Vulnerability of Buildings to Landslides: Impact Loads and Failure Mechanisms

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ABSTRACT

A large spoil slope in Shenzhen, China, failed on 20 December 2015. The landslide materials had a volume of approximately 2.6x10^6 m^3, travelled 1100 m and covered an area of 0.38x10^6 m^2. The landslide destroyed 33 buildings, leading to 73 deaths, 4 missing people, and 17 injuries. This was a rare landslide that occurred in a metropolitan setting and resulted in catastrophic consequences. Hence serious questions arose. What went wrong? Why did the debris travel so far and so fast? How were the buildings destroyed? What measures should be taken to present similar disasters in the future? This paper looks into the causes and mechanisms of the failure of the buildings by simulating the debris flow process, evaluating the impact pressures on the buildings, and exploring the failure mechanisms of the buildings. The history of the spoil soil slope is reviewed first. Then a digital elevation model of the slope, the terrain and the buildings prior to the slope failure is established using high-resolution satellite data. Subsequently, the flow process of the landslide material is simulated using a numerical program EDDA (Chen and Zhang 2015). Based on the simulations the impact pressures on the buildings are obtained. Finally, common failure mechanisms of buildings, i.e., column failure and foundation failure, are described, and the pertinent foundation failure mechanism for two buildings in the Shenzhen landslide event is analyzed in detail. The calculated flow depth and flow velocity at the distal end of the landslide are still far larger than the critical values for shear failure, so that the two buildings experienced large translational movements and collapsed.

INTRODUCTION

A large spoil soil slope in Shenzhen, China, failed on 20 December 2015. The landslide materials had a volume of approximately 2.6 x 10^6 m^3, travelled 1100 m and covered an area of 0.38 x 10^6 m^2. The landslide destroyed 33 buildings, leading to 73 deaths, 4 missing people, and 17 injuries. This was a rare landslide that occurred in a metropolitan setting and resulted in catastrophic consequences. Hence serious questions arose. What went wrong? Why did the debris travel so far so fast? How were the buildings destroyed? What measures should be taken to present similar disasters in the future?

Forensic geotechnical engineering involves scientific, legalistic investigations and deductions to detect the causes as well as the process of distress in a structural or geotechnical facility, which are attributed to geotechnical origin. This paper looks into the causes and mechanisms of the building failures in the Shenzhen landslide case by simulating the landslide debris flow process, evaluating the impact pressures on the buildings, and exploring the mechanisms of the failure of the buildings subject to impact. The causes of the failure of the spoil slope involved both technical and criminal investigations and are beyond the scope of this paper.

A number of studies have been performed to quantify the vulnerability of buildings to debris flow impacts in alpine areas (e.g., Lo 2000; Fuchs *et al*. 2007; Luna *et al.* 2011; Jakob *et al*. 2012). Lo (2000) reviewed empirical methods for estimating debris impact loads and design of natural terrain landslide debris-resisting barriers. Fuchs et al. (2007) conducted detailed site-specific analysis of the vulnerability of elements at risk to debris flows. Luna *et al.* (2011) generated three physical vulnerability curves that relate the intensity of debris flows to the economic losses. Jakob *et al*. (2012) reviewed 68 cases over the globe and classified four damage classes for building destruction. In the Shenzhen landslide case, the landslide and flow processes were well documented, and the failure mechanisms of several buildings were clearly exposed. The simulation of the landslide debris flow process and the impact pressures will further help reconstruct the failure process of the buildings.

FAILURE OF THE SPOIL SLOPE

The Hongao Spoil Soil Reception site is located at northwestern Shenzhen in Guangdong Province. The site was a quarry in late 1990's and was abandoned for some time. Figure 1(a) shows the quarry by the end of 2013, which was ponded with a reservoir volume of about 90,000 m^3. The quarry was late utilized as a spoil soil reception site, and was filled up rapidly in 2014 and 2015. Figure 1(b) shows the fill slope site in Sept. 2015, less than three months before the failure of the slope.

