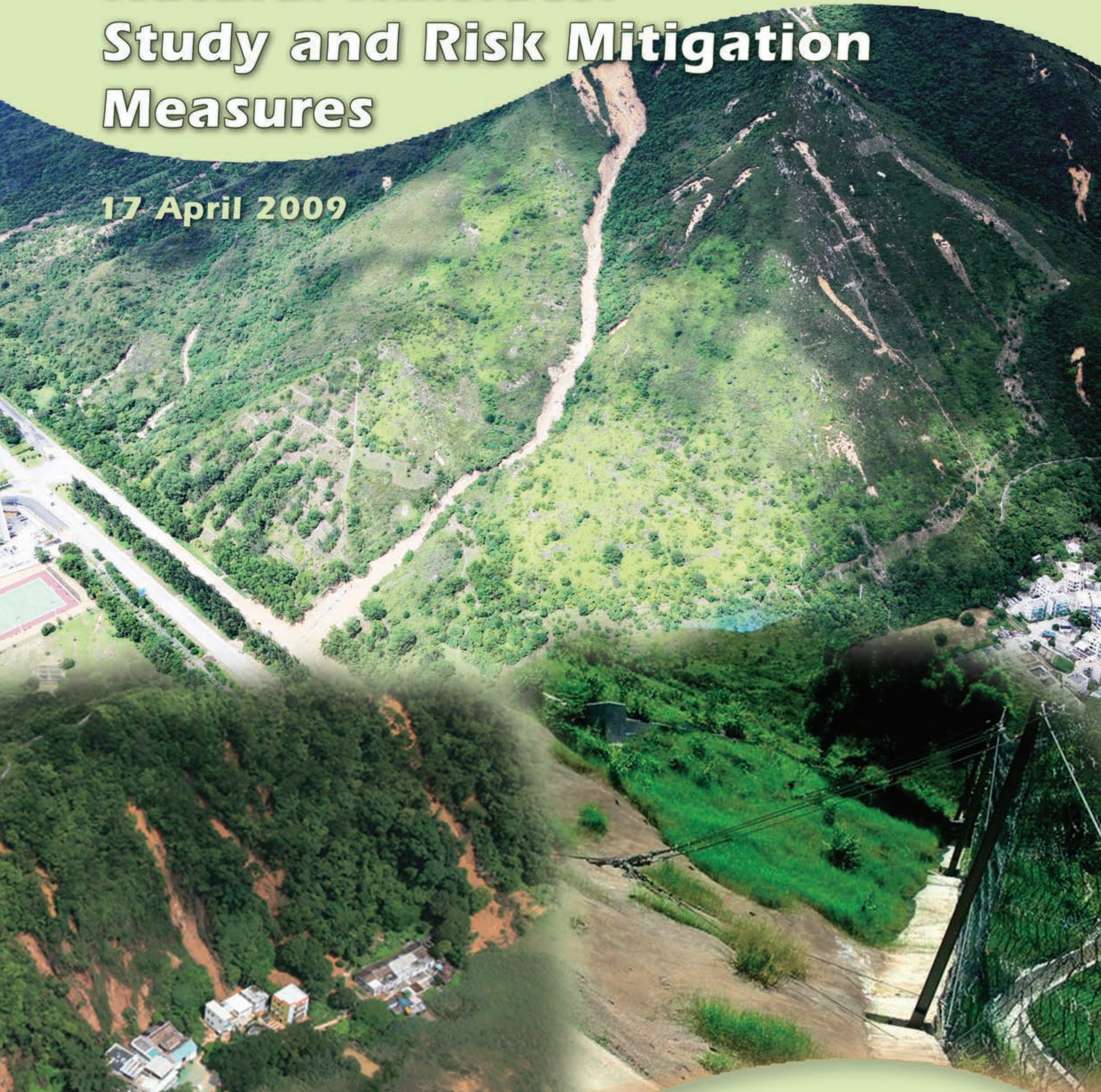
Natural Hillsides: Study and Risk Mitigation Measures

17 April 2009



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Geotechnical Division, The Hong Kong Institution of Engineers

Natural Hillsides: Study and Risk Mitigation Measures

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Captions of Figures on the Front Cover

Top figure:	Debris flow affecting a road with low traffic density (Courtesy of GEO/CEDD)
Bottom figures:	Multiple landslides on natural hillside in Tai O San Tsuen during the rainstorm on 7 June 2008 (Courtesy of GEO/CEDD) Flexible barrier installed as one of the mitigation measures in Tung Wan, Lantau (Courtesy of GEO/CEDD)

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Foreword

Last June, a number of landslides in natural terrain were induced by the heavy rain. Though we have gained much experience through over thirty years of harnessing our engineered slopes, we have realized that the nature and scale of the problems related to natural terrain are very much different from our engineered slopes and we have, unfortunately, much less experience on such hazards. It is on this background that the Organizing Committee has chosen the timely topic on "Natural Hillsides: Study and Risk Mitigation Measures" as the main theme of this Annual Seminar. We aim to provide a platform for geotechnical engineers and researchers to exchange their knowledge on hillside hazards and share, more importantly, their visions on the way forward. In addition to local engineers and researchers, we have also invited Professor Cui Peng of the Chinese Academy of Sciences to attend the Seminar and share his experience in hillside hazard mitigation in mainland. I believe that the Seminar can achieve its objectives and we will be more confident in tackling the challenges ahead of us.

I would like, on behalf of the Geotechnical Division, to take this opportunity to thank Professor J M Ko, our Guest of Honour; Professor Cui Peng and Ir H N Wong, our Keynote Lecturers; and authors of the papers. Without their supports, the seminar could not be realized. The contributions of the sponsors are also wholeheartedly acknowledged. My sincere thank is also extended to Ir W K Pun and his team for having the event organized seamlessly.

Ir Professor L G Tham
Chairman, Geotechnical Division (2008/09 Session)
The Hong Kong Institution of Engineers

April 2009

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Prevention and Mitigation System for Debris Flow Hazards in China

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ABSTRACT

Based on accumulation of scientific knowledge of debris flow control and prevention as well as engineering measures to protect buildings in China, an integrated system for debris flow mitigation and prevention as a optimal strategy is generally to be bring up to deal with debris flow hazards. China has suffered from different types of debris flow hazards in the mountainous areas in the past dozens of years. Active measures have been taken in China to struggle against debris flow hazards with a perspective of integrated control and most of them performed well. This paper introduces Chinese experience of debris flow control and produces a summary of the integrated system for debris flow mitigation and prevention, referring to following parts: debris flow control, debris flow protection, debris flow prevention, debris flow prediction and debris flow alarm.

1 INTRODUCTION

Based on accumulated scientific knowledge of debris flow control and prevention as well as engineering measures to protect buildings, an integrated system for debris flow mitigation and prevention as an optimal strategy is generally to be brought up to deal with debris flow hazards in China. Human beings have taken measures to resist debris flow hazards for a long time for their survival and development. However, these measures are not all successful. Many debris flow and landslide problems have been encountered during the construction of railway, highway and mines in China since 1950s. Based on some characteristics of debris flow, scientists from geography, geology, meteorology, hydrology, water conservancy, civil engineering, soil and water conservation carried out a series of mitigation and prevention measures aiming at debris-flow cradle and targets, such as towns, roads, bridges, mines, workshops and so on. Therefore, it could be said that countermeasure on debris flow mitigation and prevention is an integrated mitigation and prevention science.

The initiation, development and harmfulness of debris flow are closely related to its formation environment and factors. Debris flow mitigation and prevention is to adopt integrated management, partial management, preventive and forecasting measures according to debris flow formation factors and fathering needs to control the initiation and development of debris flow, and to alleviate or eliminate the damage and to protection objects. Therefore, the planed request of debris flow mitigation can be achieved.

The whole process of debris flow has its own characters and distinct regional difference. And debris flow control requires huge outlay and long period. So debris flow mitigation and prevention should comply with the following principles on the basis of experiences.

a) Overall plan and stress the key points. It is necessary to make an integrated overall plan for debris flow mitigation and prevention based on its formation conditions, activity characteristics and harmfulness. Therefore, the following points ought to be persisted in the control of debris flow hazards, referring to stressing the key problems, caring for the interest of the related departments and groups, making overall plans, balancing the benefits of long-term and short-term, preventing and mitigating debris flow hazards by controlling key inducing factors according to local environment. The control of debris flow hazards should be combined and cooperated with the local land improvement, natural resources exploitation, city and countryside construction, environment protection, agriculture development and so on. Moreover, the control and mitigation should avoid the situation that one kind of disaster transforms into another kind of disaster, and fully manifest the good benefit of local society, the economical and ecological environment.

b) The primary prevention combining with control ought to be carried out to get rid of the harmfulness in debris flow hazards mitigation.

c) Rational expenses and reliable technology ought to be adopted according to local condition and reality in the control of debris flow hazards.

The countermeasures of debris flow mitigation and prevention applied in China combine the characteristics of debris flows, the national construction need as well as the economic capacity. Therefore, the countermeasures suit China’s national conditions.

It is well known that there are two kinds of debris flows in China. One is the high-frequency debris flow. There were many high-frequency debris flows hazards in 1950-60s along with the constructions of railway and highway, and the development of mountainous economy. Through near 40 years’ control, integrated management and partial protective measures have been carried out in more than 100 debris flow valleys in China, and the obvious interests of disaster mitigation and economic development have been obtained. The Heishahe valley in Xichang, Sichuan and the Daqiaohe valley in Dongchuan, Yunnan are both the successful model of integrated management of debris flow applied in China. In the 1980s, several shocking debris flow disasters occurred in cities, enterprises, mines and important traffic routes in mountain area in China. However, these debris flow valleys are not high-frequency debris flow valleys where debris flow hazards occur frequently so that calls the people’s attention. On the opposite, they are actually low-frequency debris flow ditches which seem tranquil and no harm, but debris flow occurs when there is heavy rainstorm. The excellent human being society that is full of vigor suddenly turns into desert wasteland. As we can see, the loss caused by the low-frequency debris flow ditches is more serious than high-frequency debris flow valleys because the people are not vigilant and there is no any prevention measure. Therefore, the countermeasures for preventing this kind of debris flow have called the people’s attention and the primary control achievements have been acquired. This paper describes the present countermeasure system for debris flow hazards prevention and mitigation in China.

2 CONTROL SYSTEM

The control system is to apply integrated improvement in debris flow area and then control the occurrence and development of debris flow gradually. The system includes hillside improvement, gully improvement and accumulated area improvement (Wu et al. 1993).

2.1 Hillside improvement

The upstream of debris flow valley where water and soil loss are serious is the key of hillside improvement. The measure for hillside improvement includes: (a) Forestation measure: it can achieve the function of hillside stability through the mechanism of “tree conserves soil, soil conserves water, water protects tree”. (b) Surface runoff collection measure: generally, such project can be built in the formation area and clear water area of debris flow. It can make the rainstorm runoff collecting into the gully, and then induct it into the stable channel, thereby reducing the erosion in the formation area of debris flow. (c) Check dam engineering: it is mainly constructed in the gully in the debris flow formation area to block sediment and built into dam group (high 1-3 meter), which steady valley slope, conserve of soil and water (Figure 1). Certainly, engineering measures, including slope protection construction and landslide control engineering, also may be adopted, but this cost is too much and not feasible in China under the present economic condition.

Figure 1: Houshan mountain step-dam is located Heishui county, Sichuan province. These dams with 3-5 m of average high of every step could come into effects in common and protect each other when debris flow hazards occur

2.2 Gully improvement

This measure is mainly to build the different type of dams for blocking sediment in the floating section of debris flow to stabilize channel bed and banks. It has distinct effect on preventing shore landslide, at the same time it can stop and store part of silt and mitigate slope with high gradient. Furthermore, the special constructions for protecting the bank can be built in some important section in order to protect the banks from falling. These constructions include slope protection, soil protection engineering and so on (Tang et al. 2001). Step-check dam located in Hunshuigou valley of Yingjiang county is a typical case (Figure 2).

Figure 2: Hunshuigou valley step-dam is located in Yingjiang county, Yunnan province. It was constructed in 1998 and has deposited 6 million debris flow materials. It is 67 m (5 steps) and the highest debris flow storage sediment dam in the world by now.

2.3 Accumulated area improvement

The main preventing measure is to build prevention structures in the downstream area of debris flow to discharge, guide flow and stop silt according to people's plan. This measure can prevent harmfulness of debris flow to the downstream residential area, factory, mining company, traffic and so on. The structures include: (a) flood drainage slot, which is the important construction to control the path of debris flow, can prevent debris flow from overrunning in the accumulated fan. At present, almost all the prevention constructions of debris flow include flood drainage project (Figure 3). (b) Jetty for guiding debris flow, this structure is to guide debris flow to move along a certain path and protect the section which can be used and exploited. (c) Silt place, which is the structure in the accumulated area of debris flow, is to hold the mass of debris flow. This structure can lighten the pressure and load of debris flow to the downstream buildings and it is good for protecting the endangering targets.

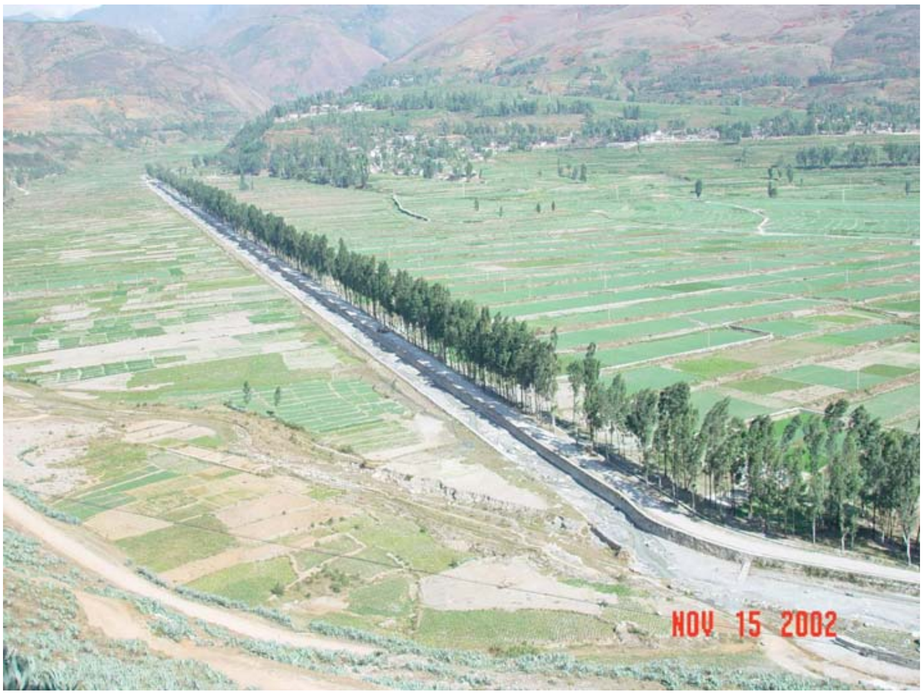

Figure 3: Daqiaohe drainage slot is located Dongchuan, Yunnan province, 6400 meter long and the longest drainage engineering for debris flow in China by now

The above three kinds of improvements can be fully applied at one debris flow gully to control the initiation and development of debris flow, then to achieve the goal of mitigate debris flow hazards. This is called integrated control. The control measure in some debris flow valley, which only controls the serious disaster, is called partial control.

2.4 Case study:Heishahe valley integrated management

Heishahe river watershed is a located in Xichang city, Sichuan province and a active debris flow valley, and began to be integrated managed since 1970. Before the control, there were 180 sites of landslide and landslip and 135 sub-valleys of debris flow in 22.7 km^2. In the past nearly 100 years, 5 villages became into wasteland and 200 ha farmland changed into sand land because of severe geological hazards. Debris flow hazard also endangered the security of Xichang section of Chengdu-Kunming railroad. After over 10 years control, in the upper of the valley, over 1670 ha had been rebuilt to plant trees, 7 shelter-belts for debris flow had been built, 210 ha terrace has been constructed, and a flood control reservoir with 22 meters dam height and 0.55 million m^3 reservoir content has been built. In the upper and middle of this valley, 7 storage sediment dams, 5 steps check dam and 7 downstream dams had founded in order to store sediment and reduce solid source of debris flow. In the lower of the valley, 3 km drainage slot and an overflow weir had been built and made flow channel stable. The integrated management not only kept road, railroad, factory and village in safe, but also improve agricultural yield and local ecological environment. Figure 4 is the design of integrated management of Heishahe valley.

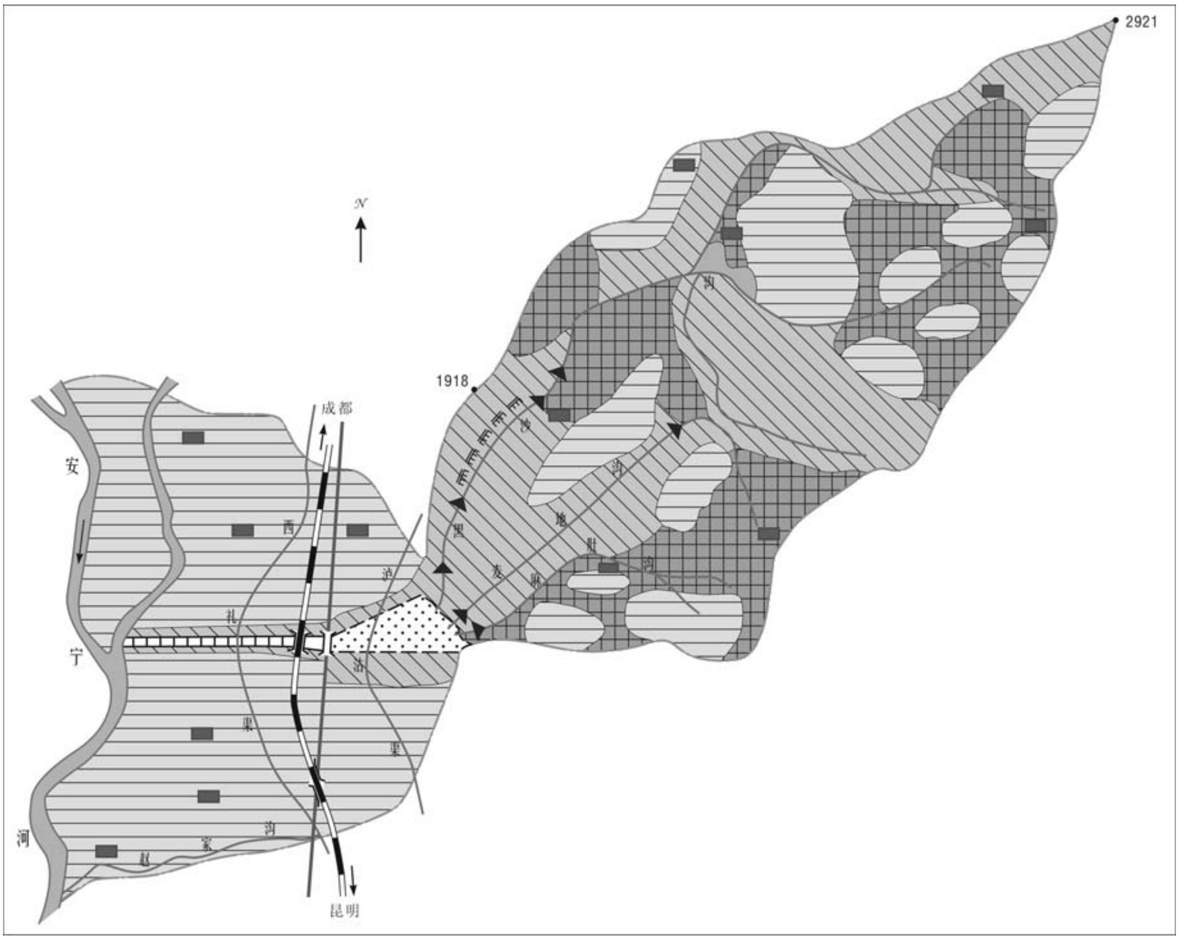

Figure 4: Integrated management of debris flow in Heshahe valley

3 PROTECTION SYSTEM

This system is to protect the endangering targets, such as cities and towns, factories and mining companies, railways, irrigation constructions and so on. The main measures are as follows:

3.1 Measures to protect cities and towns

Both controlling debris flow gully and protecting cities and towns which are threatened by debris flow are to avoid or lighten the harm caused by debris flow. However, the former is the fundamental and long-term measure and the latter is the temporary and short-term measure. However, the former costs lots of fund and its effects appear in the longer time; while the latter is contrary. The main projects include the protection bank, the flood path, the drainage slot, silt storehouse and so on. Babuligou valley drainage slot operates well to protect the city of Jinchuan county, Sichuan province (Figure 5).

