geological cross section (profile line 2-2') is shown in Fig. 12c. The maximum thickness of the source area was 40.65 m with an average of 34.83 m. The overview of the source area and surface cracks are shown in Fig. 12a and 12b, respectively.

![Image of the source area](image)

Fig. 12a An overview of the source area (image taken at the flowslide crown facing north). A large opening \( (W_{4,1}) \) formed due to the failure of the retaining slope (the exit of the original quarry pit), which provided passage for the flowslide and permitted rapid release of kinematic energy; b Surface cracks were developed due to unloading near the crest and the flanks of the flowslide; c The geological cross section (profile line 2-2') of the source area.

Pore water pressure increases as rainwater and groundwater ingress in the dumpsite. The lack of drainage resulted in waste filling saturation in the basal zone of the dumpsite which appeared as the final sliding bed of the flowslide (Fig. 12a). A large amount of silty soil was observed in the source area. The water content of the remaining materials in the source area is 17.3%-42.4% (6 sampling locations). The materials on both sides of the source area were mobilized due to the debuttressing effect as the waste filling in the lower portion of the dumpsite slid into the downstream industrial park, and in consequence, caused collapse of the dumpsite. Step-like steep scarp was formed on the west of the main scarp with tensile cracks developed on the rear edge as well as both sides of the scarp.

The presence of the aforementioned rock ridges formed unfavorable topography for the stability of the retaining slope as it promoted a narrow gully for groundwater flow (Fig. 7c and Fig. 9). As a result of the retaining slope failure, a large opening was formed at the elevation of 73.7 m (between the front edge of the steep scarp and the rear edge of the flow-accumulation area) as shown in Fig. 9, Fig. 10, and Fig. 12a. The displaced material was stratified in the middle part of the flow-accumulation area during the high-speed sliding. The shape of the failed retaining slope was half-elliptic-like with a width \( (W_{4,1}) \) of 217.7 m and a...
height ($H_b$) of 27.5 m (Fig. 13a and 13c).

Fig. 13 a The half-elliptic-like opening of the failed retaining slope; b Details on the scratch caused by the mass movement; c Geological cross section (profile line 3-3') of the failed retaining slope located at the original quarry pit between the two rock hills.

5.2 Flow-Accumulation Area

The fan-shaped flow-accumulation area mantled the retaining slope of the dumpsite as well as a large part of the industrial park as shown in Fig. 12a. The water content of the displaced material was 32.1% - 37.2% (3 sampling locations). The original slope gradient of the industrial park ($\Phi$) was 1.36° along the profile line 1-1' as shown in Fig. 12 and Table 1. The area located immediately downstream of the dumpsite was relatively flat with no major construction except a pond (area approx. 3600 m$^2$) and a channel (width: 7 m and length: 130 m). Most of the industrial structures were located on the east, west, and north side of the pond as shown in Fig. 9, and thus created an ideal flow channel for the flowslide. The failure of the rock retaining slope resulted in an opening for the waste fill movement which subsequently destroyed downstream buildings (Fig. 14 and 15).
Fig. 14 The overview of the flow-accumulation area with elevations (China News Agency); b Geological cross section (profile line 4-4’) of the flow-accumulation area. The length ($L_4$) was 664.4 m and the width ($W_3$) was 218.3 m with the width of the front edge ($W_5$) of 592.3 m. The area of the flow-accumulation area is $28.51 \times 10^4$ m$^2$, and the elevation difference ($H_4$) was 35.7 m with an apparent dip ($\Phi_6$) of 3.08°. The average thickness in the flow-accumulation area was 8.21 m with the maximum thickness of 18.82 m (Fig. 12c, Table 1, and Table 3).

Fig. 15 The damage of buildings in the industrial park (China News Agency)
6 Back-Analyses of the Flowslide

6.1 Flowslide Movement

The high sliding velocity and long-runout distance of the flowslide may be related to a more diffuse failure due to liquefaction near the base of the slope. Evidences of localized liquefaction can be found at several locations (Fig. 16). Empirical correlation was used to back-calculate the flowslide travel velocity on the basis of its geometrical characteristics. Laboratory tests and numerical analyses were performed to better understand the failure process.

The source area of the flowslide was in the elevation of 142 m and the horizontal runout distance (L) was approximately 1203 m with an elevation difference (H) of 111 m (Fig. 11). The velocity of a high-speed landslide can be estimated by

\[ v = \sqrt{2g(H - fL)} \]  

(Scheidegger, 1973), where \( v \) is the sliding velocity (m/s), \( g \) is the gravitational acceleration (m/s²), \( H \) and \( L \) are the elevation difference and horizontal distance (m) between the crown and toe of the flowslide, respectively. \( f \) is the equivalent friction coefficient referred as the ratio of height and runout distance of the flowslide \( (f = \frac{H}{L}) \). The equivalent friction coefficient of the flowslide was 0.092. The sliding velocity was back-calculated and presented in Fig. 17.