Figure 2 shows the geologic profile of the spoil soil slope. The bedrock consists of slightly to moderately decomposed granite on the back of the slope and slightly decomposed leptynite at the front of the slope. The elevation of the bedrock at the invert of the outlet of the quarry is about 60.6 m, where the elevation at the bottom of the quarry is about 44.3 m, leading to a maximum pond depth of 16.3 m. The pond was not drained before receiving soil fills hence the bottom of the spoil slope was fully saturated.

The spoil fill was stored in 9 stages and stages 7-9 were being filled and compacted before failure on the morning of 20 December 2015. The fill materials were primarily from basement and tunnel excavation, and may be classified as brown, fine to medium weak completely decomposed granite with some fill fragments. While the design slope crest elevation and the spoil soil volume were 95 m and 4.0×10^6 m^3, the actual highest elevation and soil volume reached 160 m and 5.83×10^6 m^3, respectively,

Figure. 1. The spoil slope site as of (a) 25 Nov. 2013 and (b) 29 Sept. 2015.

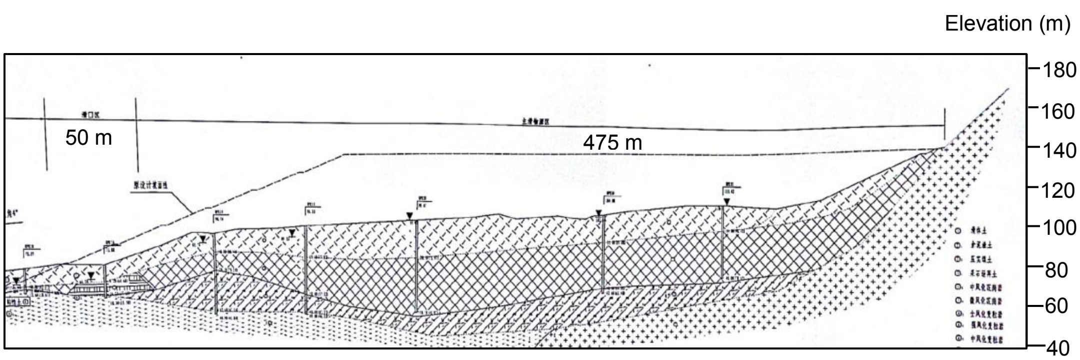

Figure. 2. Geologic profile of the spoil soil slope (after Shenzhen Geotechnical Investigation & Surveying Institute 2016).

before failure (State Council Shenzhen Landslide Investigation Panel (SCSLIP) (2016). According to SCSLIP, the failure of the spoil soil slope was caused by:

(1) No effective drainage. The quarry pond was not drained before filling spoil soils and no drainage blankets were designed inside the fill.

(2) Overloading. The fill height and volume were far larger than the design values so that the slope failed even at a relatively dry condition with only 1 mm of rainfall in the five days before the slope failure.

(3) Improper emergency management. The crest settled significantly and the slope bulged in the early morning of the failure day, showing signs of imminent failure. However the contractor failed to recognize such pre-failure signs. Rather the slope crest was filled up to compensate the settlement, accelerating the slope failure process.

ANALYSIS OF DEBRIS MOVEMENTS AND IMPACT PRESSURES

As shown in Figure 1(b), the proximity of the spoil soil slope endangers a large number of buildings downstream. In fact, the failure of the slope on 20 Dec. 2015 buried or destroyed 33 buildings as shown in Figure 3, including 24 industrial buildings, 3 residential buildings and 6 houses. To investigate the failure mechanisms of these buildings, it is essential to rebuild the landslide debris flow and building impact process. This paper reconstructs the process using numerical modelling.