Figure 5: Babuligou valley drainage slot (to protect the city of Jinchuan county, Sichuan province)

3.2 Measures to protect factories and mining companies

The construction of factories and mining companies in the debris flow area usually faces the following problems: (a) leave the path for debris flow when arranging buildings on debris flow fan, and build drainage engineering. Set up a special place to deposit debris flow silt if the situation permits in order to protect factories and mining companies better. (b) Arrange the buildings with close relationship in one side of debris flow fan, and try to avoid building bridges, culverts and pipelines of water and electricity across debris flow channel to prevent debris flow from blocking the traffic and pipeline.

3.3 Measures to protect transportation

The railways and highways in mountain areas are generally at the flat from the foot of mountain to the riverbank. So the rational route-selection engineering has significant effects for protecting the transportation of railways and highways.

(a) Rational route selection includes: Bypassing scheme: Through comparing with economic and technical factors, bypassing scheme is adopted at the sections where debris flow distributes densely, the scale of debris flow is rather big, debris flow frequently occurs and its harm is serious; Overpassing from the top of debris flow fan scheme: it is the best scheme because there is usually upper mesa at the top of debris flow fan, the channel bed is comparatively stable and the silt charge has no obvious change; Overpassing from the foot of debris flow fan scheme: the probability of silt charge at the foot of debris flow fan is smaller than that at the middle of debris flow fan so that this scheme can be adopted when the overpassing from the top of debris flow fan scheme is hard to realize.

(b) Overpassing the channel of debris flow through bridge scheme: Wherever railways or highways pass debris flow drainage, the bridge scheme is usually adopted. There are two familiar forms of bridge scheme: one is to pass by dispersive bridges according to the distribution of the natural channel bed of debris flow. This scheme is adopted where debris flow fan is wide and dispersive. The other is to pass by concentrative bridges. This scheme is adopted where debris flow fan is narrow and not dispersive (Figure 6).

Figure 6: Xiaohaihe Road and Railway Bridge is located in Dongchuan city, Yunnan province and keeps road and railway running in security because the turbulent debris flow and mountain torrent could pass through it in safety

(c) Overpassing debris flow valley by ferry scheme: Ferry is a kind of flood drainage built over rush. This scheme is usually adopted to drain debris flow when the route needs pass small hillside debris flow which has steep channel bed and the elevation of route is lower than the elevation of debris flow (Figure 7 and Figure 8).

Figure 7: The road or railway could pass through in the lower of eroded line of debris flow when the protection entity is lower than debris flow. Debris flow could reach the lower of debris flow valley through drainage slot. This is a debris flow drainage slot across Baoji-Chengdu railway in Feng county, Shaanxi province

Figure 8: The Drainage slot across debris flow valley in Wudu county, Gansu province

(d) Overpassing debris flow by cave scheme: Cave scheme can be adopted when the route passes the place where the scale of debris flow is big, the harm is serious and the changing range of sediment is big, especially passes the debris flow fan with a fast sediment rate.

Besides the above three principal protection system, there are also debris-flow protection projects which are built to protect irrigation engineering and important constructions. For instance, overflow pavement can be built in stable section with less erosion and deposition of debris flow (Figure 9).

Figure 9: Overflow pavement is a feasible scheme when a road must pass through debris flow field where the bridge and tunnel not be constructed for economic or technical reason. Although the overflow pavement is economical and easy to build, it should be located in stable section eroded and deposited by debris flow. The concrete overflow pavement in Wangjiaqing valley, Dongchuan, Yunnan Province, is located in the key area where the erosion and deposition of debris flow reached equilibrium and has resisted debris flow hazards for over 20 years

4 PREVENTION SYSTEM

This system includes the risk assessment before debris flow in order to prevent debris flow and the necessary engineering measure. The Hong Kong Special Administrative Region does this work prominently.

It is well known that the total land area of Hong Kong is 1075 km^2 with a multi-mountain hypsography. The highest elevation is 957 m (Damao peak), and the height of common mountains is around 500m. However, the population density of Hong Kong is 5000 persons per square kilometer, and is 30,000 persons per square kilometer in urban area, which is one of the areas with the highest population density in the world. The economy and life activity of inhabitants are mostly at the seashore which is beside mountain and near the sea or around the mountain. Almost all the engineering constructions there face the slope problem. The common disaster among them is mountainous soil torrent (minitype collapsing debris flow). Under the situation of slope disaster is serious and control task is very heavy, the hazards are controlled and the Hidden dangers are prevented in Hong Kong, especially the risk and consequence of mountainous soil torrent are evaluated on potential slopes. Positive and effective managing and controlling measures are adopted, and the high-level researchers and technology are gradually extracted, so the prevention measures of slope hazards (mainly mountainous soil torrent) in Hong Kong are scientific, rational, economical, safe and reliable. The loss of hazards, such as mountainous soil torrent, is reduced to the lowest. Therefore, Hong Kong becomes one of the areas where slope prevention is done excellently in the world. The main measures of preventing mountainous soil torrent in Hong Kong are as follow (Kang and Lee 2003):

4.1 Risk management of mountainous soil torrent

Risk management includes risk evaluation, whether the public accept these risk or not and measures adopted to reduce risk. Cost and benefit should be considered when selecting measure to decrease risk. The result of risk evaluation of mountainous soil torrent on one certain slope is used to measure the chance of the occurrence of mountainous soil torrent and its harm at one slope in a certain time. In fact, risk can be interpreted as the number of probability (such as one in ten thousand) of disaster multiplies the frequency of final serious consequence (such as death).

We need quantification risk evaluation to scale the effect of slope security system. Soil engineering department in Hong Kong began to study the application of quantification risk evaluation in 1993.

4.2 Estimate the risk with the quantification risk evaluation

Quantification risk evaluation is a method to systematically analyze each factor which causes and affects mountainous soil torrent and the degree of mountainous soil torrent hazard in order to measure risk order and the probability of each factor. Quantification risk evaluation should deal with the following issues:

a. What may cause harm? ⟶ Distinguish the risk of mountainous soil torrent
b. How often does it happen? ⟶ Frequency of mountainous soil torrent
c. Where is the problem? ⟶ The consequence of collapse
d. How serious is it? ⟶ The serious degree of collapse consequence
e. How to distinguish? ⟶ The receptivity of the risk of mountainous soil torrent
f. How to deal with? ⟶ The risk management of mountainous soil torrent

Based on the above analysis, the risk category is established. There are human life consequence and economical consequence after 1997. The two kinds are divided into three levels: high risk, low risk and micro risk. The result of risk evaluation put forward after comprehensive evaluation is the procedure that needs to be solved next.

4.3 Establish rational risk standard

Rational risk standard is essential for risk management and a criterion for distinguishing the calculated risk value.

Up to now, neither an international nor a national risk standard of debris flow hazard has been established for Hong Kong or the whole world to operate. In Hong Kong, mountainous soil torrent induced by man-made slope killed over 470 persons in the past 50 years (before 2000). Compared with man-made hazard, mountainous soil torrent induced by natural slope only caused 1 person in dead in the past 15 years. So the government of Hong Kong paid more attention to risk management on man-made slope. Nowadays the quantified risk assessment is used to assist the traditional analysis method for slope stability to work and contributes to the assessment result in Hong Kong. Moreover, it could resolve the problems that the traditional methods could not. However, practical validation is necessary when the quantified risk assessment was used extensively.

The government of Hong Kong makes regulation for risk management of natural slope as below: the prevention project for natural slope is expensive and difficult because natural slope is generally in the limiting state of slope stability. So the large engineering of cutting slope for slope stability is unfeasible. In this situation, any activity or protection engineering disturbing natural slope is required to stop, and buildings should be away from the area where mountainous soil torrent landslip and landslide may occur.

5 FORECASTING SYSTEM

Debris flow forecast is a methodology that forecast the initiation of debris flow hazards using the factors of debris flow formation, including rainfall, soil water content and so on. The spatial scale of the forecast is various and from a valley to local region, and the time scale varies from several months, several dozens days, several hours and several minutes. Moreover, debris flow prediction need to predict the time, magnitude, characters and impact area of debris flow by analyzing the basal factors of debris flow formation, including terrain, precipitation and solid mass (Cui and Liu 2000). Generally, debris flow occurs frequently and takes heavy loss in the valley where slope gradient is high, rainfall is in abundance and intensive and solid mass is in the gross and unstable, otherwise it is contrary.

At present, debris flow forecast predicts the initiation and magnitude using precipitation factor in nearly all countries and the spatial scale of the forecast focuses on local point (a valley) and the region (Wei and Xie 1997; Wilson 1997). In the past several decades, the debris flow forecast of China has focused on local site (single gully), and recently begin in the region scale (Tan and Wang 1994; Wang and Yu 2001). Meanwhile, the debris flow forecast of China concentrates on short-term and short-impending forecast (Tan and Duan 1995). Nowadays，how to improve space scale and time scale of debris flow forecast is necessary and urgent for researchers.

Besides rainfall, the relation between rainfall and soil water content in the upper of a valley is also used to forecast debris flow hazards. If soil water content in different gradient area of debris flow valley are monitored in real time, debris flow hazards could be predicted precisely by identified the time of landslide and landslip occurring in the upper of the valley. This methodology deserves attempting to forecast landslide and is especially suitable to forecast debris flow hazard induced by small landslide or landslip.

6 ALARM SYSTEM

The role of alarm system is to send out alarm message for the people ahead of several or several decade minutes before debris flow destroys the protected objects. Therefore, it can help the people avoid or reduce the damage of debris flow towards railway, factory, mines and towns.

There are many methods and instruments of debris flow alarm. Here several primary alarm methods are introduced as below:

6.1 Contact alarm

In China, the railway department ever installed probe in the shoreside of stable section to prevent the railway bridge from damaging by debris flow hazard. There are two methods of probe contact alarm, wire-snip method and on-state method. For wire-snip method, the probe sends out hazard alarm message to the people when debris flow slip the thread .For on-state method, the probe sends out hazard alarm message to the people through turning circuit on when debris flow touch it. The Chengdu Railroad Bureau installed probe alarm for debris flow hazards in 8 valleys of debris flow along Chengdu-Kunming railroad in 1985, but probe alarm only predicted one debris flow hazard successfully (Ye 1991).

6.2 Non-contact alarm

Non-contact alarm is a methodology that gets the hazard message by the sensor of instrument and sends it out to the people. Compared with the contact alarm, the instrument of non-contact alarm could be used longer time due to avoiding the destruction of debris flow. Nevertheless, it is very important to improve the accuracy and precision of the instrument since it does not contact debris flow mass.

For non-contact alarm methodology, the alarm instruments used widely are manufactured according to geological sound theory or ultrasonic theory. Geological sound alarm instruments have different responses to different debris flow valley due to the different lithology and hazard characters of different debris flow valley. For example, during the joint investigation between China and Japan, the auto-camera observation system presented by Japan did not work efficiently in Jiangjiagou gully because the probe did not induce the signal of debris flow and then drive the camera to photograph. So geological sound alarm instrument should be adjusted according to local the characters of a valley and then be used.

The principle of ultrasonic alarm instrument is that: the probe emits ultrasonic, which spreads with a certain velocity in homogeneous medium. When ultrasonic encounters different medium, it will reflect from the interface of different medium. Therefore, using the propagation velocity and time of ultrasonic, the distance (s) from the probe to the interface can be computed by the following equation:

$$s = \frac{1}{2}ut \qquad (1)$$

where u= the propagation velocity of ultrasonic, t = the propagation time of ultrasonic.

Ultrasonic mud position alarm is made and in operation according to this principle. It has been extensively used to alarm debris flow hazards and observe mud position in China, Japan, USA, and Europe where suffered heavy debris flow hazards due to its high accuracy and stability. For example, on Jul 25, 1985, the good alarm developed by ultrasonic principle was applied for the observation of debris flow in Jiangjiagou gully, Yunnan province, China (Kang 1990).

The observation results of mud position alarm showed as follow (Figure 1): A debris flow with total 75 surges was observed from 20:16:38 to 22:47:31.It could be divided into three parts: early stage, mid-stage and late stage. Alarms at early stage account for 53% of the total surges, mid-stage account for 20% and late stage account for null due to mud position below 2m (mini-observation mud position). In figure 1, the relation between alarm process and alarm duration showed that alarm of 1st surge lasted for about 2 seconds, 2nd surge for about 1 second and 46th surge for null. As a result, alarm duration can reflect the process of over-mud position and debris flow of magnitude (Figure 10).

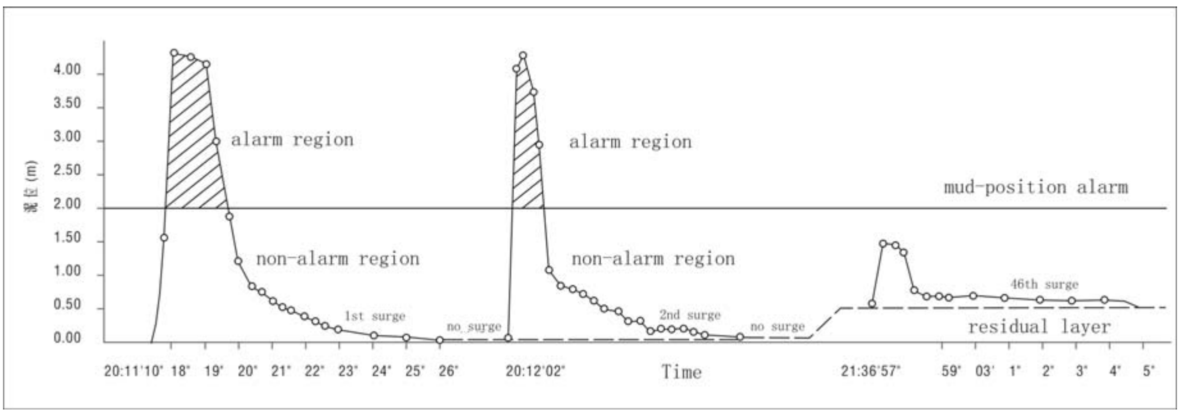

Figure 10: Comparison of measured value of debris flow's mud position between alarm region and non-alarm region in Jiangjiagou gully

Ultrasonic alarm is feasible to pre-warn the initiation and magnitude of debris flow by observing mud position. It is very convenient and high accuracy to alarm debris flow hazard, however, the following points should be paid attention:

a) The probe of mud poison alarm must be installed in stable river reach, because alarm is achieved through monitoring mud position of debris flow across flow section. If the probe is hanged over the section with fluctuant change in deposition and erosion, the measured value will be invalid and even a serious mistake.
b) If the alarm is only used to monitor whether debris flow occurs, the probe need to be installed in main gully and sub-gully of the upper reach. Moreover, if the alarm is also used to monitor the magnitude of debris flow, the probe need to be installed in main gully of the upper or middle reach, where the discharge of debris flow keep relatively stable in order to reduce alarm error.
c) During 1985-1986, this instrument alarmed 14 debris flow hazards including debris flow, un-coherent debris flow and stony flood. Therefore, it can be used to measure and alarm mountain torrent.

6.3 Multi-function alarm's choice and application

The above introduced alarming methods have their unique feature. In practice, the felicitous method should be chosen to use according to the characteristics and significance of the protection objects. For towns, factories, mines, etc., the method providing more time for the people to mitigate debris flow should be selected to use. For road, railway, etc., ultrasonic mud position alarm, geological sound and contact alarm could provide alarm ahead enough time for traffic department to avoid debris flow(Aleotti,2004) For some important towns, factories, mines, road and railway directly endangered by debris flow, several methods could be integrated to prevent debris flow hazards. Based on the united methods, the mitigation scheme of "short-term forecast (preparation)"-"impending forecast (ready to withdraw)"-"alarm (withdraw)"-"cancel alarm" should be made.

7 OTHER PREVENTION MEASURES (SYSTEMS)

The above control systems are propitious to short-period and high-frequency debris flow hazards but not to debris flow hazards that occur alternately several decades and even over 100 years because these instruments is not probable to be used to monitor these debris flow in such long time. However, this kind of debris flow hazard generally takes heavy losses to local people's life and property when it occurs. Therefore, besides debris flow alarm, other prevention measures for debris flow hazards are necessary to configured in cities, towns, villages and key traffic engineering that suffered by debris flow hazards.

a) All factories located in the mouth of small watershed and the lower of still slope should conduct field investigation about mountain torrent, debris flow and landslide in order to confirm whether geological hazards will occur in this valley. Then the prevention scheme is made to mitigate geological hazards, especially debris flow hazards.
b) In the valley debris flow hazards developing in low frequency, the measures of ecology control and restore should be performed effectively in order to reduce solid mass of debris flow by decreasing soil erosion and the accumulation of sediment, especially coarse grains soil. Moreover, the valley channel should be unblocked and stable, and be forbidden to narrow and dig in order to prevent flood from scouring, initiating and transporting channel bed and initiating debris flow.
c) For the valleys confirmed developing low-frequency debris flow hazard by investigation, the history of debris flow events should be documented and publicized for more people through rational manners in order to mitigate debris flow hazards effectively. For example, Guanmiaogou valley, located in Nanping county, Sichuan province, develops very low-frequency debris flow. During Ming dynasty to Qing dynasty, a severe debris flow hazard even occurred in this valley, destroyed the towns in the lower of this valley and resulted in the county city relocating. However, the people forgot the painful history of debris flow hazard, and built many structures in the dangerous area of this valley without adopting any prevention measures after 1950's. When debris flow occurred on July 18, 984, the history replayed and the people suffered heavy loss again.
d) For the low-frequency debris flow valley with the people and important public engineering distributed in the lower, it is recommended through debris flow hazard investigation that 1-2 grid barrages with 8-meter high should building the main valley to block solid mass and decrease the impact to the lower when debris flow occurs. For example, debris flow in Heishagou valley, Yinmin mine, destroyed 11 family houses by

tremendous dynamic and then rushed at the accommodation region of several hundred workers. However, debris flow was blocked by a four floor building located in the mouth of the valley and stopped after destroyed the building. As a result, all workers survived for the irrational building. Why not is the grid barrages built in the upper to protect lives and estate of the people in the lower? This is not only a lesson but also an experience.

e) As a strategic prevention measure, the dangerous area (serious, slight) of debris flow should be zoned after the impact extent and affective extent of debris flow are identified according to the investigation data.

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Rising to the Challenges of Natural Terrain Landslides

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ABSTRACT

Systematic study and mitigation of natural terrain landslide risk is a core component of Government's post-2010 Landslip Prevention and Mitigation Programme (LPMitP). Tackling natural terrain landslide hazards marks a new chapter in Hong Kong's landslide risk management, and the geotechnical profession must rise to the challenges of its enhanced responsibilities. This paper summarises the technological development work on natural terrain in the past twenty years, which has led to the formulation of the LPMitP. Technical issues that may confront the profession as we venture into this new field of work are described, with particular focus on natural terrain failures, debris movement and risk management strategies. Lessons learnt from ongoing studies, including those from the June 2008 landslides, and their implications are highlighted.

1 NEW CHAPTER OF LANDSLIDE RISK MANAGEMENT IN HONG KONG

1.1 History of Landscape Evolution in Hong Kong

Hong Kong has a population of over 7 million and a land area of about 1,100 km^2. The terrain is hilly, with some 75% of the land steeper than 15º and over 30% steeper than 30º. Intense urban development has taken place on flat ground and in foothill areas, and is progressively encroaching on the steep hillsides where landslides from man-made slopes and natural terrain could pose a significant hazard (Figure 1).