Based on the back-calculated velocity profile, two sharp increases were identified, including the initiation of the waste filling near the steep scarp and the acceleration of the flowslide when exiting the dumpsite. The sliding velocity was increased to 15.17 m/s as waste filling reached the bottom of the steep scarp. The velocity displaced material gradually decreased to approximately 13 m/s before accelerate to the maximum velocity of 25.15 m/s as it reached the opening of the quarry. The second sharp acceleration was followed by the rapid dissipation of kinematic energy and reduction in velocity after reaching the flow-accumulation area (elev. 50 m). It was estimated that the sliding velocity was reduced to 15.68 m/s when it made contact with the downstream buildings, and then ceased moving due to the obstruction. The geometry and velocity exhibited clear characteristics associated with high-speed long-runout flowslides.
6.2 Numerical Analyses

Basic material properties were obtained from in-situ and laboratory tests. Dry density of the waste filling was 1.25-1.48 g/cm$^3$ with a void ratio of 0.83-1.31. Standard compaction tests suggested that the optimal water content of 15.3% with highest dry density of 1.79 g/cm$^3$. The surface of the filling was in loose state with the degree of compaction of 69.8%-82.7%. Based on the undrained shear test, the $c$ and $\phi$ of the filling were 4.7 kPa and 31.9$^\circ$, respectively, and thus the friction angle was considerably higher than the gradient of the slope. No strain-softening was observed in the saturated specimen under triaxial tests.

Numerical simulation for diffuse failure involves liquefaction and post-failure propagation is challenging (Take and Beddoe, 2014), and the conventional LEB is typically not applicable for analyzing propagation of landslide originating from diffuse failure induced by liquefaction (Cascini et al., 2009, 2013). Back-analyses was performed using SLOPE/W with Morgenstern Prince Limit Equilibrium Analysis under the assumption of a fully saturated basal zone in the dumpsite before failure. The back-analyses using LEB method typically set the FoS to unity for back-calculating the mobilized strength. The final sliding surface and a hypothetic groundwater level were added to determine the shear strength in an iterative approach with estimated dry density of filling of 1.65 g/cm$^3$ (Fig. 18), however the back-calculated friction angle was significantly less than the experimental result.

![Fig. 18 Back-calculated mobilized strength by using 2D numerical model with LEM.](image)

The implausible mobilized strength shows that back-analyses using LEB is not applicable to the flowslide, as the failure may involve liquefaction with subsequent progressive failure and post-failure propagation. Notwithstanding the complex progressive mechanism and over-simplified force equilibrium method, the misleading results of the back-analyses can be directly ascribe to the misuse of static pre-shearing pore-water pressure with measured final sliding surface. Such erroneous back-analyses strategy was discussed in detail by Take et al., (2004). It was postulated that the failure may be initiated at a relatively shallow depth near the base of the slope as a result of static liquefaction, and followed by progressive backward mobilization of the fillings in the dumpsite.

The exact reason of excess pore water pressure remains unclear but it was a key factor in causing the failure (Ouyang et al., 2016). The excess pore water pressure can be induced by: (1) rapid surcharge on the dumpsite while pore water pressure cannot dissipate sufficiently fast, and/or (2) the waste filling was loosely packed with large pores in the meso-structure, and the collapse of the structure lead to the shrinkage of pores, which resulted in the excess pore water pressure. Since the permeability of the waste filling was considerably low, it may result in saturation in the basal zone of the waste filling. As no clear drainage passage were found in the filling, the displaced material may remain undrained ‘at failure’.

7 Conclusion

The Hongao dumpsite failure is of direct interest to the scientific community due to its complex progressive failure mechanism and significant societal impact. The flowslide was investigated here to better understand its characteristics and mechanism. The flowslide is divided into the source area and the flow-accumulation area. The volume of the source area was $2.32 \times 10^4$ m$^3$ with the average thickness of 20.5 m (max. 41.5 m).
The volume of the fan-shaped flow-accumulating area was $2.34 \times 10^6 \text{ m}^3$ with the average thickness of 8.2 m (max. 18.8 m). The volume expansion coefficient of the flowslide was 1.007. The maximum sliding velocity of the flowslide was 25.15 m/s at the opening of the dumpsite, and reduced to 15.68 m/s as it reached the industrial park and ceased moving due to the obstruction of buildings.

The flowslide was characterized by high-speed and long runout distance, which may be related to the (1) suitable topography with a height of 124 m between the crown and toe with large potential energy needed for high-speed and long runout flowslide, and (2) the low permeability and lack of drainage in the waste filling in the dumpsite with impermeable bedrock resulted in groundwater stagnation, and thus high pore-water pressure. Additionally, the volume of the waste filling was estimated as $6.27 \times 10^6 \text{ m}^3$, which was three times larger than the design capacity of the dumpsite. The failure of the retaining slope constructed between two bedrock ridges formed a narrow opening for the flowslide, which facilitated the sudden release of kinematic energy generating high sliding velocity and long travel distance.

The failure mechanism remains unclear yet it was clear that a more diffuse failure occurred with liquefaction and post-failure propagation of the flowslide. The inapplicability of the LEM demonstrated the complexity of the mechanism by yield erroneous mobilized strength, which also indicates the predicament of simulating liquefaction-induced slope failures with conventional numerical approaches. The cause(s) of excess pore-water pressure is not clear, but the unregularly disposal activities in addition to the ingress of rainwater and high pore water pressure played important roles in deformation of the dumpsite. Although the flowslide destruction process was fast with excessive deposits, it is postulated that signs of deformation may have already appeared in the study site but not discovered due to the absence of field monitoring. Further analysis is undergoing at SKLGP to assess the cause(s) involved for generating the excess pore water pressure.

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Reference