A digital elevation model of the slope, the terrain and the buildings prior to the slope failure is established using high-resolution satellite data (Figure 4). Three-dimensional (3D) image analysis is also performed to rebuild the 3D images of some of the damaged buildings. Subsequently, the flow process of the landslide material is simulated using a quasi-three-dimensional depth-integrated numerical model, EDDA (Erosion–Deposition Debris flow Analysis) (Chen and Zhang 2015). EDDA simulates debris flow erosion, deposition and induced material property changes. In this model, the flow is driven by the potential energy of the landslide debris. The energy dissipation during the flow process is caused by frictional resistance, viscous energy loss and turbulence and particle interactions:

$$S_f = \frac{\tau_y}{\rho g h} + \frac{K\mu|v|}{8\rho g h^2} + \frac{{n_{td}}^2|v|^2}{h^{4/3}} \quad (1)$$

where K is a resistance parameter for laminar flows; h is the flow depth; v is the flow velocity; τ_y is the yield stress; μ is the dynamic viscosity; n_{td} is an equivalent Manning coefficient. The determination of τ_y, μ and n_{td} has been described by FLO-2D Software Inc. (2009). The model considers changes in debris flow density, yield stress and dynamic viscosity during the flow process, and is applicable to both dry flows and wet flows. As the soil was wet, particularly the soil at the slip surface was nearly fully

Figure. 3. Buildings that faced the landslide.

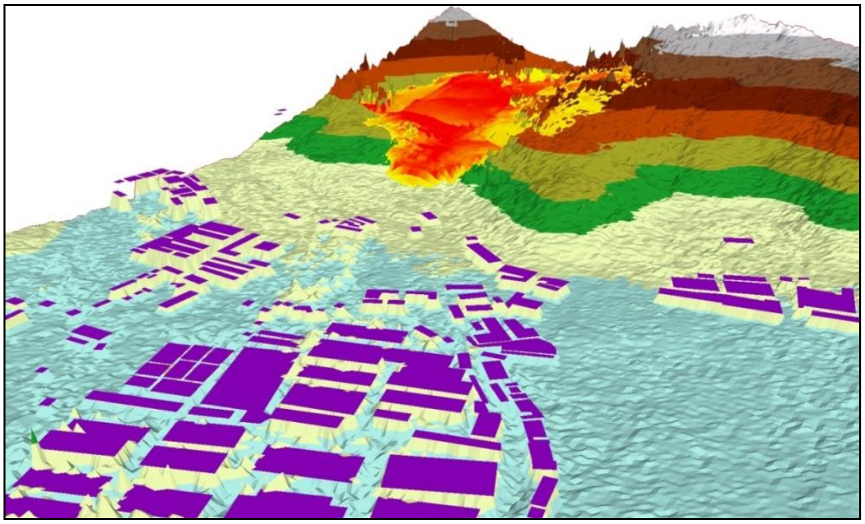

Figure. 4. Setting the flow model: the material above the slip surface is assumed fluid properties at time zero.

saturated, the flow was taken as a wet flow and typical values of the yield stress, dynamic viscosity and Manning's coefficient were assigned, as shown in Table 1. For comparison purposes, a dry flow case is also analyzed, assuming a yield stress based on a deposition angle of 12° as used for coal mining landslides by Hungr (1995). No viscous loss is present in the dry flow.

Table 1. Parameters for landslide debris flow analyses.

Flow type	Porosity	Density (kg/m^3)	Yield stress (kPa)	Dynamic viscosity (mPa.s)	Manning coefficient
Wet flow	0.37	2041	8.3	90*	0.06
Dry flow	0.37	1672	12**	0	0.06

* Water at 20 °C has a viscosity of 1.002 mPa·s.
** Based on Hungr (1995) for coal mine waste.

The soil volume above the slip surface was estimated as 2.4 x 10^6 m^3 based on post-failure field investigations. The model is set based on the filling geometry prior to the slope failure and the slip surface. At time zero, the material above the slip surface is set to flow. Figure 5 shows the impact areas and the maximum flow depths of the wet flow and the dry flow, as well as the actual landslide deposition area.