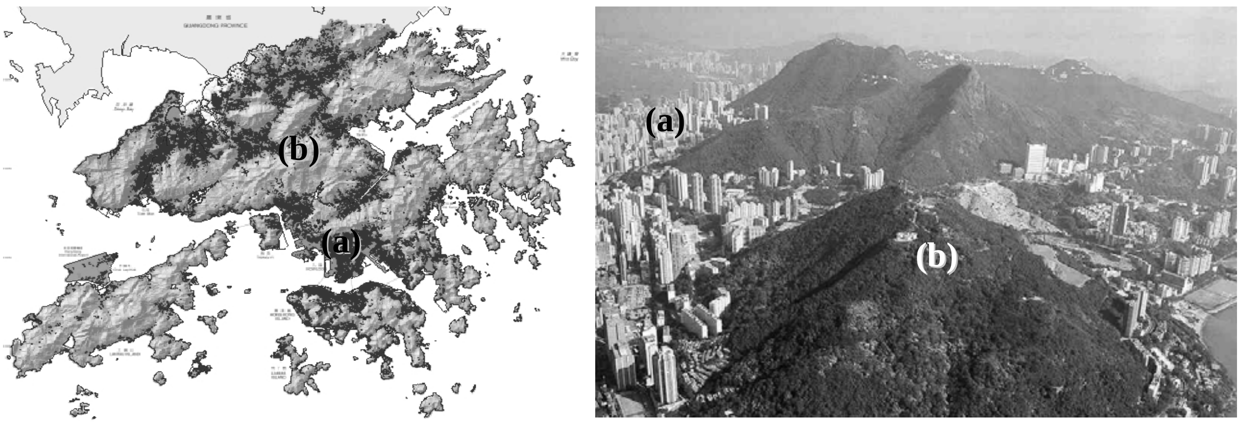


Figure 1: High concentration of developments in Hong Kong mingled with man-made slopes and natural hillsides (Note: (a) urban development fringing (b) steep natural hillsides)

Natural terrain occupies about 60% of the land in Hong Kong. Much of it is steeply sloping and mantled by weak saprolitic or residual soils, or colluvial deposits derived from past landslide and erosion processes. The vast majority of the terrain is underlain by volcanic and granitic rocks, which were formed over 100 million years ago (Sewell et al. 2000). The details of the present day landscape are thought to have formed mainly during the Quaternary Period, from about 1.4 million years ago to the present. The subtropical deep weathering of the rocks probably began during the Tertiary period as long as 60 million years ago under relatively stable, warm and humid climatic conditions (Ruxton & Berry 1957). The last glaciation commenced about 25,000 years ago, reaching a maximum about 17,000 years ago. At this time, sea level was around 130 m below the current level, and the coastline was about 120 km south of Hong Kong (Figure 2). When global deglaciation accelerated about 12,000 years ago, the sea level rose and the coastline moved northward. A postglacial sea level high, possibly reaching 1 to 3 m above its present level, was attained about 7,000 years ago (Fyfe et al. 2000). Substantial erosion and mass movements probably occurred at the new coastal region. These involved coastal landslides and reactivation of relict failures, which modified the landscape. Erosion and mass movements would have also taken place on mountain slopes, particularly during wetter periods, which are postulated at about 12,500 to 11,000, 10,500 to 8,000, and 7,000 to 5,500 years ago (Ng et al. 2003).

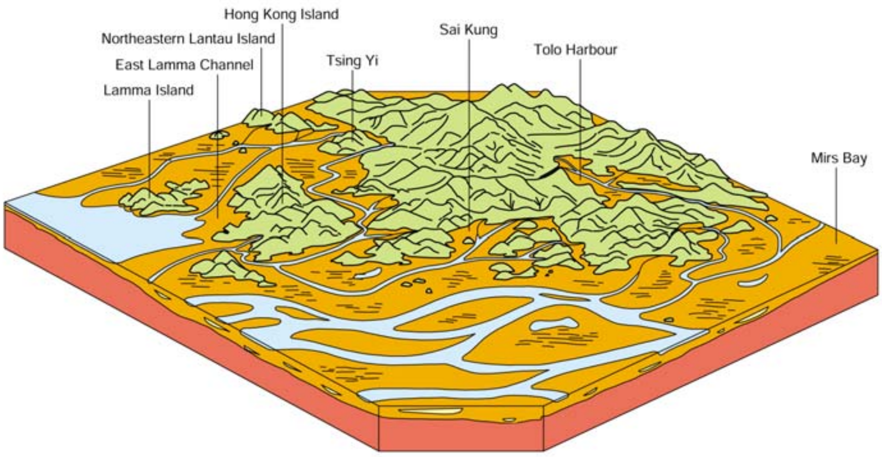


Figure 2: Topography of Hong Kong during a low sea level stand (from Fyfe et al. 2000)

While Hong Kong's natural hillsides have experienced a long history of landscape evolution, they remain highly susceptible to rain-induced landslides (Figure 3), which could develop into debris flows upon entering drainage lines. Sizeable failures on steep open hillslopes could also result in discharge of fast-moving debris downslope. Should the debris reach developed areas, serious consequences may occur.

Figure 3: Landslide-prone natural terrain in Hong Kong

1.2 Natural Terrain-related Studies in Early Years

Lumb (1975) summarized in his paper the state of knowledge on landslides in Hong Kong at the time, with attention given mainly to man-made slope failures.

In 1977, the Geotechnical Control Office (GCO, renamed Geotechnical Engineering Office in 1991) was set up as the central body to regulate geotechnical engineering and slope safety in Hong Kong. Since then, Hong Kong landslip preventive works have primarily focused on landslide risks associated with man-made slopes, which posed a much greater overall risk to the community than natural terrain hazards. Knowledge of natural terrain landslides was rather limited in Hong Kong in the 1970s.

Some pioneering studies on Hong Kong's natural terrain were pursued in the 1980s. These included the territory-wide geological survey mapping (Fletcher 1997), which succeeded the earlier work by Allen & Stephens (1971), and the terrain classification mapping published under GCO's Geotechnical Area Studies Programme (Styles & Hansen 1989). The above provided useful information, at a regional scale, on the geology of Hong Kong and the general terrain characteristics. Apart from these major mapping programmes, the Mid-levels Study (GCO 1982) was a landmark in regional slope studies, comprising three years of extensive ground investigation, hydrogeological monitoring and subsequent analysis. Some technical publications in the 1980s also covered aspects of the hillside processes, e.g. recognition of soil piping in colluvium and saprolites as a widespread near-surface process (Brand et al. 1986), and engineering geomorphology of Hong Kong's terrain (Hanson 1984; So 1986). Information on recorded natural terrain landslides was summarised in GCO's reports on rainfall and landslides. These include reports on two major rainstorms of 1982 (Hudson 1982; Tang 1982), in which many natural terrain landslides were observed. Engineering methods for treatment of unstable boulders (Au & Chan 1991) and design of boulder barriers were developed (Chan et al. 1986).

1.3 Technology Development Since the 1990s

With increased awareness of the potential hazards from natural terrain failures, the Geotechnical Engineering Office (GEO) (now, part of the Civil Engineering and Development Department), in collaboration with geotechnical practitioners and researchers, has been undertaking technical development work on the subject since the early 1990s. This has led to improved understanding of the nature and characteristics of natural terrain landslides, their potential risk and approaches for risk management. Some initiatives that are either milestones in the technological development or have notable impact to professional practice are described below:

(a) **Study of the 1990 Tsing Shan debris flow**: This large channelised debris flow (Figure 4) was studied in detail by the GEO, and reported by Chan et al. (1991) and King (1996). The incident started as a debris slide of about 350 m^3 at the landslide source. It developed into a 20,000 m^3 debris flow through material entrainment, with a runout distance of about 1 km. The debris flow has been regarded locally as an example of a low-frequency, large-magnitude natural terrain landslide event, and quoted internationally as a case that illustrates high debris flow entrainment (Jakob & Hungr 2005). Before the debris flow, the planned development in the region was encroaching on the Tsing Shan foothills. The debris flow could have resulted in serious consequences if the site traversed by the debris flow had been developed at the time. After the debris flow, development at the site was changed to a golf driving range to minimise risk exposure. The case is a vivid illustration of the risk of debris flows and the importance of proper land-use and development planning in controlling undue increase in the risk of natural terrain landslides.

(b) **Study of the 1993 Lantau landslides**: In the rainstorm of November 1993, over 860 natural terrain landslides occurred on Lantau Island (Figure 5). The landslides resulted in blockage of roads and catchwaters. As Lantau Island was largely undeveloped at the time, there were no landslide casualties. The landslides were mapped and the data collected analysed (Wong et al. 1998). The study established that a high density of natural terrain landslides could be triggered in a severe rainstorm (about 7 nos./km^2 in the region affected by high rainfall intensity in this rainstorm), and that terrain at a gradient of 30° to 35° underlain by volcanic rocks was most susceptible to failure. Empirical assessment of landslide debris runout data showed that debris mobility was affected by the mechanism of debris movement, with channelized debris flows being more mobile than open hillslope failures. The study carried out by Franks (1998) on the landslides overlooking Tung Chung New Town, which was being developed in 1993, also arrived at similar observations.

Figure 4: The 1990 Tsing Shan debris flow

Figure 5: Natural terrain landslides on Lantau Island in the November 1993 rainstorm

(c) **Natural Terrain Landslide Inventory (NTLI)**: In the mid 1990s, the GEO compiled the Natural Terrain Landslide Inventory (NTLI), a Geographic Information System (GIS)-based inventory of historical natural terrain landslides identified from interpretation of high-flight aerial photographs (2,400 m or above) taken since 1943 (King 1999). Figure 6 shows a graphical display of NTLI, including the identified natural terrain landslide crowns and the debris trails. Up to the year 2000, the NTLI has catalogued some 30,000 landslides, about 11,000 and 19,000 of which are recent and relict landslides respectively. Factors such as photograph coverage, cloud cover, ground shadows, vegetation cover, and scale and resolution of the high-flight aerial photographs impose certain limitations on the dataset. Consequently, some landslides may have been missed, and some landslides shown in the inventory may be other features mis-identified as landslides. Despite these constraints, the NTLI was one of the most comprehensive catalogues of historical natural terrain landslides that had ever been compiled at the time. It provided important data for studies of natural terrain hazards, and was widely used by the geotechnical profession until it was replaced by the enhanced inventory described in Item (o) below.

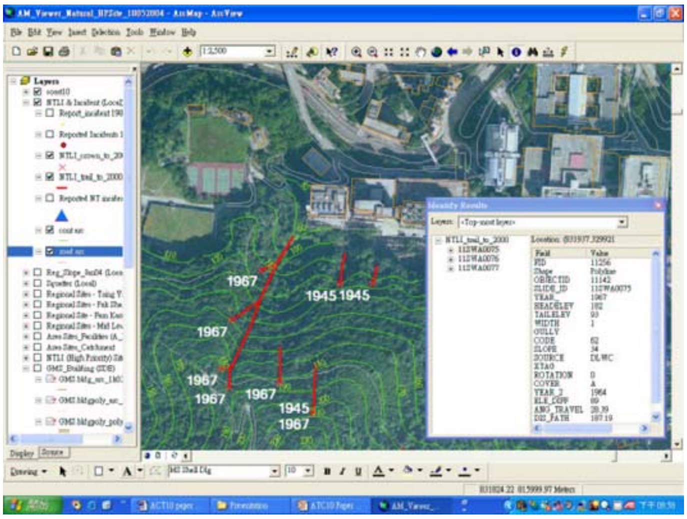
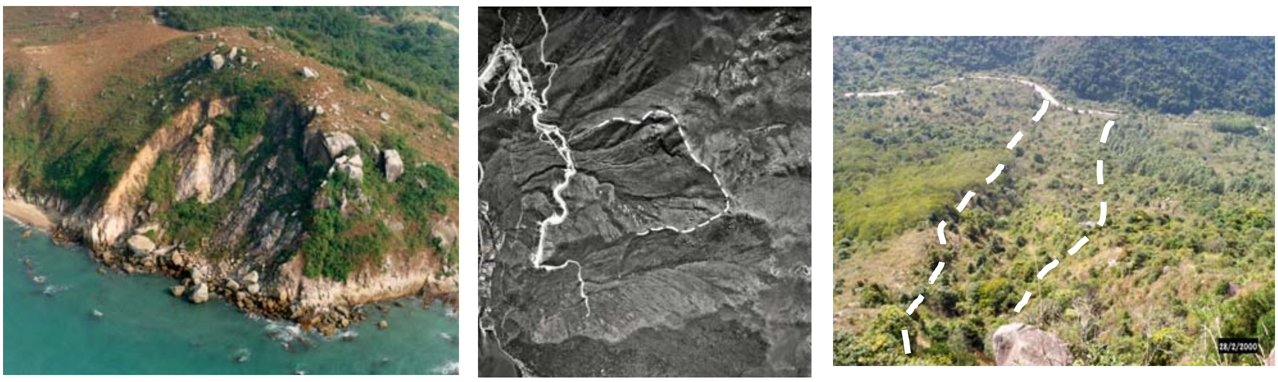


Figure 6: Natural Terrain Landslide Inventory in GIS Format

(d) **Large Landslide Dataset**: A database of about 1,900 nos. large natural terrain landslides was compiled based on interpretation of aerial photographs and hillside geomorphology (Scott Wilson 1999a, b). These large landslides include large relict morphological features and recent natural terrain failures with a scar width exceeding 20 m. Some examples of the identified large relict landslides are shown in Figure 7. Studies were carried out on the possible scale and age of some of the large relict landslides. For instance, the large coastal landslide on Lamma Island had an estimated volume of about 30,000 m^3 and probably occurred within the last few hundred years. The massive debris lobe at Sham Wat, Lantau covers a plan area of about 0.3 km^2. Age-dating revealed that the main body of the hillside probably failed some 30,000 years ago, but further sizeable detachments continued to take place and the youngest one was dated as only about 2,000 years old (Sewell & Campbell 2005). The large relict landslide scar that was left in place after a massive debris flow near the present Tung Chung Road was found to occur about 8,000 years ago. These landslides are relatively young in geological time scale. They could have implications to the assessment of the current landslide risk.



(a) Large coastal landslide on Lamma Island | (b) Massive debris lobe at Sham Wat, Lantau | (c) Large relict landslide above Tung Chung Road

Figure 7: Examples of large relict landslide features

(e) **Landslide susceptibility analysis**: Evans & King (1998) carried out a territory-wide landslide susceptibility zoning, based on correlation of natural terrain landslides with slope angle and geology. Nineteen geological groups and thirteen slope angle classes were adopted, which resulted in some 247 different types of terrain units. Natural terrain landslide data from the NTLI up to the year 1994 were used in the analysis. The Digital Elevation Model was compiled from the 1:5,000-scale 10 m contour topographical maps, but the susceptibility zoning map was prepared in 1:20,000 scale. More refined susceptibility analyses have subsequently been carried out at selected regions, using additional terrain attributes and more elaborate analytical techniques. Wong (2003) reviewed the state of practice in susceptibility analysis, and cautioned against the limited resolution and reliability achieved in susceptibility zoning, which affect its applicability to risk management.

(f) **Systematic landslide investigations**: Since 1997, significant natural terrain landslides were systematically investigated under GEO's landslide investigation programme. Notable cases investigated include: the 1997 Shatin Heights landslides (FMSW 2001), 1999 Sham Tseng San Tsuen debris flow (FMSW 2000), the distressed hillside at Queen's Hill (FSW 1999), the 2000 Tsing Shan debris flows (King 2001), the 2000 Leung King Estate debris flows (HCL 2001), the 2001 Lei Pui Street debris flow (MGSL 2004), landslides at Cloudy Hill (HCL 2003), the slow-moving landslide at Tsing Shan foothills (Parry & Campbell 2003), and the 2005 Kwun Yam Shan landslide (MGSL 2007). The investigations have brought about further insights into the causes, mechanisms and characteristics of natural terrain landslides.

(g) **Interim risk guidelines**: A set of landslide risk guidelines, benchmarked against that adopted for Potentially Hazardous Installations in Hong Kong, was formulated by the GEO (ERM 1998). The risk guidelines stipulate the tolerable risk criteria for natural terrain landslides in respect of Individual Risk and Societal Risk. The Individual Risk criteria apply to the annual probability of fatality for the most vulnerable person affected by landslide hazards. The maximum allowable limit is 10^{-5} for a new development, and 10^{-4} for an existing development. The Societal Risk criteria apply to the total risk-to-life posed on the community by landslide hazards within the consultation zone. The criteria were expressed as a frequency (F) versus number of fatalities (N) distribution.

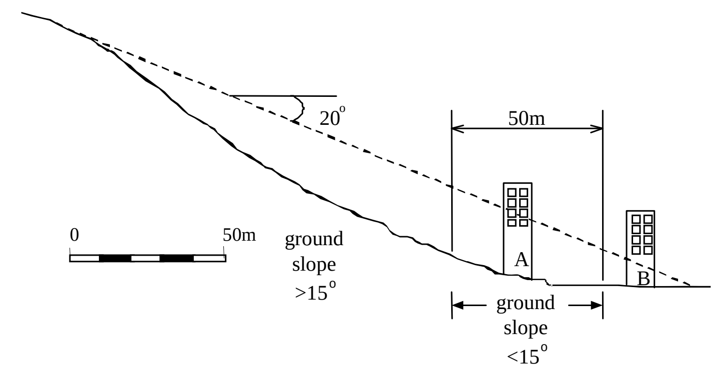
(h) **Guidelines for Natural Terrain Hazard Studies (NTHS)**: The present framework for assessment of natural terrain hazards and mitigation requirements was developed in the early 2000s (Wong 2003). Based on the framework, guidelines on recommended good practice for Natural Terrain Hazard Studies (NTHS) and the design requirements of mitigation measures are set out by Ng et al. (2003).

(i) **Guidelines for design of debris-resisting barriers**: Given the typical scale of debris flows in Hong Kong, it is often practical and cost-effective to adopt debris-resisting barriers as risk mitigation measures. Technical guidelines on assessment of debris impact forces and design of debris-resisting barriers were issued by the GEO (Lo 2000).

(j) **Development and application of Quantitative Risk Assessment (QRA)**: Technical development work undertaken in Hong Kong has been instrumental in formulating landslide QRA methodology, for application to natural terrain landslide risk management. The landslide risk assessment concepts, techniques and applications are summarised in a number of state-of-the-art papers, e.g. Wong et al. (1997), Ho et al. (2000), and Wong (2005).

(k) **Rainfall-landslide correlation**: Study of the correlations between the density of natural terrain landslides and rainfall intensity provided insights into the effects of rainfall on natural terrain landslide occurrence and assisted in risk management and disaster preparedness. The study incorporated the methodology that has been applied over the years in establishing rainfall-landslide correlations for man-made slopes in Hong Kong, together with the use of GIS and statistical techniques to further improve the assessment (Ko 2003; Wong et al. 2004). The detailed rainfall records available in Hong Kong since 1985 and the NTLI data provided the essential data for the study.

(l) **Assessment of debris mobility**: The GEO has commenced empirical assessment of debris mobility since the study of the 1993 Lantau landslides (Wong & Ho 1996). This was followed by analysis of the additional data collated from landslide investigations and from the NLTI. In the early 2000s, a set of simple and conservative guidelines was established, on the basis of runout characteristics of landslide debris. The guidelines have been used in initial screening of land-use planning and development proposals, to assess whether a given site may be subject to natural terrain hazards and may require hazard study and mitigation as part of the development (Figure 8). Development of 2-D and 3-D numerical modelling of debris movement has enhanced the capability of assessment of the debris influence zone and design of risk mitigation works.



Site	Natural Terrain Hazard Study Requirement Criteria		Natural Terrain Hazard Study Required ?
	Within 20° Angular Elevation of Natural Terrain ?	Within 50m of Ground Sloping at > 15°?	
A	Yes	Yes	Yes
B	Yes	No	No

Figure 8: Screening criteria for assessing the requirement for NTHS in new development projects

(m) **Engineering geological assessment**: Engineering geological assessment is essential to NTHS. There has been continual development of natural terrain-related engineering geological mapping techniques in Hong Kong. Process-based regolith classification and mapping methodology was formulated in the Tsing Shan foothills regional study (MFJV 2002; Fletcher et al. 2002). Useful results were reported with the use of geomorphological assessments, e.g. Parry & Ruse (2002), HCL (2003), Mott Connell (2003) and OAP (2004). More recently, the methodology of natural terrain geomorphological mapping has been further developed in the north-eastern Hong Kong Island regional study, incorporating geomorphological interpretation and use of historical landslide data and air-borne Light Detection and Ranging (LiDAR) survey results.