The debris flow impact force on buildings or any resistance structures may be composed into that from the impact of isolated boulders and that from the impact of the mixture. The impact force by a boulder can be expressed using the simplified Hertz equation:

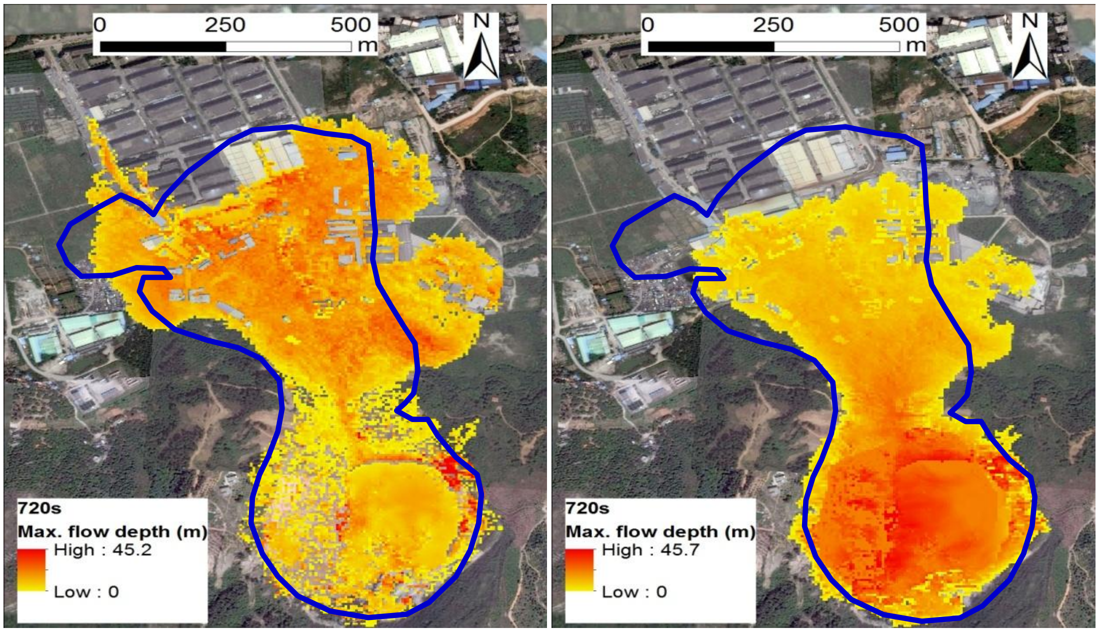

Figure. 5. Impact area and flow depth: (a) wet flow; (b) dry flow.

$$F_b = K_c\, 4000\, v^{1.2}\, r^2 \tag{2}$$

where F_b is the impact force (kN); r is the radius of the boulder (m); v is the impact velocity normal to the resistance structure (m/s); and K_c is a coefficient accounting for the rigidity of the boulder and the medium and other factors.

The hydrodynamic debris flow impact pressure (Pa) on an idealized two- dimensional rigid barrier is:

$$p_{\mathrm{h}} = \rho v^2 \tag{3}$$

where ρ is the density of the flow mixture (kg/m^3) and v is the flow velocity normal to the barrier (m/s). The impact pressure will be smaller if the barrier is flexible or if the barrier is not sufficiently large in the direction perpendicular to the impact direction (i.e., for a circular or square barrier). The total debris flow impact force on an obstacle, contributed from both the boulder impact pressure and the hydrodynamic pressure, is often expressed in a simplified form (Lo 2000; Kwan 2012):

$$F = \alpha\, \rho\, v^2 \sin \beta\, h\, w \tag{4}$$

where F is the impact force (kN); α is the dynamic pressure coefficient considering the geometry and rigidity of the obstacle, boulder impact and other effects; h is the debris thickness (m), w is the debris width (m); and β is the angle between the impact face on the barrier and the debris motion direction. A design α value of 2.5 is recommended by Kwan (2012). In the Shenzhen soil slope case, the spoil soil was primarily silty or gravelly sand, hence the boulder impact effects were not considered and a theoretical α value of 1.0 was used.

Figure 6 shows the distributions of the flow velocity and impact pressure when the front of the flow reaches the building at the northwest corner. The depth-averaged flow velocity is still very high at the distal end of the debris flow, and the maximum depth-averaged impact pressure reaches 900 kPa.

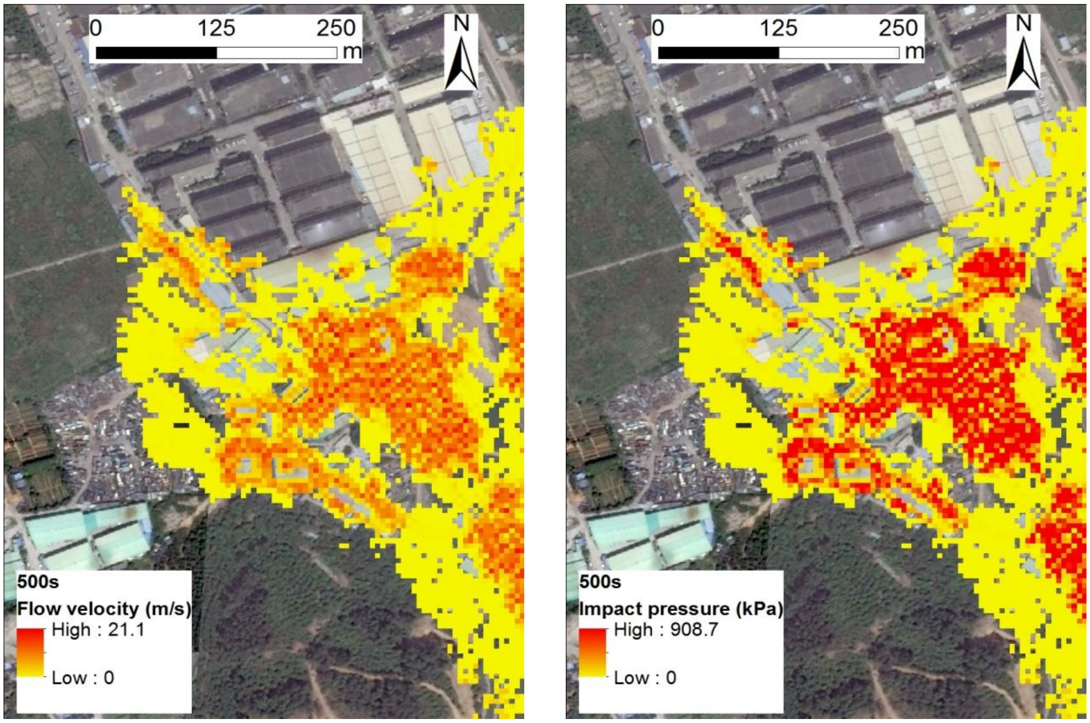

Figure. 6. Distributions of flow depth and impact pressure at northwest corner.

FAILURE OF THE BUILDINGS

The failure of a building due to debris flow impact can be divided into three damage modes:

(1) Burying damage. Burying refers to the debris flow destroying the ground floor external walls and breaking into the building interior. The building is basically intact but loses its intended function.

(2) Structural damage owing to the frontal impact of debris flow. This failure mode can result in the collapse of the entire structure if the main load-bearing components are seriously damaged and unable to resist the loads of the building. Moreover, damage can occur with the failure of only a few structural elements without the collapse of the whole structure.

(3) Damage to building foundations. The building foundations can be damaged due to soil erosion and bearing capacity failure in a sliding or bending mode.

Among the three failure modes, burying damage is the most common in small- scale debris flow events. Frontal impact often causes building collapses by structural damage or foundation damage in areas vulnerable to debris flows, and is thus a key element in engineering design.

In order to simplify the evaluation of the damage to reinforced concrete buildings under debris flow impact, a study of the critical velocity of debris flow and the diameter of large boulders is carried out considering two design scenarios:

(1) When a single column is damaged by a debris flow;

(2) Horizontal shear failure in the shallow foundation due to debris flow impact.

Column Collapse Mechanisms

The failure of a reinforced column can be described by the formation of plastic hinges. A design methodology of column failure analysis due to debris impact has been developed by Zeng *et al.* (2014). When the bending moment in the column caused by a debris flow reaches the ultimate moment of the column (M_u), plastic deformation of the column would be observed and plastic hinges would form at both ends of the column. If the column is a non-load-bearing element, another plastic hinge would also form in the mid-span of the column due to the continuous transfer of external bending moment. Therefore, the failure of the column can be classified into two types:

(1) Two-plastic-hinge collapse at the column ends (Figure 7(a)).
(2) Shear failure at location of direct impact (Figure 7(b)) or at the connections with horizontal structural component (Figure 7(c)).