(n) **Novel technology**: Significant advances have been made in the application of digital and remote sensing technologies to enhance the capability and efficiency of NTHS in Hong Kong. Among these, digital photogrammetry, Geographic Information System (GIS), Global Positioning System (GPS), and terrestrial and air-borne LiDAR (Figure 9) have provided promising results. A summary of the technological development and applications is given by Wong (2007). Recent progress made in air-borne LiDAR survey in Hong Kong is described in Ng & Chiu (2008). The improved capability in real-time slope monitoring is also notable (Lau et al. 2008; Solomon et al. 2008).

(a) Hillside in December 2006 | (b) Ground points from LiDAR Survey | (c) Digital elevation model generated using LiDAR data

Figure 9: LiDAR data (from Ng & Chiu 2008)

(o) **Enhanced Natural Terrain Landslide Inventory (ENTLI)**: In recognition of the limited resolution and temporal coverage of the high-flight aerial photographs, the GEO completed in 2007 a major enhancement of the NTLI to incorporate results from mapping of historical natural terrain landslides using the available low-flight (taken at less than 2,400 m) and high-flight aerial photographs (MFJV 2007a). The improved inventory is known as the Enhanced Natural Terrain Landslide Inventory (ENTLI), which replaces the NTLI.

(p) **Inventory of Historical Landslide Catchments (HLC)**: Based on the ENTLI, an inventory of hillside catchments with historical natural terrain landslides that occurred close to existing buildings and important transport corridors was compiled (MFJV 2007b). These catchments are denoted as Historical Landslide Catchments (HLC). The inventory comprises about 2,700 HLC, and is the basic dataset for planning the future risk mitigation works for hillsides flanking existing developments in Hong Kong. The GEO has completed a global QRA to assess the risk levels of the HLC, diagnose their risk characteristics and project the overall risk of natural terrain landslides in Hong Kong (Wong et al. 2004). This provided key information for formulation of the natural terrain risk management strategy. Using the QRA results, a risk-based ranking system was devised for establishing the priority of the 2,700 HLC for systematic follow-up actions (Cheng & Ko 2008).

(q) **Studies of June 2008 landslides**: A severe rainstorm hit Lantau Island in June 2008 and caused about 1,600 natural terrain landslides (Figure 10). Many of the landslides were sizeable and resulted in a much greater runout distance than those that have previously occurred in Hong Kong. Detailed mapping of the landslides and the related technical development studies are being carried out. The landslides serve as a vivid reminder of the risk associated with natural terrain landslides in the densely developed setting of Hong Kong.

Figure 10: Landslides in Tai O, Lantau in the June 2008 rainstorm

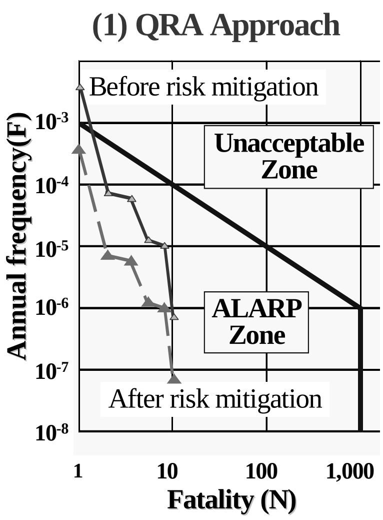
1.4 1977 to 2010 Landslip Preventive Measures (LPM) Programme

Prior to the establishment of GEO in 1977, there was very limited geotechnical control of slope formation both in the private and public sectors. Many old man-made slopes formed before 1977 did not meet the current safety standards, and they constituted the main share of the landslide risk in Hong Kong. Since 1977, the Government has been implementing the Landslip Preventive Measures (LPM) Programme to retrofit substandard government man-made slopes and to conduct safety-screening studies on private slopes, according to their ranked order of priority.

Government's concerted effort in the past 30 years has brought about substantial improvement in the safety of man-made slopes and a significant reduction in the number of landslide fatalities. When the current phase of the LPM Programme is completed in 2010, a total of about 7,000 man-made slopes will have been dealt with under the Programme and the overall landslide risk from man-made slopes will be substantially reduced to less than 25% of that which existed in 1977, reaching a reasonably low level that is commensurate with the international best practice in risk management.

With improved awareness of the potential natural terrain hazards and trend of increasing development close to steep hillsides, administrative measures were introduced in the early 2000s to control undue risk increase from new developments, through avoidance of development in hazardous areas as far as possible, and study and mitigation of hazards as part of the developments where necessary. Ad-hoc studies and mitigation of natural terrain landslide hazards affecting existing developments have also been undertaken following the ‘react-to-known-hazard’ principle, i.e. when significant hazard becomes evident, the case is injected into the LPM Programme for study and risk mitigation. In practice, the majority of the ‘react-to-known-hazard’ cases dealt with under the LPM Programme since 2000 comprised sites affected by newly occurring landslides, and the expenditure incurred constituted a small proportion (within 3%) of that of the LPM Programme.

The insights gained on natural terrain hazards through the technical development work have resulted in rationalisation of the technical approach for dealing with the hazards. Three different approaches, viz., Factor of Safety, QRA and Design Event (Figure 11), are recommended for dealing with natural terrain hazards in Hong Kong. The ‘react-to-known-hazard’ cases dealt with under the LPM Programme and other cases as part of new development works have provided the geotechnical profession with experience in applying the approaches:



(2) Design Event Approach

Required design event	Notional return period
Worst credible	1000 yrs
Conservative	100 yrs
Nil	N/A

(3) FOS Approach

Consequence category	FOS
1 (e.g. building)	1.4
2 (e.g. road)	1.2
3 (e.g. parks)	N/A

Figure 11: Different approaches adopted in assessment and mitigation of natural terrain landslide hazards in Hong Kong

(a) **Factor of Safety Approach**: The conventional engineering approach, which is commonly applied to man-made slopes, has been to design the slopes to the required factor of safety. The relevant slope design requirements are stipulated in the Geotechnical Manual for Slopes (GCO 1984). This approach aims to avert landslides by ensuring a prescribed margin of safety, and is also applicable to natural terrain if the design objective is to reduce the likelihood of slope failure. A typical example of such application is in the study of natural terrain below a development site to ensure that natural terrain landslides which may adversely undermine the site will not occur. However, this approach is often not suitable for use in dealing with a large area of natural terrain that poses a risk to the facilities located at its toe.

(b) **QRA Approach**: This approach is applicable when designers opt for quantification and management of natural terrain landslide risk instead of prevention of failures. Based on this approach, the designer carries out QRA to quantify the risk of natural terrain landslides. The need for any necessary risk mitigation measures is assessed by reference to GEO’s risk guidelines (ERM 1998). This approach would entail a detailed assessment of the probability and consequence of natural terrain landslides, with account taken of the uncertainties in an explicit and systematic manner, and consideration of the tolerability of the assessed risk level. It may be considered as the most rigorous and comprehensive assessment. The assessment often requires expert input and may be fairly involved and costly.

(c) **Design Event Approach**: This approach adopts a risk-based design framework and is applicable when designers opt for mitigation of natural terrain landslide risk without carrying out a formal QRA. Under this approach, the required mitigation measures (e.g. debris-resisting barriers) to protect a development from natural terrain landslides are determined by reference to an assessment of the design landslide event that may occur on the hillside affecting the development. Uncertainties are generally considered in an implicit manner through the assessment of the design event, e.g. a landslide of a certain size with a given degree of mobility. Depending on the potential landslide consequence and susceptibility of the site, the required design event may either be a 'worst credible' event or 'conservative' event, which correspond to a notional return period of 1,000 years and 100 years, respectively. The design event approach is relatively easy to apply as it does not demand formal and rigorous quantification of risk, and is favoured by many practitioners in Hong Kong. However, it gives no provision for consideration of the practicality and cost-effectiveness of risk mitigation. Such consideration is inherent in the QRA approach if the risk level is found to be within the 'As Low As Reasonably Practicable (ALARP)' region.

1.5 Post-2010 Landslip Prevention and Mitigation Programme (LPMitP)

While the overall safety of man-made slopes has greatly improved with the progress of the LPM Programme, the risk of natural terrain landslides is on the rise due to increase in population and more developments taking place close to steep hillsides. From QRA, it was projected that the overall risk of landslides from natural terrain would be comparable to that from man-made slopes by 2010 (Wong et al. 2004). In particular, the identified HLC form a known target group of sites deserving attention, pursuant to the ‘react-to-known-hazard’ principle. This calls for an expanded effort to systematically combat and contain their risk to a level that is as low as reasonably achievable, in order to discharge Government’s due diligence.

In 2007, the post-2010 Landslip Prevention and Mitigation Programme (LPMitP) was endorsed as a long-term landslide risk mitigation initiative (Development Bureau 2007). The LPMitP will commence in 2010 upon completion of the current LPM Programme, as a rolling programme with the following annual output: (a) upgrade 150 Government man-made slopes; (b) conduct safety-screening studies for 100 private man-made slopes; and (c) implement risk mitigation works for 30 natural hillside catchments.

About 50% of the LPMitP resources will be deployed to deal with natural terrain landslide hazards, which is commensurate with the projected risk distribution in 2010. Following the ‘react-to-known-hazard’ principle, catchments in the HLC inventory will be selected based on their risk-based ranking order for action under LPMitP. The LPMitP marks a new chapter in Hong Kong’s landslide risk management, by incorporating systematic study and mitigation of natural terrain landslide risk as an integral part of Hong Kong’s long-term slope safety endeavour.

2 CHALLENGES AHEAD

2.1 Challenging Undertaking

The advances in technology and understanding of natural terrain landslides in Hong Kong over the years have paved the way for combating natural terrain landslide hazards under the LPMitP. However, the challenges that the geotechnical profession will face in this task cannot be overlooked, particularly in view of the following:

- Study and mitigation of natural terrain landslide risk is a relatively new undertaking in Hong Kong. Geotechnical practitioners, including geotechnical engineers and engineering geologists, need to gear up their knowledge and skills in order to meet the new challenges.

- The annual output of natural terrain landslide risk mitigation works under the LPMitP is more than 10 times of that under the current LPM Programme. To cope with the sharp increase in natural terrain work, the demand for human resources and other related provisions for delivery of the LPMitP is enormous.

- Natural hillsides cover large areas and involve highly variable ground and hydrogeological conditions. Their behaviour is affected by geomorphological processes and anthropogenic and climatic influences, which are not fully understood given the current state of knowledge and the variability and uncertainties involved. The conventional geotechnical approaches of detailed ground investigation and slope engineering are generally not applicable. The uncertainties involved need to be properly addressed in risk assessment and mitigation.

- Unlike the retrofitting of man-made slopes, it is often impractical, costly and environmentally undesirable to carry out extensive slope stabilization works on natural hillsides. Instead, natural terrain landslide risk is typically dealt with by provision of mitigation measures, such as the debris-resisting barriers and diversion channels. Hillsides after risk mitigation would remain susceptible to landslides which, in the event of occurrence, will inevitably result in social disruption and nuisance. Furthermore, residual risk will always exist because risk mitigation works are not aimed at achieving 100% risk reduction, which is not credible. The general public may not fully comprehend what can be achieved and the uncertainties involved.

- The number, scale and severity of natural terrain landslides are sensitive to rainfall conditions. Given the observed trend of climate change, it is possible that extreme rainfall conditions will occur more frequently in future. This could result in increased impact from natural terrain landslides and introduce further uncertainties to risk assessment and mitigation.

2.2 What Could Go Wrong

The geotechnical profession has to be vigilant in meeting the challenges associated with natural terrain hazards. The few years ahead may involve a critical period for building up experience and overcoming teething problems. The following are some possible scenarios of what may go wrong:

(a) **Occurrence of a landslide/debris flow with debris volume significantly in excess of the Design Event allowed for in the provision of mitigation measures**: This could result in significant damage or collapse of debris-resisting barriers, over-flow of debris from fully filled barriers, etc. The scenario may arise from under-estimation of the possible scale of the source failure volume, e.g. an unforeseen deep-seated failure, shallow but spatially extensive landslide, and multiple-source failures. Insufficient provision for a debris flow involving significant entrainment or watery debris, which can escalate the debris volume, may also give rise to this problem.

(b) **Occurrence of a landslide/debris flow discharging debris that is significantly more mobile than that considered in the design of mitigation measures**: This could result in significant damage or collapse of debris-resisting barriers, impact on areas that have been assessed as being beyond the reach of the debris, etc. Inadequate allowance for debris mobility may occur because of under-estimation of debris volume. It may also be the outcome of a lack of comprehension of the possibility of development of watery debris. Use of inappropriate rheological models and debris runout parameters may also under-predict the reach of landslide debris.

(c) **The actual debris runout path is significantly different from that considered in the design of mitigation measures**: This could result in debris hitting unprotected zones that have not been recognized as being within the debris runout path, whereas the debris-resisting barriers are placed at wrong locations that fail to intercept the debris runout. The debris runout path may be wrongly predicted if the debris runout is simply prescribed as following the direction of the steepest slope, without due consideration of other relevant factors that may change the debris runout direction. Use of unreliable digital terrain data and dynamic debris modelling algorithms, e.g. those that do not cater for possible bifurcation of the debris trail, could also result in a misleading assessment of the likely debris flow path.

(d) **Occurrence of a landslide in a low-ranked HLC resulting in serious consequences**: This could cast doubt on the reliability of the priority ranking methodology and possible human errors in assessing the ranking scores. It could also invite criticism that the LPMitP should be speeded up. The scenario may occur in association with some ‘unfortunate’ combinations of events that lead to dire consequences, e.g. a debris flow inundating vehicles queuing on a minor road that is blocked by flooding or a minor roadside landslide (Figure 12), as in the case of the 1995 debris flow that caused 22 fatalities at a slip road in the Genting Highland, Malaysia (Abdullah et al. 2007). The public’s perception may even be worse if the failure occurs on a catchment adjoining other catchments where mitigation works have been carried out but without occurrence of any noticeable landslides in the rain storm.

Figure 12: Debris flow affecting a road with low traffic density
(Note: Consequences can be serious in case of queuing of traffic on the road during debris flow)

(e) **Serious failure at a hillside that has not been identified as an HLC**: This could arouse scepticism about the ‘react-to-known-hazard’ principle, and the criteria adopted and any human errors in the identification of HLC. The scenario could occur if the hillside that fails does not have any known past landslides, or the known past landslides occurred at some distance from the existing facilities (i.e. not meeting the proximity criteria adopted for identification of HLC), or where the existing facilities being affected are not categorised as important facilities (i.e. building structures and important transport corridors) as considered in identification of HLC. This may result in a demand for adopting a more proactive approach in searching for catchments that are vulnerable to landslides, or expanding the criteria adopted in compiling the HLC inventory, or both.

(f) **Inadequate emergency response**: The number of natural terrain landslides will increase exponentially with rainfall intensity. Thousands of natural terrain landslides could occur in an extreme rainstorm, and pose an acute and unprecedented strain to the landslide emergency response system. Should the emergency system become overloaded, possible drop in efficiency in attending to landslide incidents may result in delay in taking emergency actions to minimise risk exposure. Difficulty in mobilization of a large number of experienced inspection teams to ascertain the hillside conditions and the possible residual risk after widespread landslides, particularly in inclement weather conditions, could result in emergency decisions being made in the face of considerable uncertainties. Recommendations on emergency actions to be taken, which subsequently turn out to be either insufficiently safe or too conservative, could affect public safety and convenience and attract dissatisfaction. The June 2008 rainstorm that affected western Lantau is an illustration of the potential challenge. Had the rainstorm hit a more densely populated region, the demand for emergency responses would have been much more severe.

Scenarios (a) to (c) above are directly related to practitioners that are involved in natural terrain risk assessment and mitigation works. In the event of occurrence, they could result in concerns being raised about the adequacy of the risk mitigation provisions, credibility of the professional practice, and professional competence. Scenarios (d) to (f) above primarily involve the 'risk manager', i.e. the GEO. Apart from putting the natural terrain risk management strategy in question, they could potentially jeopardise the public's trust in Hong Kong's Slope Safety System.

There is no room for complacency notwithstanding the advances made in the subject of natural terrain landslides in Hong Kong over the years. As we embark on a new era of landslip prevention and mitigation works under the LPMitP, more has yet to be learnt by the geotechnical profession in improving the understanding of natural terrain landslides and capability in combating their risk. Some specific aspects of technological development concerning natural terrain failures, debris movement and risk mitigation strategy are highlighted in the next few sections.

3 INSIGHTS INTO NATURAL TERRAIN FAILURES

3.1 Some Misunderstanding

Before better knowledge about natural terrain landslides became available, there had been some early suppositions that the natural hillsides in Hong Kong are old and mature landforms and thereby should have evolved into a relatively 'stable' condition. Hencher (2000) gave a commentary on these early suppositions, with reference to the observations reported in Lumb (1975) and Ruxton (1980). These early suppositions included the views that natural hillsides show no signs of creep, all failures are first-time slips, and the concept of 'ripening' being dismissed in view of the long time needed for the degree of weathering to become significantly altered by chemical weathering.

These suppositions might sound reasonable in the light of the Second Law of Thermodynamics, which states that the entropy of an isolated system which is not in equilibrium will tend to increase over time, approaching a maximum value at equilibrium. In simple terms, it implies that over time, differences in temperature, pressure, density, etc. will tend to even out in a physical system that is isolated from the outside world. Given the longevity of the existence of the Hong Kong rocks and the relatively inactive tectonic setting, one might expect that the terrain should have reached an advanced stage of self-equilibrium (i.e. entropy). As a contrast, in the Campania Region of Italy, rain-induced natural terrain landslides occur on pyroclastic deposits (Figure 13) that have been in place for only a few hundred years (Versace et al. 2007). This notion could lead to a false sense of security about the stability of natural terrain in Hong Kong.

Figure13: Natural terrain landslides in Sarno, Campania, Italy
(Note: Photo from http://www.commissario2994.it)

Whilst acknowledging that 'natural slopes are frequently close to limiting equilibrium over very large areas', the Geotechnical Manual for Slopes (GCO 1984) stipulates that natural slopes need not meet the factor of safety requirements for slope design provided that two conditions are met: (i) the slope is undisturbed; and (ii) a careful examination is made to determine that there is no evidence of instability or severe surface erosion. There was apparently a time in the past that some practitioners presumed that these two conditions could be easily met in many places. Indeed, as far as natural terrain hazards are concerned, much of the attention was given in the past to boulder falls, instead of slope instability. In 1997, the Geotechnical Control Conference of the GEO clarified that 'evidence of instability' refers to 'evidence relevant to future instability'. It is now known that the above two criteria are very difficult to meet in reality. Hong Kong's hillsides rarely have a clean sheet in terms of past instabilities, not to mention potential future instabilities. Furthermore, disturbed hillsides are also fairly commonplace within or bordering developed areas.

3.2 Proper Perspective

In the 1990s, with the large amount of data available from the study of the November 1993 Lantau landslides, review of records of failures in the rainstorms of 1966 and 1982, together with compilation of the NTLI, it became evident that natural hillsides in Hong Kong were rather susceptible to rain-induced, shallow failures. Field investigations have revealed that failures typically occur within 1 to 2 m of the surface mantle, where erosion pipe holes, dilation and partial infilling of relict discontinuities, and localised tension cracks are often observed. The hillsides are subject to on-going degradation, and a large number of shallow landslides can be triggered by heavy rain.

Shallow landsliding is still an active process on the natural hillsides, under the prevailing climatic conditions. The potential for further failures is far from depletion, even on catchments that have many known historical landslides. This is also supported by the findings of rainfall-landslide correlations that the density of natural terrain landslides increases exponentially with rainfall intensity (Figure 14). Landslide occurrence has been found to be sensitive to rainfall intensities that cover durations ranging primarily from 4 to 24 hours, which is consistent with the observed shallow depth of failure triggered by slope saturation and transient build-up of water pressures as a result of infiltration and sub-surface seepage flows.