The development of the plastic hinges is governed by the intensity of debris flow impact, the bearing capacity of the column and the impact condition between the debris flow and the column. The bearing capacity of the column is determined by its cross-sectional area, concrete and reinforcement strengths, the quantity of reinforcements, shear stirrups and the applied axial load in columns.

The study of Chan (2016) shows that the two-plastic hinge mechanism is more likely to occur than the shear failure mechanism. However, the column may fail easily by either mechanism when a boulder of 1 m diameter or larger impacts into the column. In fact, boulder impact is an important column failure mechanism.

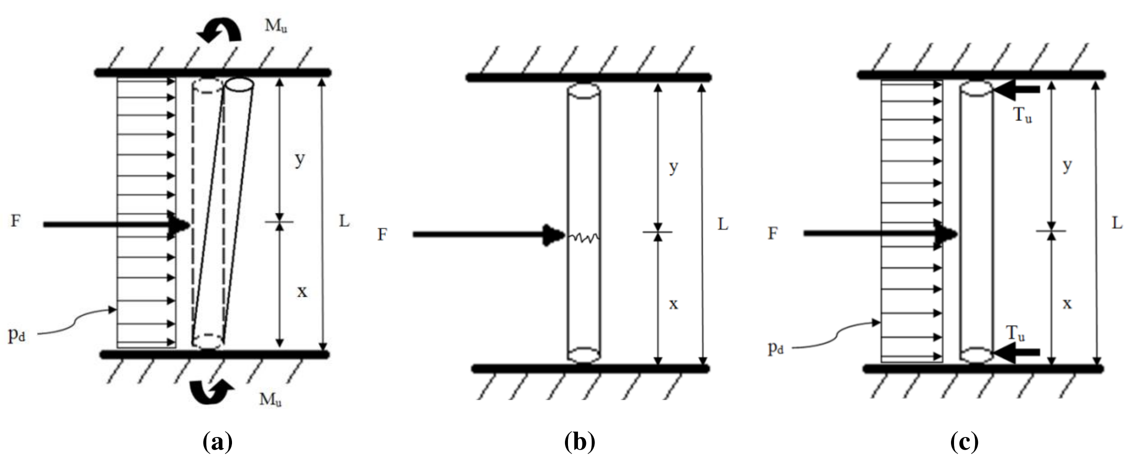

Figure. 7. Column collapse mechanisms under debris flow impact: (a) formation of two plastic hinges due to bending failure; (b) shear failure at impact location; (c) shear failure at end connections.

Foundation Shearing Failure Mechanism

Shallow foundations, where feasible, are generally more economical than deep foundations. Most shallow foundation failures are related to excessive movements rather than loss of load-carrying capacity. Under normal circumstances, the horizontal shear is small and the possibility of sliding is negligible. When stricken by a debris flow, the horizontal impact force brought by the debris flow may lead to sliding failure of the foundation. The friction between the base and the soil provides resistance to sliding, which depends on the axial load on the foundation, the self-weight of the foundation and the friction between the footing and the surrounding soil. The passive resistance of the soil in front of the foundation also plays an important role.

Consider a building on a level ground, supported by shallow foundations and subject to debris flow impact as shown in Figure 8. The impact load is resisted by the passive pressure on back side of the foundation (P_p), and the sliding resistance at the base of the foundation. At limiting equilibrium, the shear capacity, V_{fa}, is:

$$V_{fa} = (P + W_f + W_s)\,\mu + 0.5\,\lambda_a\,W\,D^2 \tag{5}$$

where P is the applied axial load from the column (kN); W_f is the total weight of the footing; W_s is the weight of the soil on the footing (kN), μ is the coefficient of friction between concrete and soil, λ_a is the net result of the active and passive pressures; W is the width of the footing and D is the depth of the footing. The coefficient of friction μ is given by