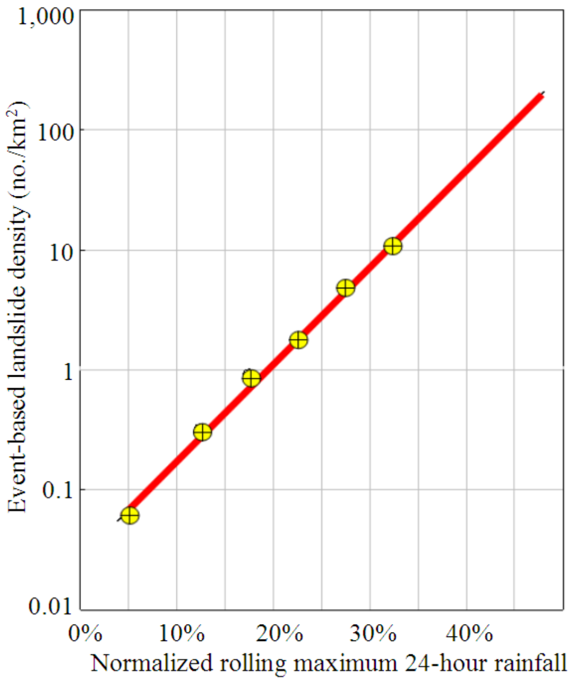
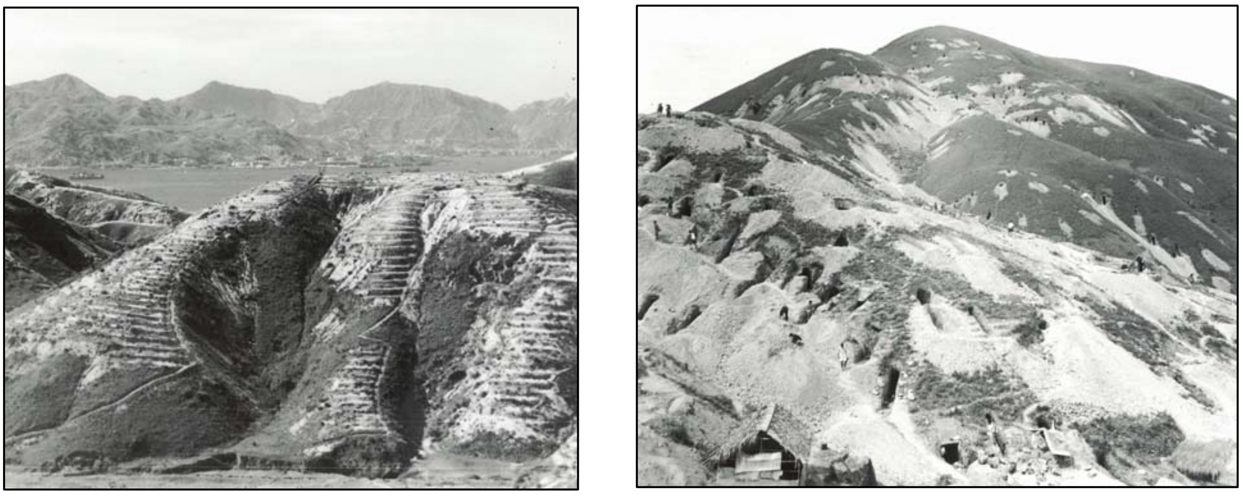


Figure 14: Rainfall-natural terrain landslide correlation

$$(\text{Note : Normalised rainfall} = \frac{\text{rainfall intensity}}{\text{mean annual rainfall}})$$

The apparent mismatch between the active landsliding process and the relatively old landform may be explained by a number of reasons. Firstly, the weathered profiles that overlie the solid rocks are much younger than the rock formation. Ruxton (1980) postulated that the weathered profiles in Hong Kong might be more than 200,000 years old, while it might take about 17,000 years for one metre thickness of rock to be weathered completely to halloysite and quartz. Hence, the surface one metre or so of the ground, which is susceptible to landslides, may be relatively young as far as its present properties are concerned. Secondly, the present-day hydrogeological and climatic conditions may be rather different from those in the geological past. This could be reflected by the drastic changes in the sea level over the past 20,000 years, with the current sea level possibly near the historical highest level. Thirdly, the hillsides may have been disturbed, to different degrees, in the recent history. Deforestation took place in Hong Kong in the past several hundred years including the Second World War period. This could have effects on slope hydrogeology, which affects slope stability. Anthropogenic activities, including formation of walking trails and roads, mining, military trenches and tracks, grazing, localised cutting and filling, cultivation, village graves, etc, could bring physical disturbance to the terrain (Figure 15).



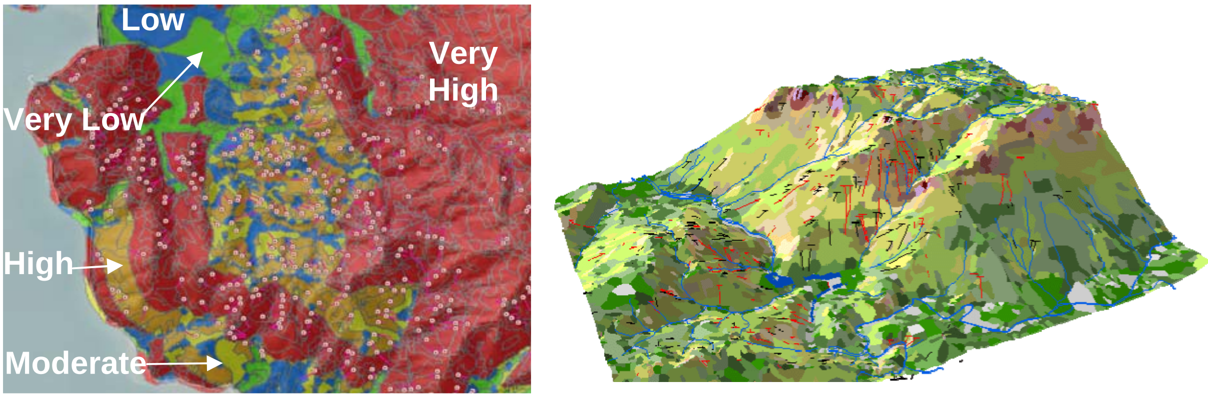
(a) Agricultural terracing

(b) Mining activities

Figure 15: Human disturbance to hillsides in Hong Kong

Statistical analyses of landslide susceptibility provide further insight into the spatial distribution of natural terrain landslides. The territory-wide landslide susceptibility analysis by Evans and King (1998), based on correlation with slope angle and geology, found that the natural terrain of Hong Kong could be differentiated into five susceptibility classes. The calculated landslide densities for the five classes vary from ≤10 to >100 landslides per km^2, which corresponds to an average frequency of failure from about ≤0.1 to >1 landslides/km^2/year (Figure 16). Wong (2003) diagnosed the implications of the limited resolution in the calculated landslide frequency among the classes, which spans only about one order of magnitude between the least and most susceptible classes. Such a resolution is considered insufficient for differentiation of vulnerable hillsides, given the potentially high consequence of landslides in Hong Kong. From a different perspective, the low resolution probably demonstrates a lack of understanding of the quantifiable factors that control landslide susceptibility. It may also reflect the possibility that the natural hillsides in Hong Kong are generally susceptible to failure, with a relatively small difference in the landslide susceptibility between the more problematic and less problematic terrain.

Combined rainfall-susceptibility analyses by GEO showed that spatial and temporal variations in rainfall intensity have a significant influence on landslide susceptibility. In particular, it was observed that at low rainfall intensity, landslides tend to occur predominately on more susceptible terrain. However, under high rainfall intensity, even less susceptible terrain would suffer from failures. This leads to the same inference: Hong Kong's steep hillsides are actively responding to heavy rain and are far from reaching a state of high entropy against shallow failures.



(a) 2-D landslide susceptibility map (b) 3-D presentation

Figure 16: Terrain landslide susceptibility classification.

(Note: Very High = landslide frequency > 100 no./km^2
High = landslide frequency 40 - 100 no./km^2
Moderate = landslide frequency 20 - 40 no./km^2
Low = landslide frequency 10 - 20 no./km^2
Very Low = landslide frequency ≤ 10 no./km^2)

The above observations do not apply to deep-seated failures, say, depth of landslide exceeding a few metres. Deep-seated failures rarely occur on the natural terrain of Hong Kong. Some of the isolated cases of deep-seated landslides that have taken place were significantly affected by human activities, e.g. the 1995 Shum Wan Road landslide (GEO 1996), which was affected by uncontrolled discharge of a large amount of surface water from the road that traversed the site. Some were slow-moving landslide bodies subject to prolonged deformation/displacement, e.g. the Tsing Shan debris lobe (Parry & Campbell 2003). Some appeared to be controlled by special geological structures or conditions (e.g. McMackin et al. 2009). There are no indications that the natural hillsides in Hong Kong are active in deep-seated landsliding process. It seems that, in general, such a process has reached a state of high entropy, although hillsides with latent geological weaknesses may be re-activated by major human disturbance or under very adverse weather conditions.

3.3 Recent Development

Recent studies, including those on the June 2008 landslides, are throwing further light on natural terrain failures in Hong Kong. The following are some reflections on areas for further advances in the assessment and mitigation of natural terrain hazards:

(a) **Insights from historical landslides and geomorphological assessment**: The ENTLI has provided improved data for assessment of the spatial and temporal distribution of natural terrain landslides. Analysis of the June 2008 landslides on Lantau showed that 80% of the landslides occurred within 50 m of two or more ENTLI features, and 92% within at least one ENTLI feature. When the proximity criterion is reduced to within 30 m, the calculated figures are still as high as 55% and 79%, respectively. Historical landslides appear to have certain degree of clustering, and new landslides tend to occur within or close to these clustered zones (Figure 17). Further work is required to interpret the phenomenon in geomorphological and engineering geological context, to facilitate demarcation of terrain that is more prone to failure.

(b) **Prospect of susceptibility analysis**: So far, landslide susceptibility analyses have mainly focused on consideration of landslide density, without explicit classifications that account for landslide type, scale, runout distance, etc. This would not only affect the resolution of the analysis due to lumping of different types of landslides in the analysis, but also render the results of susceptibility zoning not useful. While it is tenable that steep hillsides are all susceptible to failures, the majority of the failures are small-scale landslides with insignificant risk concern. Confining the susceptibility analysis to sizeable landslides and debris flows, i.e. excluding the more commonly occurring small-scale failures, may give more insights into the potential sources of major hazards. Many other countries are giving increasing attention to landslide susceptibility and hazard zoning for use in development planning and landslide risk management. Publication of related technical guidelines by AGS (2007) and JTC-1(2008) may further promote the work. Hong Kong's experience has shown that the susceptibility and hazard zoning methodology commonly adopted in other countries is of limited resolution and reliability for dealing with relatively small-scale failures, in Hong Kong's context. With the use of improved data and methodology, there could be some prospect of further pursuing the work and achieving better results.

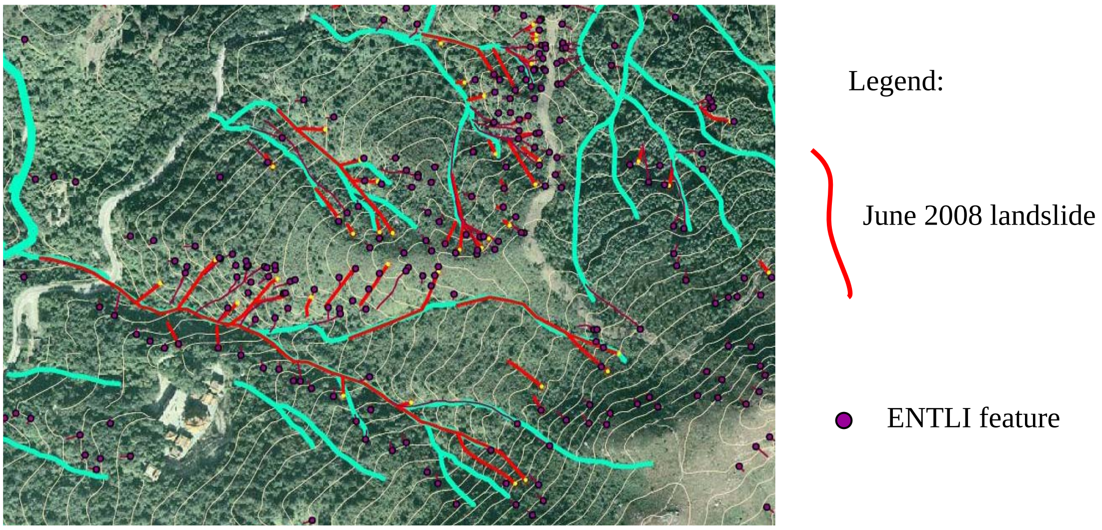
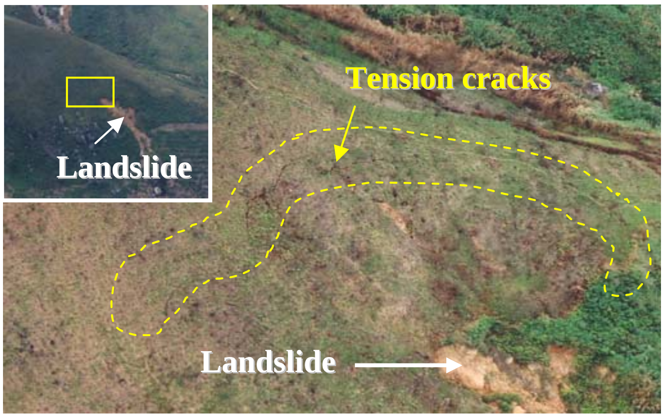


Figure 17: June 2008 natural terrain landslides occurred in closely proximity to ENTLI features

(c) **Hillside deterioration**: While field data showing evidence of hillside deterioration have become more abundant (e.g. Hughes et al. 2002; Wong et al. 2004; MGSL 2007), details of the deterioration process are still not fully known. These include possible causes and mechanisms of deterioration, time span to ripening, and factors controlling the transition from slope distress to occurrence of uncontrolled detachment. From time to time, tension cracks are found on hillsides after heavy rain or after vegetation is burnt off by hill fire (Figure 18). These may be newly formed tension cracks, or pre-existing cracks subject to intermittent movement. Development of tension cracks would affect the hydrogeology and strength of the groundmass, and significantly alter the stability conditions of the hillside. It appears that some hillsides may go through a prolonged stage of development of tension cracks, or other forms of signs of distress (e.g. slope deformation involving dilation of relict discontinuities in weathered rock, Figure 18), before occurrence of uncontrolled detachment. This offers some scope for use of continuous slope monitoring to acquire information for diagnosing the hillside behaviour and managing landslide risk. It also brings out the importance of identification of signs of distress in NTHS and the need for re-assessment after development of signs of distress on a hillside that has previously been subjected to NTHS.

(d) **Low-frequency, large-magnitude events**: Low-frequency, large-magnitude landslides pose a distinct challenge to landslide risk management, in that a large failure may lead to serious consequences and that little knowledge is available about the nature of these infrequent events and where they may take place. While the 1990 Tsing Shan debris flow and 1995 Shum Wan Road landslide may be taken as examples, there seems to be a perception that similar events rarely occur in Hong Kong. Recent mapping of the June 2008 landslides reveals that large magnitude events, in particular sizeable debris flows with long runout distance, may not be as rare as one might have perceived (Figure 19). These newly available field data indicate that a range of circumstances could result in sizeable failures or debris flows. They include: (i) deep-seated landslides controlled by geological structures or involving re-activation of relict landslide mass; (ii) shallow failures that are spatially extensive; (iii) detachment of distressed hillsides with extensive tension cracking; (iv) confluence of multiple landslides in a debris flow catchment, forming a sizeable debris flow; and (v) debris flows involving significant entrainment or watery debris, particularly at major drainage lines. Further work is needed to enhance our knowledge of these events and capability in assessing and managing their risk.



(a) Extensive tension cracking above a recent landslide in Tsing Shan foothills

(b) Open and infilled discontinuities at landslide scarp of the main source of the June 2008 Shek Mun Kap debris flow

Figure 18: Signs of deterioration of natural hillsides

Figure 19: Large debris flows on Lantau in the June 2008 rainstorm
(Note: Shek Pik No. 1 and No. 2 debris flows are shown; see Figure 5 for debris flows in the area in November 1993)

4 UNDERSTANDING DEBRIS MOVEMENT

4.1 Prevailing Knowledge

Establishing the possible reach of debris runout is essential to assessing natural terrain landslide risk. Although studies on debris mobility were started in Hong Kong several years later than in some other countries, major advances have been made over the years, which put Hong Kong on a par with other key technical leaders. This is attributed to: (i) availability of good quality landslide data on debris runout; (ii) development of numerical modelling capability in parallel with acquisition of field data, which facilitates understanding of mechanisms and calibration of numerical models; and (iii) a demand for application given the need for assessment of landslide consequences and design of debris-resisting structures.

In respect of assessment of debris mobility, the progress made and knowledge acquired over the years may be summarised as follows:

(a) **Empirical assessment**: Collection of field data on the runout of landslide debris since 1993 has enabled empirical correlations of debris mobility with landslide mechanism and volume. The work initially covered landslides on man-made slopes (Wong & Ho 1996), and was extended to include natural terrain landslides (Lau & Woods 1997; Wong et al. 1998). The correlations were principally based on consideration of the debris travel angle (Cruden & Varnes 1996), which is related to the apparent angle of friction that accounts for the energy loss in the event of a frictional material sliding along the runout path. When applied to steep hillsides, use of travel angle in assessing the reach of debris has some limitations, in that uncertainties in travel angle may result in significant difference in the predicted travel distance. Direct correlations with travel distance offer a convenient alternative, although this is not rigorous in terms of energy consideration. Attempts were made in such correlations, which have occasionally been applied in site-specific NTHS (e.g. OAP 2003). In other cases, empirical correlations were made with proximity zones defined by a combined consideration of travel angles and travel distance, using historical landslide data (Figure 20). These have been adopted in global QRA and in site-specific NTHS (Wong 2005).

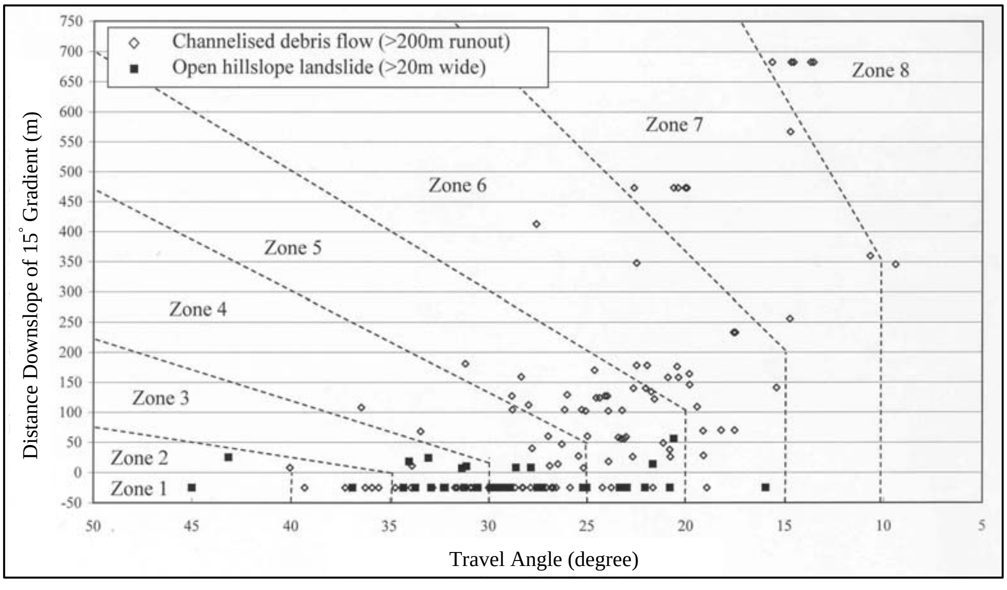


Figure 20: Empirical proximity zoning based on historical debris runout data

(b) **Analytical simulation**: In the late 1990s, analytical approaches were introduced for use in assessment of the reach of landslide debris in Hong Kong. The mass-balance approach has been adopted to evaluate the changes in the active volume of a debris flow based on empirical correlations between the rates of debris entrainment/deposition and channel characteristics, e.g. gradient and channelization ratio (Lau & Woods 1997; OAP 2004). However, this method suffers from lack of consideration of debris flow rheology and may give misleading results if the empirical data are not representative of the characteristics and site conditions of the debris flow that is being analysed. Dynamic modelling of debris as a continuum based on consideration of the principles of conservation of mass, momentum and energy has gained popularity. The method gives a more rigorous simulation of the debris flow rheology. The governing debris runout input parameters may be back-analysed from historical landslide cases. The 2-D Dynamic Analysis (DAN) model developed by Hungr (1995) was introduced for use in Hong Kong (Ayotte & Hungr 1998). Subsequently, the GEO developed its own 2-D dynamic modelling algorithm, commonly known as Debris Mobility Modeller (2D-DMM, Figure 21), based on similar formulation and solution methodology (Kwan & Sun 2006). Since then, the 2D-DAN and 2D-DMM codes have been routinely applied in the assessment of debris mobility in Hong Kong. Based on what was known at the time, Lo (2000) reviewed methods of debris mobility analysis, and gave suggestions for assessing debris mobility and debris impact loads in the design of landslide debris-resisting barriers.

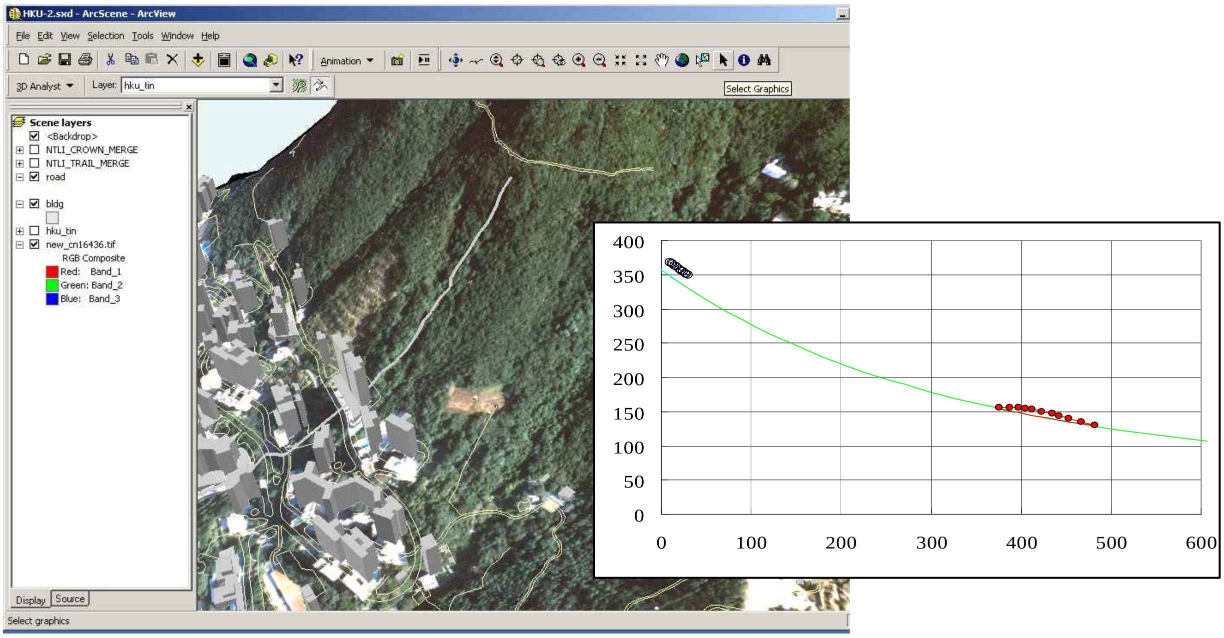
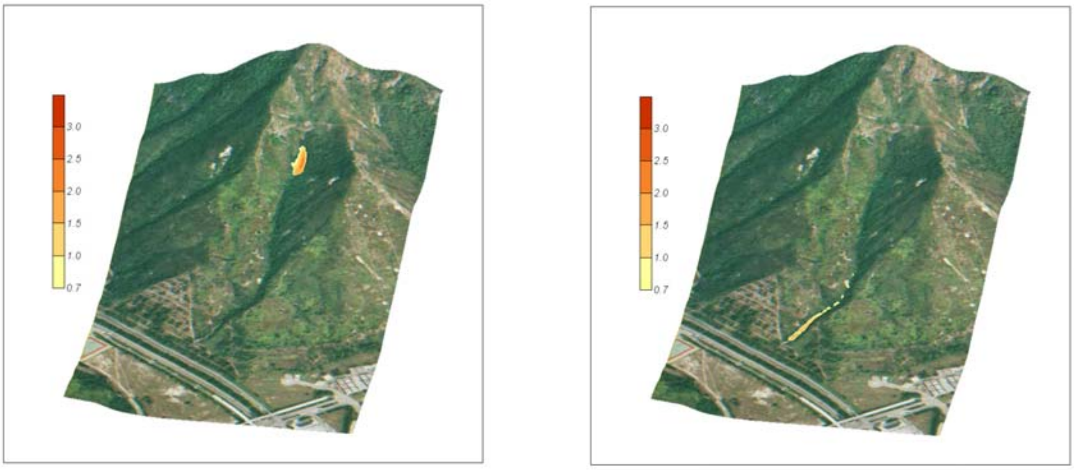


Figure 21: 2D-DMM debris runout modelling

(c) **Advanced numerical modelling**: In recent years, there was major development of 3-D dynamic continuum modelling capability. The key areas of development and performance of these 3-D algorithms are described in an expert panel review report on the international benchmarking exercise on debris mobility modelling hosted in Hong Kong in 2007 (Hungr et al. 2007a). The exercise revealed that several 3-D algorithms had the capability of simulating a wide range of cases and achieving consistent performance. These included: the DAN3D (McDougall 2006; Hungr et al. 2007b), GEO's 3D-DMM (Kwan & Sun 2007), RASH3D (Pirulli 2005; Pirulli & Scavia 2007), and the SPH code developed by Pastor et al. (2007). The finite element code MADflow developed at the University of Hong Kong (Chen et al. 2006) also gave promising results, although it could not cater for splitting and merging of landslide debris. These 3-D algorithms call for solution methodologies that are different from their 2-D counterparts, but the rheological models and debris runout parameters adopted are the same. The 3-D codes offer several distinct functionalities, which are superior to 2-D modelling and important to risk assessment: (i) the debris runout path and the lateral debris inundation zone are simulated in the modelling, instead of subjectively prescribed; (ii) splitting and merging of debris during runout, which occurred in many actual cases, can be allowed for; and (iii) effects of entrainment, presence of debris diversion and retention facilities, debris flow depth, length of the debris flow mass, 3-D profile of debris deposition, etc. can be more realistically simulated (Figure 22). Apart from these algorithms, the FLO-2D (Julien & O'Brien 1997) and PFC (ITASCA 1999) codes, which are commercially available, have also been applied in debris runout analysis in some special circumstances. FLO-2D adopts flood routing models and has been used in 3-D simulation of debris flood events in Hong Kong (Figure 23). PFC models the dynamic behaviour and interaction of discrete particles, and can be applied to simulate rock falls and avalanches.



(a) When debris entered drainage line (b) When debris was discharged at the drainage outlet

Figure 22: Three dimensional debris mobility modelling of the June 2008 Yu Tung Road debris flow using 3D-DMM (Note: Modelling results showed that the length of the debris body increased from about 50 m in (a) to about 190 m in (b), which is consistent with the video record of the debris flow)

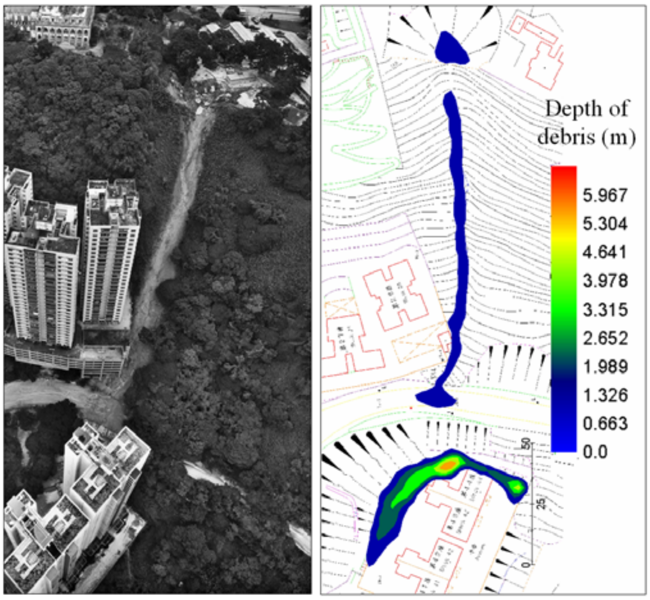


Figure 23: FLO-2D simulation of debris flow event
(Note: The 1992 Baguio Villa landslide is shown)

Based on the findings of the back-analyses of some 20 natural terrain landslide cases in Hong Kong (Ayotte & Hungr 1998), the GEO suggested the following conservative scenarios for use in the assessment of natural terrain landslide debris runout (Lo 2000): (i) for open hillside failures, the Friction rheology can be used with an apparent angle of friction of 25° for debris volumes < 400 m^3 and 20° for debris volumes ≥ 400 m^3; and (ii) for channelised debris flows, either the Friction rheology with an apparent angle of friction of 20°, or the Voellmy rheology with an apparent angle of friction of 11° and a turbulence coefficient of 500 m/s^2 can be used.

Further back-analyses were carried out by the GEO in recent years. These included review of the empirical runout data extracted from the NTLI and systematic back-analysis of about 60 known long runout cases. The work has shown that use of the Voellmy rheology is more appropriate in simulation of debris flows in Hong Kong. The probabilistic distribution of different sets of runout parameters for mobile debris flows was derived (Figure 24), which provides improved data for use in predicting the debris impact zones and assessing landslide risk under a probabilistic framework.

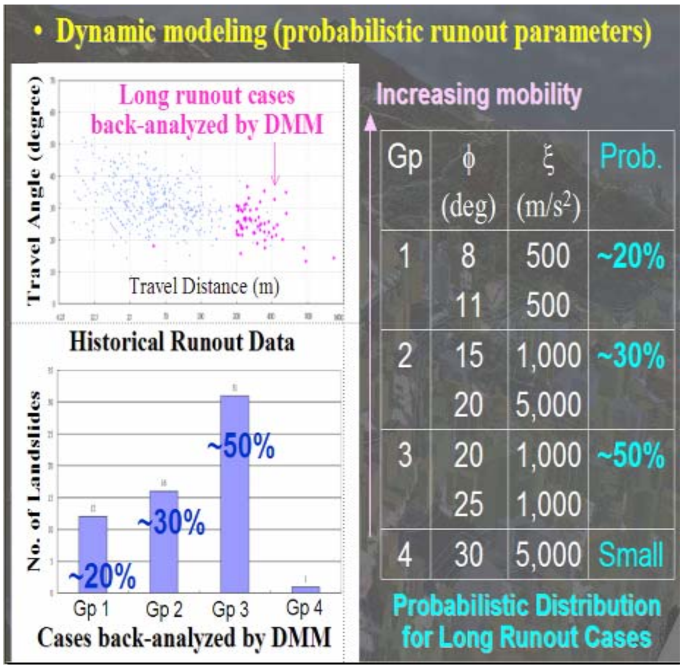
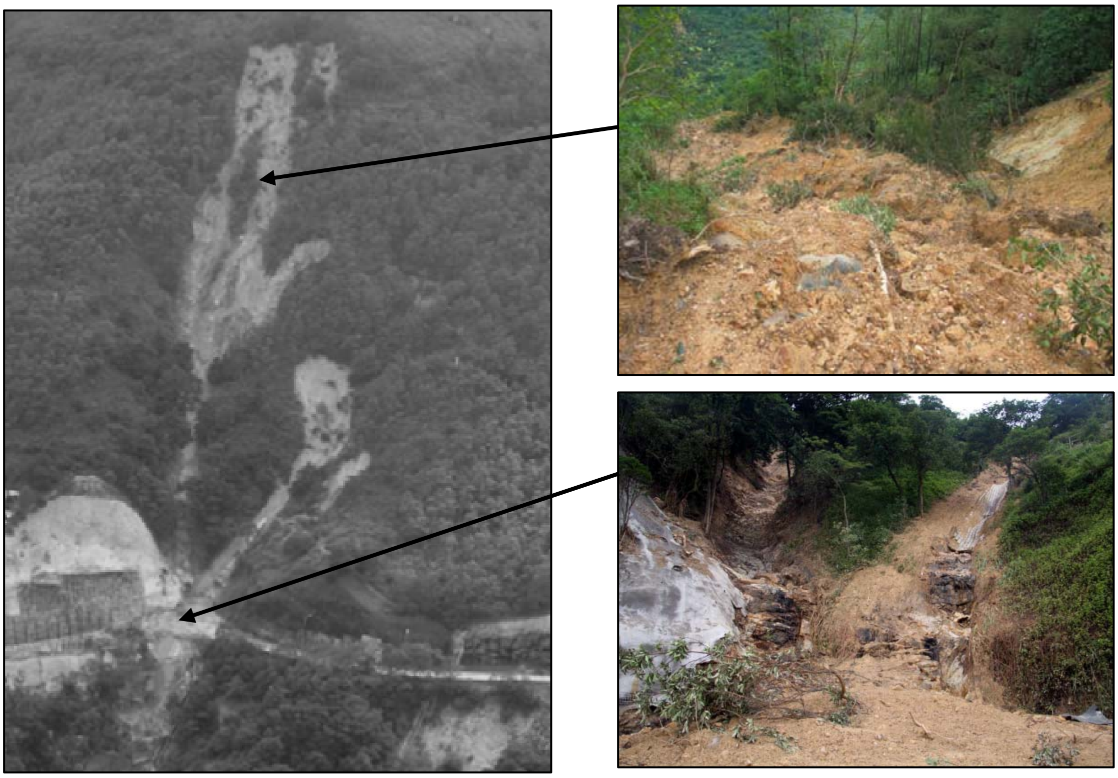


Figure 24: Probabilistic distribution of runout parameters for mobile debris flows based on back-analysis of historical long runout cases

While the potential risk of debris flows is well recognised given their mobility and concentrated discharge, there seems to be a general perception that debris flows in Hong Kong are relatively 'dry' events compared with those in other countries. This stems from the consideration that hillsides in Hong Kong have a limited altitude and do not contain sizeable drainage basins and large stream courses (Figure 25). This is consistent with the general observations from studies of natural terrain landslides in the past 20 years, which suggested that the debris flow events did not contain high water content. Experience acquired from NTHS in recent years also indicated that the scale of the Design Event for debris flow risk mitigation was typically in the order of several hundred cubic metres. These notions are implicit in the rheological models and runout parameters derived from back-analysis, which were benchmarked with the same dataset of known historical natural terrain landslides. However, the available dataset only covers natural terrain landslides that occurred in Hong Kong over the past few decades. This is a relatively short observation period as far as extreme events are concerned.

4.2 Recent Observations

The June 2008 rainstorm is arguably the most intense event since the setting up of the GEO. In particular, its rolling maximum rainfall intensities over the 2-hour to 4-hour duration were exceedingly high, with a statistical return period of about 500 to 1,000 years over a large part of western Lantau Island (Figure 26). The 24-hour intensity was also severe, with a return period of about 100 to 200 years. Previous work on rainfall-natural terrain landslide correlations has shown that 4-hour to 24-hour rainfall intensities are critical to triggering natural terrain landslides in Hong Kong (Wong & Ho 2006). Hence, the severity of the June 2008 rainstorm resembles a 'worst credible event' scenario to be considered in assessment and mitigation of natural terrain landslide risk, based on the design requirements stipulated in the Design Event Approach (Ng et al. 2003). The landslides in this rainstorm provide reference information on the possible characteristics of design events for natural terrain risk management.



(a) Hillside with a small catchment and drainage line

(b) Debris flow involving relatively 'dry' debris

Figure 25: Typical debris flow in a small catchment
(Note: The June 2008 Keng Shan Road debris flow No. KS2 is shown)

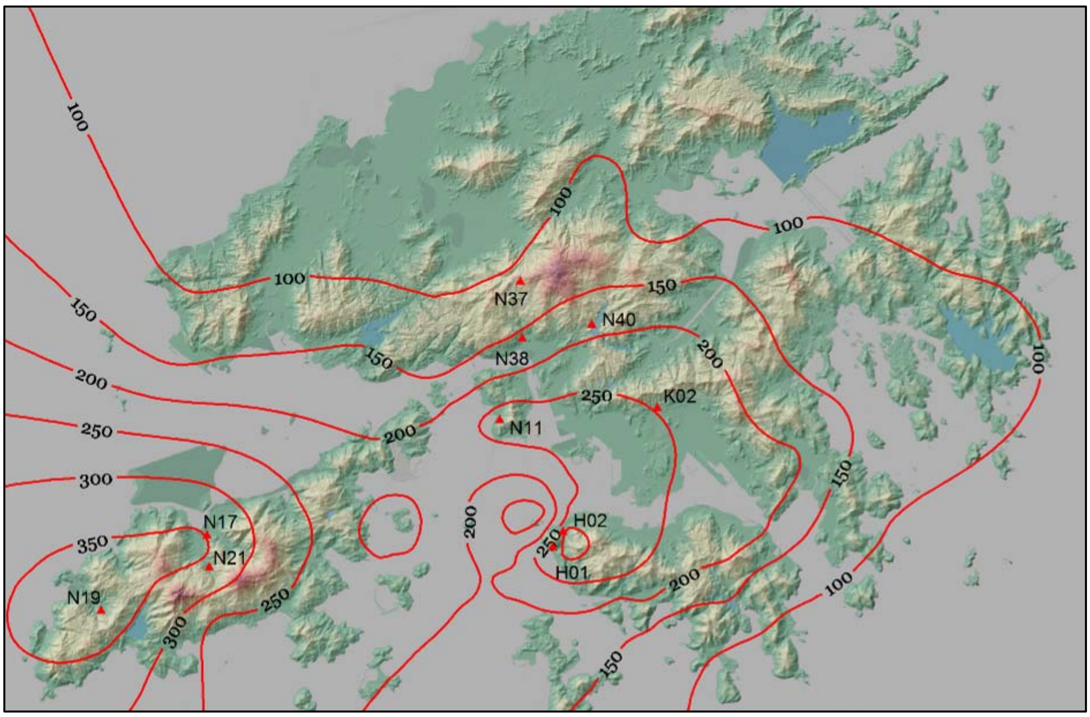


Figure 26: Isohyets of maximum rolling 4-hour rainfall in the 6 to 8 June 2008 rainstorm
(Note: Statistical return periods at raingauges No. N19, N17 and N21 are 1,100, 570 and 485 years, respectively)

Studies of the June 2008 landslides to date have brought about new observations about debris movement:

(a) **Increased debris mobility**: It is known that natural terrain landslide density escalates with rainfall intensity. Data from the June 2008 landslides, when compared with the previously available data on historical natural terrain landslides, indicate that debris flows in a severe rainstorm can also become more mobile. Figure 27 shows that the June 2008 landslides on Lantau have considerably higher mobility than that of the historical landslides in the ENTLI. Studies of debris runout form an important part of the field and analytical work that is being carried out on the June 2008 landslides. There is a need to revisit the back-analyses that have previously been undertaken on debris mobility, by incorporating the newly available data. The findings would help to improve the robustness of the assessment of the reach of landslide debris, as well as the debris velocity and impact forces to be considered in the design of mitigation measures.