$$\mu = \tan\,(0.7\,\phi') \tag{6}$$

where ϕ' is the friction angle of soil. The value of μ is normally in the range of 0.3 to 0.7 depending on the soil characteristics (Coduto 2001). The net result of the active and passive pressures λ_a is given by

$$\lambda_a = \gamma\,[\tan^2\left(45° + \frac{\phi'}{2}\right) - \tan^2\left(45° - \frac{\phi'}{2}\right)] \tag{7}$$

where γ is the unit weight of soil. Suppose the building is impacted by n boulders of radius r and the debris mixture, and the boulders travel at the same velocity as the mixture. The critical velocity of the debris flow and the boulder size can be defined when the ultimate bearing resistance V_{fa} is reached:

$$400{,}000\,n\,v^{1.2}r^2 + \rho\,v^2\,W\,h = V_{fa} \tag{8}$$

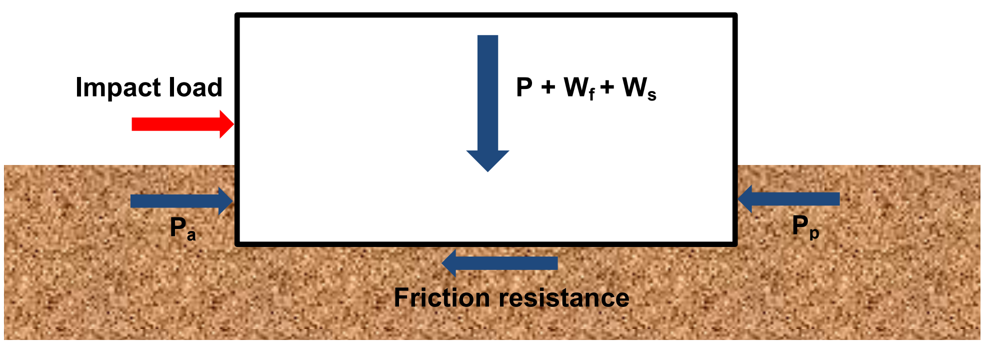

Figure. 8. Stability of building foundation under impact loading.

Foundation Shearing Failure Mechanism in the Shenzhen Spoil Landslide

In the Shenzhen spoil slope case, no large boulders were present as the spoil materials were sorted. Consider a building manhole of 3.0 m and an foundation embedment depth of 1.5 m. The density of the saturated spoil soil is taken as 2041 kg/m^3 and the unit weight of the foundation soil is taken as 20.0 kN/m^3. The friction angle of the completely decomposed granite soil is 30°-35°, corresponding to coefficients of friction between concrete and soil in the range of 0.384 to 0.455.

The vertical load on the foundation is the sum of the self-weight of the foundation and the superstructure load. The superstructure loading may be assumed to be 15 kPa per floor. The vertical pressures on the foundations of 3-, 5- and 10-storey buildings are shown in Table 2. Given the pressures and according to Equations (5) and (8), critical debris flow depth and velocity values are generated as shown in Figure 9.

Table 2. Footing dimensions and characteristics

Building storey	Vertical pressure (kPa)	Foundation width (m)	Foundation self-weight pressure (kPa)	Critical impact velocity at μ = 0.455 and h = 6 m (m/s)
3	45.0	3.0	30.0	3.87
5	75.0	3.0	30.0	4.29
10	150.0	3.0	30.0	5.19