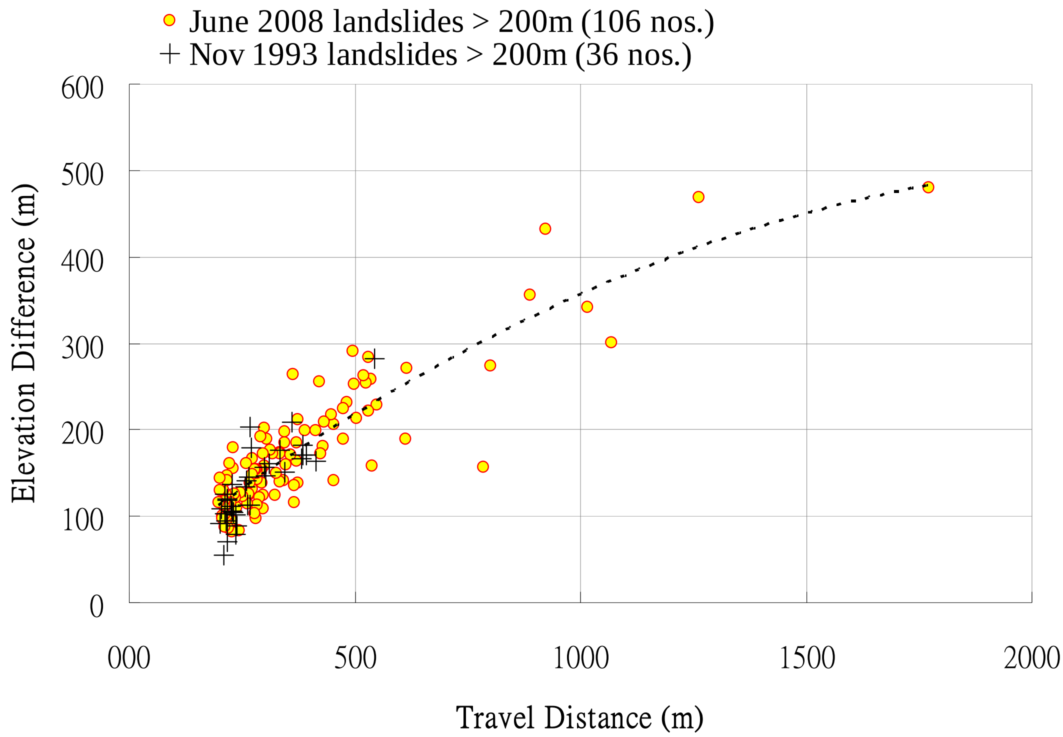


Figure 27: Data showing many long runout and high mobility landslides in the June 2008 rainstorm (Note: 18 debris flows in the June 2008 rainstorm have runout distance exceeding 500 m, while there are only 10 historical cases in the ENTLI including one case on Lantau Island; landslides with runout distance ≤ 200 m not shown for clarity)

(b) **Debris flows with watery debris**: While the high debris mobility is partly related to increase in the volume of debris flows in some cases, field mapping has found that some of the long runout landslides in the June 2008 rainstorm evidently involved flows of watery debris (Figure 28). Watery debris was mobile due to its high water content, which is contrary to the prevailing perception that debris flows in Hong Kong tend to be relatively 'dry'. The bulked, active volume of the debris flows, with solid mixed with a large amount of water, could be much greater than the volume that is normally estimated from the size of the landslide mass. The entrainment and deposition characteristics of these watery debris flows are different from those of the less wet events. Wet debris deposited on the drainage line, e.g. forming a local landslide dam or deposited mass with high, unconsolidated pore water pressures, would be more susceptible to remobilization in the event of passage of a subsequent debris flow. These have significant implications to risk assessment and mitigation (Figure 29).

Figure 28: Deposits of a watery debris flow at the deposition zone
(Note: The June 2008 Shek Pik No. 4 debris flow is shown; the deposits have been subject to considerable sorting)

Figure 29: Grid-type steel sabo structure
(Note: The open structure would trap sizeable clasts and allow free drainage; photo from Lo 2000)

(c) **Significance of entrainment**: The 1990 Tsing Shan debris flow is known to be a case of significant entrainment, which was thought to be unusual in the past. Many debris flows with significant entrainment, e.g. entrainment ratio up to ten or even higher, occurred in the June 2008 rainstorm. Many of the long runout debris flows in the rainstorm started with a small landslide at the source, e.g. within two to three hundred cubic metres. The active volume escalated as the materials on the drainage line were entrained in the debris flows, and the eventual scale of the debris flows was largely controlled by the degree of entrainment (Figure 30). This shows the importance of assessment of the potential for entrainment in managing debris flow hazards.

Figure 30: Debris flow with significant entrainment in the June 2008 rainstorm
(Note: Debris flow above Shum Wat Road is shown; the source volume is less than 100 m^3 and the entrainment ratio exceeds 10)

(d) **Dynamic nature of drainage lines**: Mapping of the June 2008 debris flows suggests that entrainment tends to involve the materials perching on the drainage line, instead of depleting extensively into the existing side slopes and bed of the drainage line (Figure 31). This seems to suggest that the side slopes and bed of the drainage line are generally more resistant to entrainment, which may reflect their relative maturity in the formation process. Materials perching on the drainage lines, particularly in the upper part of the drainage lines within the debris flow depletion zone, are mostly sizeable boulders left in place from previous landslides. These perched materials in the depletion zone are often completely, or almost completely, swept away in the event of a mobile debris flow. The debris flow resulted in deposition of debris including sizeable boulders at the accumulation zone in the lower part of the drainage line (Figure 32). After debris deposition, and upon removal of the fines following prolonged outwash and erosion, the sizeable boulders will become perched materials again. Hence, mobile debris flows, such as those that occurred in the June 2008 rainstorm, effectively result in 'pushing' the perched materials downstream. This would significantly alter the entrainment characteristics of the drainage line and may arguably help to reduce the risk of entrainment in future events. On the contrary, less mobile landslides and debris flows would result in accumulation of debris on the upper part of the drainage lines (Figures 5 and 33). It would increase entrainment potential, as well as the risk of future debris flows. This suggests that the scale and characteristics of debris flow hazards at a given drainage line may not be static, but could change with time as landslides and debris flows take place intermittently in the catchment. In terms of entrainment potential and characteristics, some debris flow catchments may evolve with time from one stage to another within a life cycle. The dynamic nature of drainage lines should be understood and judiciously accounted for in risk assessment and mitigation.

Figure 31: Complete removal of perched materials in the depletion zone of debris flow
(Note: The June 2008 Yu Tung Road debris flow is shown; the side slopes and bed of the drainage lines not subject to significant material loss)

(a) Depletion of perched boulders in upper part of drainage line

(b) Accumulation of loose boulders at lower part of drainage line

Figure 32: Transport of perched materials in a debris flow
(Note: The June 2008 Yi O debris flow is shown)

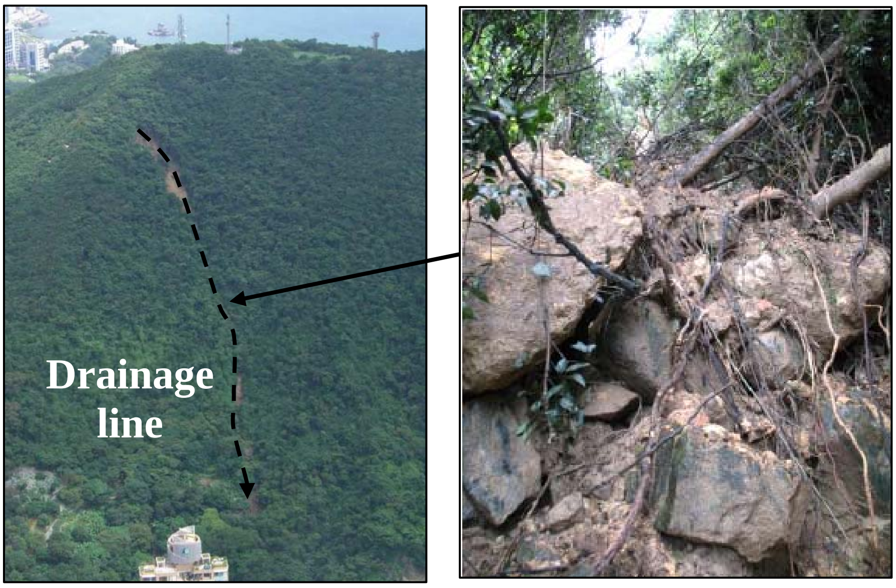


Figure 33: Less mobile landslide resulting in accumulation of loose debris on drainage line
(Note: The June 2008 Mount David debris flow above Police Quarters Block C is shown)

(e) **Deviation of debris flow path from drainage line**: A number of debris flows in the June 2008 rainstorm were found to have resulted in debris trails deviating from the drainage lines (Figure 34). As such, the debris flow did not entirely follow the drainage line, which is typically aligned with the direction of the steepest plane. This may be attributed to a number of reasons: (i) the failure at the landslide source may be subject to the control of geological structures that do not follow the direction of the steepest plane of the surface ground profile; (ii) the momentum of a fast-moving debris may cause the debris to move in the direction of the velocity vector and overshoot from the drainage line as it makes a sharp turn; (iii) run-up of a sizeable debris flow may result in overspill of debris over the watershed of the drainage line; (iv) when the debris flow hits an existing debris dam that blocks the drainage line, the flow direction may be deflected (Figure 35); (v) debris deposition in a debris flow may fill up local low points, change the topographical profile and alter the direction of the subsequent debris movement; and (vi) the existing topographical maps may not reliably represent the actual hillside topography. The above circumstances are occasionally encountered on site. It should not be taken for granted that debris flows would necessarily follow the alignment of the drainage line as indicated in the topographical map. Otherwise, the risk mitigation measures may be sited at a wrong location.

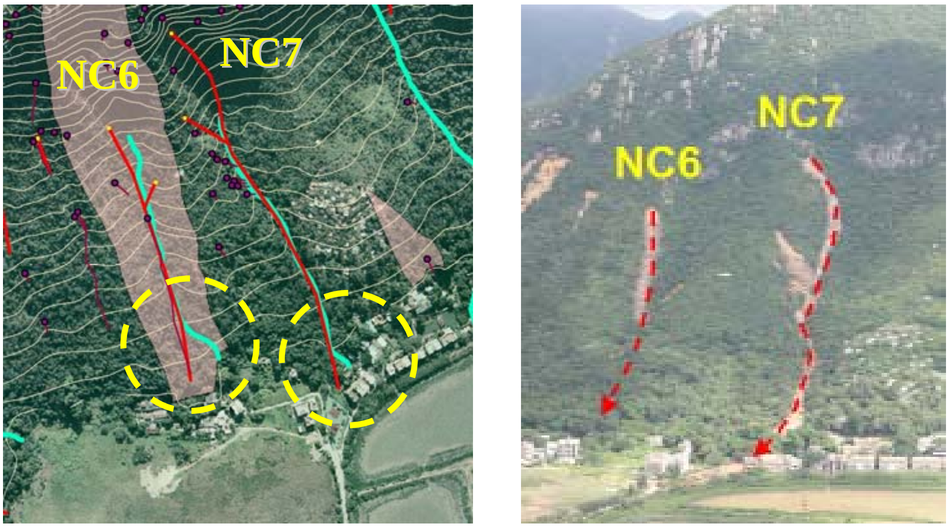


Figure 34: Examples of debris flow path deviating from drainage line
(Note: The June 2008 Nam Chung Tsuen debris flows No. NC6 & NC7 are shown)

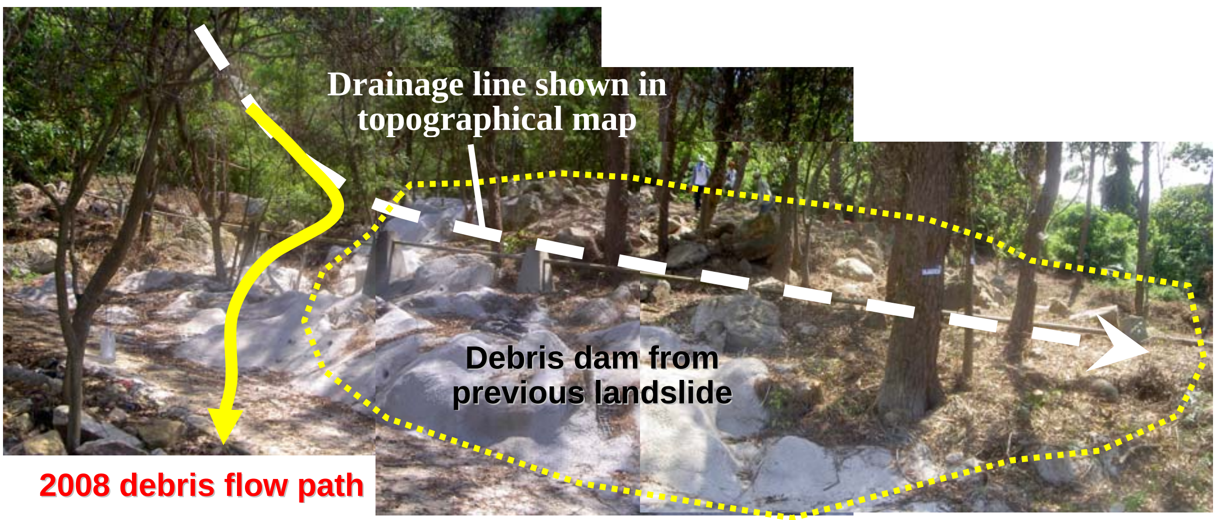


Figure 35: Debris flow diverted by an existing debris dam
(Note: The June 2008 Nan Chung Tsuen debris flow No. NC6 is shown)

4.3 Improving Understanding and Practice

In the light of the findings of presence of watery debris and their effects on the runout behaviour of debris flows, the importance of identifying the circumstances that may lead to watery debris and accounting for possible occurrence of watery debris in NTHS cannot be over-emphasised. In practice, there is a need to differentiate drainage lines that have the potential of discharging watery debris, from those where debris flows are relatively 'dry' as is commonly encountered in Hong Kong. Observations from field mapping suggest that the following settings might contribute to development of watery debris in a debris flow:

- debris flow at a major drainage line, i.e. with a large catchment and a long flow path, where a large amount of storm water may be available for mixing with the landslide debris (Figure 36)

- debris flow taking place during heavy rain, i.e. when the drainage line is full of running storm water

- debris flow with fast-moving debris, which overtakes the flow of the storm water in the drainage line and hence results in increasing water content during the debris runout

- debris flow along a main drainage line into which many tributaries of drainage lines are feeding, i.e. the water content of the debris will increase whenever the debris passes through a confluence point due to merging of storm water from the tributary onto the moving chain of debris

- other site settings, e.g. discharge of debris onto a pool of water on the drainage line, discharge of debris from a small drainage line onto a major drainage line or a catchwater channel where a large amount of storm water was running (Figure 37)

Figure 36: Major drainage lines fed by many tributaries
(Note: The June 2008 Shek Pik No. 4 debris flow is shown; the runout distance is about 1.8 km)

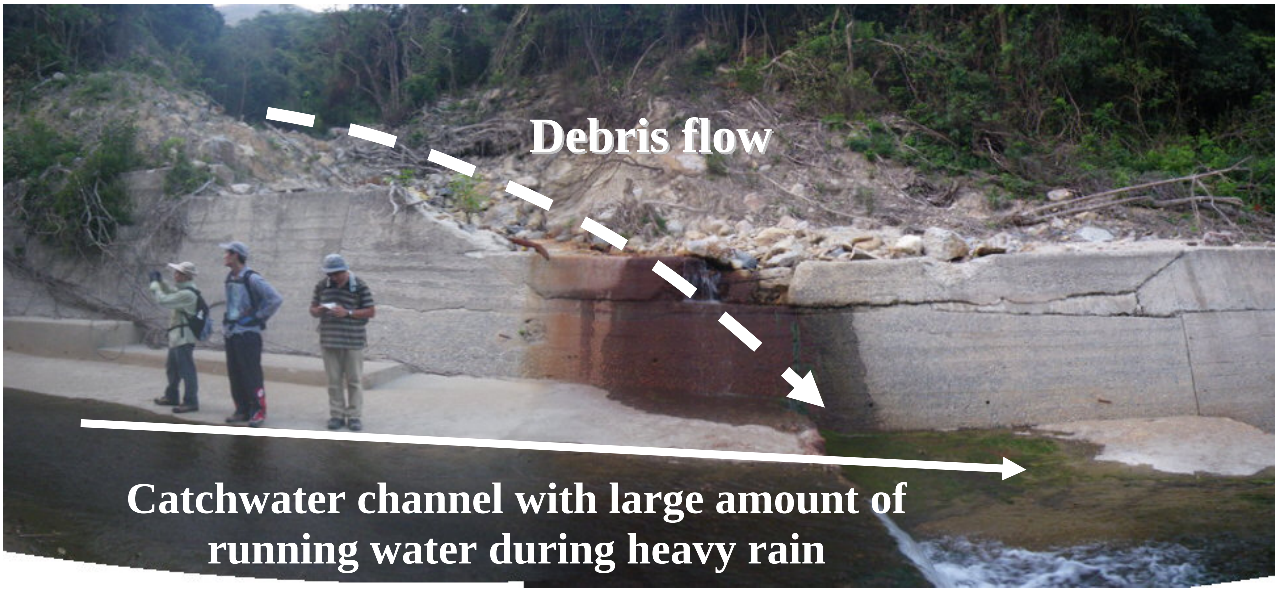


Figure 37: Debris flow entering catchwater channel
(Note: The June 2008 Shek Pik No. 1 debris flow is shown; the debris flow turned into a debris flood after entering the catchwater channel)

Apart from the possibility of discharging watery debris, the potential for major entrainment and the current life-cycle stage of a drainage line would deserve consideration in characterising debris flow catchments, for debris runout and risk assessment. From a preliminary review of the information available, it is noted that a higher degree of entrainment may be associated with the following factors:

- presence of steep terrain below the landslide source, thereby promoting acceleration of the debris after the initial detachment from the source

- presence of steep rock cliffs along the debris path, where debris velocity increases or free fall of debris may occur (Figure 38)

- debris with high rock/boulder content, which tends to involve considerable rolling and bouncing actions as the rock/boulders travel on a steep slope, particularly on a rocky channel bed

- debris flow in a drainage line with a large amount of perched boulders (Figure 39), which are susceptible to entrainment; this may coincide with a specific life-cycle stage of the drainage line, in which a large amount of entrainable materials have been stored up in the drainage line for eventual discharge downstream in a sizeable and mobile debris flow that is due to occur

- long runout debris flows with watery debris, in which the scale and mobility of the event provide sufficient energy to mobilize the entrainable materials over a long section of the drainage line

At present, knowledge of the entrainment potential and the possible characteristics and time frame of the life-cycle stages of drainage lines is rather limited. The GEO is exploring the practicality of, and strategy for, long-term monitoring of selected drainage lines, to improve the understanding of their behaviour and debris transport mechanisms.