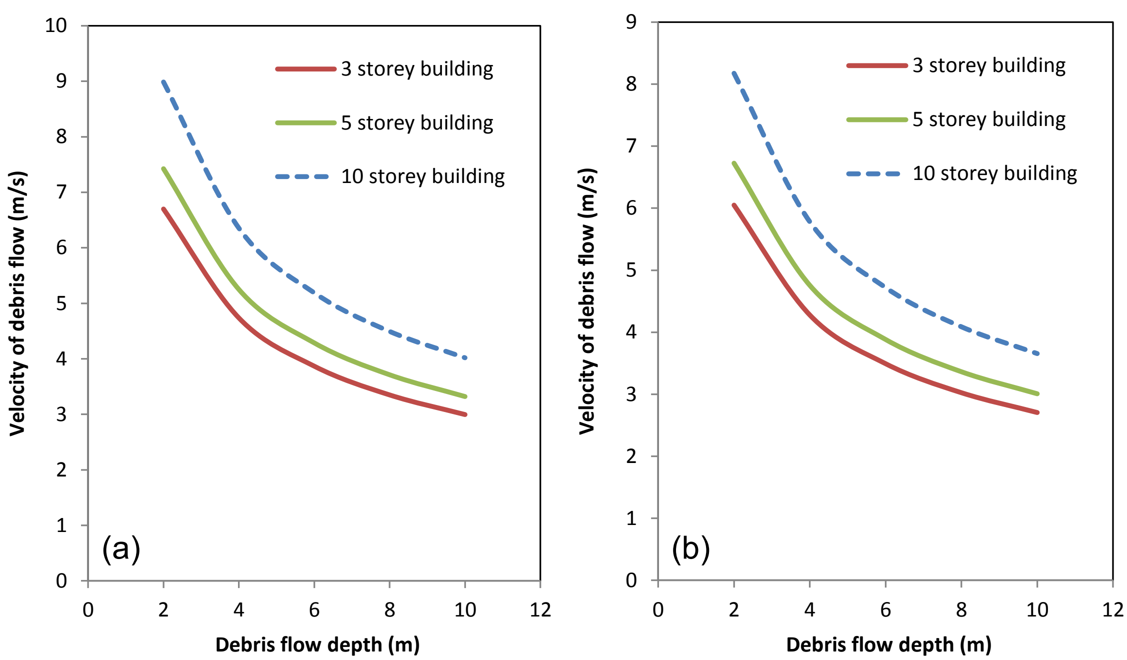

Figure. 9. Critical flow depth and flow velocity when no boulders are involved: (a) ϕ' =35° and μ = 0.455; (b) ϕ' =30° and μ = 0.384.

Figures 3 and 10 show the failure of several buildings at the front of the debris flow in the Shenzhen slope failure case. Two buildings (Buildings I and II in Figure 10(a)) exhibited a typical shear failure mechanism. Building I was sheared into two parts; the first part collapsed at place while the second part was sheared off its original location by about 50 m. The second building was sheared off site completely, and moved by more than 140 m.

The flow velocity in the frontal area is 5-7 m/s and the flow depth was about 6 m. The flow velocities are larger than those required to push over a 5 story building (e.g. 4.29 m/s). The flow velocities when the debris flow just arrived at the buildings in Area B in Figure 1(a) are estimated to be as large as 13 m/s and the flow depths are in the range of 7-13 m. These buildings might have been destroyed instantly.

FIG. 10. Front view of the building that was subject to sliding and overturning failure. (3D images with permission from Prof. Quan Long, https://www. altizure.com/project/567791fe9e0bd5b06ced19ba/model).

SUMMARY AND CONCLUSIONS

A large soil landslide in Shenzhen in Dec. 2015 caused 71 fatalities and destroyed 33 buildings. Detailed forensic investigations were conducted on geotechnical, operational, organizational and risk management aspects. This paper reproduces the landslide flow process and the distributions of debris impact velocity and pressure through numerical modelling, and evaluates the building failure mechanisms. The landslide debris travelled at a high speed of approximately 15 m/s when hitting the buildings, and slowed down as it swept over the building blocks. The flow velocity was still nearly 5 m/s at the distal end of the flow, sufficient to destroy the buildings.

Both the columns and the foundations of a building can fail under debris impact. A column can collapse in a two-plastic-hinge mechanism at the column ends or in a shear failure mechanism at the location of head-on impact or at the connections with horizontal structural component. The two-plastic hinge mechanism is more likely to occur than the shear failure mechanism. However, the column may fail easily by either mechanism when a boulder of 1 m diameter or larger impacts into the column. When a large debris flow impacts into a building, the impact load can be far larger than the horizontal resistance of the foundation; hence sliding failure can take place easily. The two buildings investigated in this paper were pushed off their original locations; one of them experienced a translational movement of approximately 140 m.

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