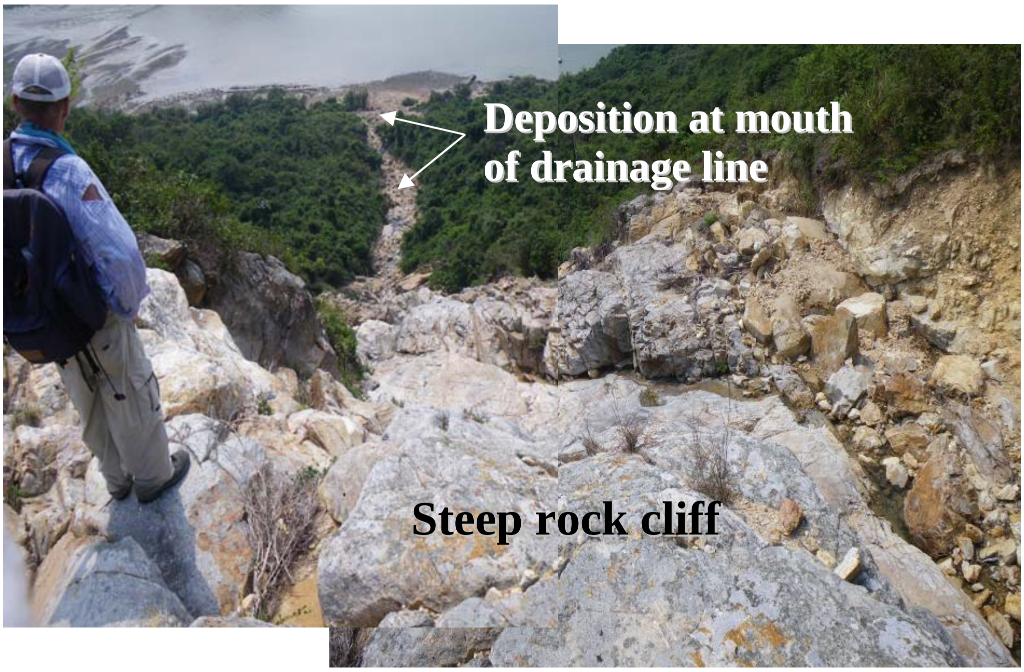


Figure 38: Steep rock cliff on drainage line
(Note: The June 2008 Yi O debris flow is shown; the debris including many sizeable boulders made their way to the mouth of the drainage with no deposition on the rock cliff)

(a) Adjoining Shek Pik No. 1

(b) Adjoining Shek Pik No. 2

(c) Adjoining 1990 Tsing Shan debris flow site

Figure 39: Drainage line filled with perched bouldery deposits that are susceptible to entrainament

In respect of risk mitigation, the appreciation of the possible need to deal with sizeable debris flows (e.g. several thousands cubic metres or more) and watery debris in some circumstances has already initiated discussion and a rethink of the strategy for debris flow risk mitigation. In particular, the need to explore alternative engineering solutions to the provision of a single debris-resisting barrier at the toe of the drainage line is noted (Figure 40). For instance, building a series of barriers along the drainage line may prove to be more effective in minimising entrainment and overcoming site constraints, while construction and related maintenance issues would require deliberation. Debris diversion channels, deflection structures and protective canopies, which are rarely used in Hong Kong in the past, may be suitable solutions for some cases. Attention has to be given to minimizing disruption to the environment and landscaping the risk mitigation measures, as the works will become more widespread in the years to come. Overall, there is considerable room for engineering innovation for improving the design, construction and maintenance of natural terrain landslide risk mitigation measures.

The observation that debris runout may not entirely follow the topographical steepest slope casts doubt on the reliability of subjective determination of the debris flow path in risk assessment and design of mitigation works. Three-dimensional debris runout and mobility modelling offers a technical means of objective assessment of the possible debris runout paths. In dealing with important or sensitive cases, 3-D modelling should be carried out to supplement subjective assessment of the debris flow path and 2-D mobility modelling.

At present, 3-D modelling algorithms are not widely accessible to practitioners and are not very user-friendly. This is an area for improvement. Furthermore, use of reliable 3-D digital terrain data is essential to 3-D debris runout and mobility modelling. Remote sensing techniques, in particular multi-return airborne Light Detection and Ranging (LiDAR), is promising in acquiring 3-D topography of terrain under vegetation cover in a cost-effective manner. The geotechnical profession should be better equipped with knowledge of remote sensing technology and experience in using remote sensing data, in addition to the conventional surveying methods, to prepare for undertaking natural terrain-related assignments.

(a) Single debris-resisting barrier at the toe of drainage line

(b) Series of debris-resisting barriers along the drainage line (Photo from Lo 2000)

Figure 40: Single vs multiple debris-resisting barriers

The above calls for continual efforts in studies and research of debris flows, as well as promulgation of knowledge among the geotechnical professionals. The geology and physics of debris flows are relatively new technical fields. Typically, they are seldom covered in depth in tertiary education and in professional training. In the past, hydrologists, geologists and engineers used to approach the subject from different perspectives and with different emphasis. Lack of cross-discipline synergy and collaborative efforts is not conducive to technological advances, particularly for a complex and challenging subject like this. There is a great demand for development of engineering geological expertise in mapping and assessing debris flow hazards. For example, recognising and logging different types of debris flow-related features and deposits are akin to identification and description of soil and rock in conventional engineering geological work. Assessing potential landslide sources, volume of failure and degree of entrainment may be comparable to diagnosing geological materials and structures that affect the design and performance of slopes, foundations, excavations, etc. Likewise, engineering geological and geomorphological models need to be established for assessment of natural terrain hazards, as in the case of development of ground and design models for geotechnical assessments (GEO 2007). In terms of the engineering aspects, improved knowledge of debris flow physics is needed (Iverson 1997). The geotechnical profession is generally unfamiliar with the mechanisms of debris flow mobilization, transport and deposition, as well as the physics of viscous fluid, solid-fluid interaction, concepts of kinetic sieving (Figure 41) and granular temperature, etc. These should be included in our professional toolbox, as we rise to the natural terrain challenge.

Figure 41: Large boulders and woody debris at the snout of debris flow
(Note: Distal end of the deposition zone of the June 2008 Shek Mun Kap debris flow at about 1 km from the landslide source is shown)

5 DEVELOPMENT OF RISK MANAGEMENT STRATEGY

5.1 Evolving Nature of Strategy

A detailed evaluation of the natural terrain landslide risk mitigation strategy is beyond the scope of this technical paper, and may be premature given that the planned natural terrain works under LPMitP are only at the early stage of launching. In formulating the post-2010 landslip prevention and mitigation strategy, the Administration has pledged to conduct a review of the progress and effectiveness of the LPMitP in 2015 (Development Bureau 2007). This offers an opportunity for a formal review and refinement of the natural terrain risk management strategy, which is necessary in view of the developing nature of our technical know-how and the new insights and experience to be gained from implementing the LPMitP initiatives.

While development of natural terrain landslide risk management strategy is an ongoing process and the review of LPMitP is scheduled to be carried out some years later, the knowledge available and lessons learnt to date suggest that certain aspects may warrant further thoughts in strategy development. A number of issues that may be of interest or concern to the profession are briefly described in the following sections.

5.2 React-to-Known-Hazard Principle

The current strategy for studies and mitigation of natural terrain landslide risk on existing developments is founded on the ‘react-to-known-hazard’ principle. This is a pragmatic strategy, given that: (i) catchments with historical failures are generally more active in landslide occurrence and hence deserve priority attention; (ii) the Government has a due diligence to act on known significant hazards; and (iii) reliable means of identification of other vulnerable catchments are not yet available. As a starting point of implementation of risk mitigation works, this principle appears to be well received. However, public expectation may shift with time following occurrence of major landslide incidents. Our capability in identifying vulnerable catchments may also improve with time. Hence, the need, practicality and resource implications of adopting a more proactive approach may be an issue for deliberation in future.

5.3 Urban Hillside Pockets

Apart from natural terrain located outside the present development boundaries, hillside pockets within developed areas are also subject to failures from time to time (Figure 42). By nature of their location, these hillside pockets may have been subject to different degrees of human disturbance. Their close proximity to developed facilities would also render them technically different to deal with, as compared with natural hillsides outside the development lines. Furthermore, there is potentially a link between the stability of these urban hillside pockets and other urban facilities, e.g. urban drainage, construction works and land-use. Their interaction may need to be addressed in a holistic manner, instead of being dealt with separately under different portfolios.

Figure 42: Landslide on hillside pocket within developed area
(Note: The June 2008 landslide above Kennedy Road is shown)

5.4 Output of Natural Terrain Risk Mitigation

The planned annual output of natural terrain risk mitigation works under the LPMitP is more than ten times of the current level. This represents a sharp increase in the amount of work, and its achievement is dependent on, among other factors, the availability of personnel with suitable geotechnical expertise. Given the constraints, further increase in the output is considered not practical in the early stage of implementing the LPMitP. However, increased public awareness of natural terrain landslide risk and expectation of slope safety may turn into demand for an increase in the output of natural terrain risk mitigation. The speed with which the geotechnical profession can build up the capability and capacity for undertaking natural terrain work may remain a constraint. The effectiveness in setting up an efficient and robust LPMitP process for delivery of natural terrain risk assessment and mitigation works is also a challenge for the GEO and practitioners to tackle in partnership. On the other hand, economic downturns and complacency that may typically arise following an uneventful period may result in pressure to cut resources and investment in long-term slope safety management.

5.5 Prescriptive Approach

Use of prescriptive measures has played a role in dealing with man-made slopes in Hong Kong. These measures are suitably conservative, experience-based modules of slope stabilization, protective and drainage provisions. They are applied in accordance with some established prescriptive design criteria and procedures, without the need for detailed ground investigation and analytical design (Wong et al. 1999). Over the past decade, prescriptive measures have been used under the LPM Programme, and as part of Government's enhanced slope maintenance initiatives in speeding up the improvement of the safety of a large stock of old man-made slopes.

In the case of natural terrain, prescriptive design may entail prescription of the volume of the landslide debris that may be discharged from a catchment, for use in the design of risk mitigation works. The prescription may be based on consideration of the characteristics of the catchment, its historical landslide activities and potential consequence of failure, under an empirical-based framework or expert judgement procedures. Prescriptive design may serve as an alternative to the prevailing approaches for determination of the Design Event through a detailed NTHS, which takes considerable time and resources, and thereby may help to fast-track the provision of risk mitigation measures at a large number of sites in order to maximise the rate of risk reduction.

Whilst this may offer a pragmatic option of speeding up natural terrain risk mitigation works, the adequacy of the prescriptive risk mitigation provisions at individual sites may have to be reviewed at a later stage via a detailed NTHS to confirm whether additional provisions are required to further control the risk. Doubtlessly, prescriptive design is subject to potential technical constraints, particularly due to the lack of detailed assessment and optimization of design. However, many other countries are commonly adopting approaches that are largely prescriptive in nature for dealing with natural terrain hazards. In Hong Kong, even if a detailed NTHS is carried out, expert judgement will often have to be exercised in the determination of the Design Event in the face of the uncertainties involved. Hence, it is arguable that some degree of prescription is always implicit in our prevailing assessment and design process. At present, use of prescriptive approach for dealing with natural terrain hazards in Hong Kong is only at a conception stage. Further technical development work is required for formulating a practical and robust prescriptive design methodology. The prescriptive approach may have a more explicit role to play in future, particularly if there is a demand for acceleration of risk mitigation works.

5.6 Risk Communication

Dealing with natural terrain landslides would inevitably involve considerable uncertainties. Risk management initiatives are aimed at minimizing risk to an ALARP level. Our technical knowledge and capability in tackling natural terrain hazards are still fairly limited. Some circumstances, such as climate change, are not entirely within our comprehension and control. While the geotechnical profession is aware of these constraints, the public and other stakeholders may not fully appreciate the nature of the problem. In addition, their risk tolerability and expectations are sensitive to other factors, e.g. an increase in risk perception after occurrence of major landslide incidents, particularly those involving multiple fatalities and major social disruption. Effective risk communication is vital to maintaining public awareness of the nature and reality of natural terrain landslide risk, rationalizing their risk perception, and gaining their support and participation in risk management. This in turn is crucial to maintaining a healthy and realistic pace of risk mitigation works, and to facilitating the further development and enhancement of risk management strategy.

6 CONCLUSIONS

A new era of natural terrain landslide risk management is dawning. This would not have become possible without the efforts and technological advances made over the years by the geotechnical profession. However, the challenges to face in taking the work forward must be viewed in the right perception. The profession has to get geared up to meet the challenges in discharging its enhanced responsibility.

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A New Flexible Barrier System for Hong Kong

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ABSTRACT

After a brief introduction to the increasing importance of mitigation measures to reduce risks from natural slopes, this paper introduces the Swiss Isostop barrier system which is being used in Hong Kong for landslide mitigation works. The main features of the system, in particular the ease of maintenance after an impact event due to the use of diagonal net (which has a cramp at every crossing to render the net intact even if a wire breaks), parallel cable and rope brakes (which provide unhindered extension under constant load), are discussed, The licensed system in Switzerland, together with the results of some full scale tests there are discussed. The results of a recent numerical analysis on the 3000 kJ system, carried out in Italy using a 3-D finite element computer program, are also presented and discussed. Finally, the first project in Hong Kong is briefly mentioned.

1 INTRODUCTION

With the establishment of the Geotechnical Control Office (GCO) in 1977 (then called the Geotechnical Engineering Office), landslip preventive measures (LPM) in Hong Kong have been dealing mostly with man made slopes in the past. With the effort of the authorities, the risk associated with man made slopes has greatly reduced. Attention is now being drawn to natural slopes (which by far out numbered man made slopes), partly because of developments getting closer and closer to natural slopes because of shortage of land. The recent rainstorm in June 2008 and the situation in Tai O indicated that risks from natural slopes are becoming more and more significant (RTHK 2008), and urgent actions are needed.

Engineering-wise it is not practical to stablise natural slopes, and the usual mitigation method is to provide barrier systems and traps to prevent the slide debris from reaching developed areas and causing damage. Although barrier system in Hong Kong can be traced back to the 1980s in the Mid-levels, these have not been common until recently. These used to be dominated by only one or two products, typically the Geobrugg system.

Recent attention to natural slopes opens up potential markets for other systems, and a new system from Switzerland, the Isostop system, has been selected. This paper discusses the main features of the system, including results of some recent research and development work. The first project in Hong Kong is also briefly mentioned.

2 THE ISOSTOP FLEXIBLE BARRIER SYSTEM

2.1 Background

The Isostop flexible barrier system is developed by Isofer ag (website: www.pfeifer-isofer.ch), a Swiss based company now owned by Pfeifer of Germany (website: www.pfeifer.de), which is a distributor and supplier of chains and wire ropes. The design energy of the Isostip system covers the range from 250 kJ to 3000 kJ. Products are generally manufactured to specification and requirements of the Swiss Agency for the Environment, Forests and Landscape (SAEFL) of the Swiss Federal Research Institute WSL. Product certification and license have been obtained. Its quality management system is also certified to ISO9001:2000.

2.2 Main features

The main features of system are given in Figure 1, consisting of the diagonal net restrained by cables fitted with rope brakes and anchored into pillars.

There are clamps at every crossing of the diagonal net (Plate 1) so that even if a wire breaks, the adjacent clamps will render the net intact. This will limit deformation of the net and allow the net to stand impact several times before replacement, which is also simple.

Specially-designed rope brakes (Plate 2) provide an unhindered extension of the loaded cable at a constant force (Figure 2). This feature allows large energy absorption with relatively little force on the anchors. The braking characteristics can be controlled by varying the parameters of the rope brake according to the energy requirement and the type of impact (rock fall or flow debris) to optimise the design. After an impact event, these rope brakes can be simply reset, or at worst replaced.

These rope brakes are attached to double parallel cables which provide restraining forces to the net during impact. The cables are attached to steel pillars which are anchored to the ground. If necessary, the based of these pillars may be fitted with shear bolts to allow rotation of the pillars, which would further dissipate energy. The anchoring of these pillars to the ground is similar to other systems, and depends on the ground conditions. However, owing to the efficient energy dissipation of the system, the anchor force of the Isostop system are generally smaller compared to other systems for the same design energy. Table 1 shows the anchor forces for various systems which are official test results from Walenstadt.

Regarding corrosion protection, system components, including threaded part of the rods, are generally galvanised to BN EN ISO 1461 at 80 gm/m^2. Zinc-aluminium galvanization can be provided on special request. Alarm systems integrating with the latest information technology can also be incorporated.

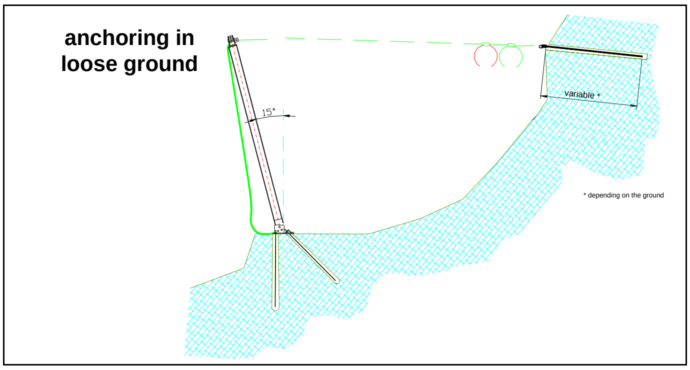


Figure 1: General arrangement of barrier system


Plate 1: Diagonal net with clamp


Plate 2: Specially-designed rope brakes

Table 1: Rope forces (kN) in various systems of Isofer (based on test results in Walenstadt)

Rope Location	Energy Class (kJ)				
	500	1000	1500	2000	3000
Retention Ropes	110	120	180	117	214
Upper Ropes	118	195	240	350	488
Lower Ropes	140	154	195	255	381

2.3 Research and development

Isofer has carried out over 160 full scale tests for research and development since it held its first test in 1985 (in Knonau). It now owns two test sites, one involving a slope cable car and the other a crane. Its products are also tested at the official test site in Walenstadt (Plate 3), licensed by the SAEFL Tests are carried out according to Gerber (2001), for product certification in the Swiss system

Plate 3: Testing Isostop 250 kJ system in Walenstadt

Full scale field tests are, however, expensive to perform, and measurements limited to a few essential components. On the other hand, suitable computer analysis can model the behaviour of the system in detail, and can be used for parametric study for design optimisation. Recently, Isofer has carried out a detailed 3-D finite element (FE) analysis on its 3000 kJ system (Isofer 2008), using the software [1]Ls-Dyna produced by Livermore Software Technology Corporation. [1]Ls-Dyna is a general purpose FE program particularly suitable for analysis of highly non-linear and high velocity dynamic problems of 3D structures. It makes use of the equations of motion and explicit time integration to obtain stress and deformation. It is commonly applied in automobile crash simulation and drop test simulation for durability of products. It has applications in the military sector and simulations of high velocity impacts in the aerospace sector. In real situation of the the debris-barrier interaction, the impact is completed in a few seconds, presenting a high non-linearity due to large displacements as well as non-linearity in geometry and material behaviour. A summary of the analysis and results are given below.

3 FINITE ELEMENT ANALYSIS

3.1 Model

The assumed geometry and loading are shown in Figure 2. The debris loading is simulated in 9 steps and the component characteristics are input into the model.

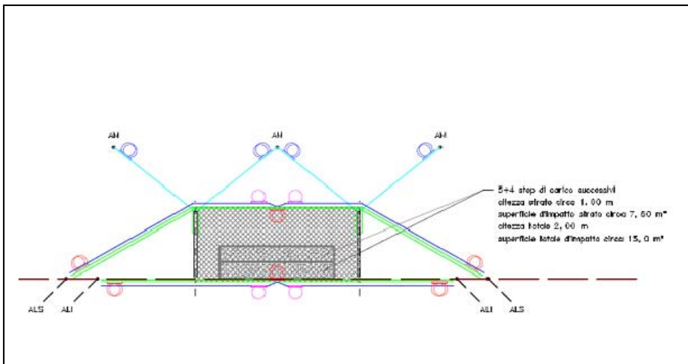


Figure 2a: Geometry and loading for FEA – Plan

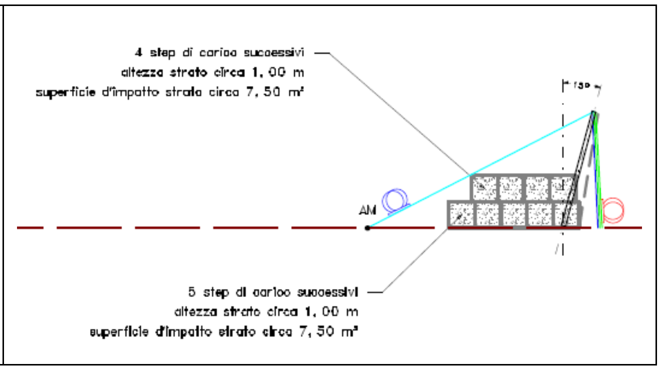


Figure 2b: Geometry and loading for FEA - Elevation

3.2 Main results

(a) Time

Duration of impact is 1.5 seconds. Maximum deformation occurs at 0.946 seconds.

(b) Deformation

Computed maximum deformation is 2.57m on the net (Figure 3).

Lowest residual height is 4.96m, or 88% of height retained (Figure 4)

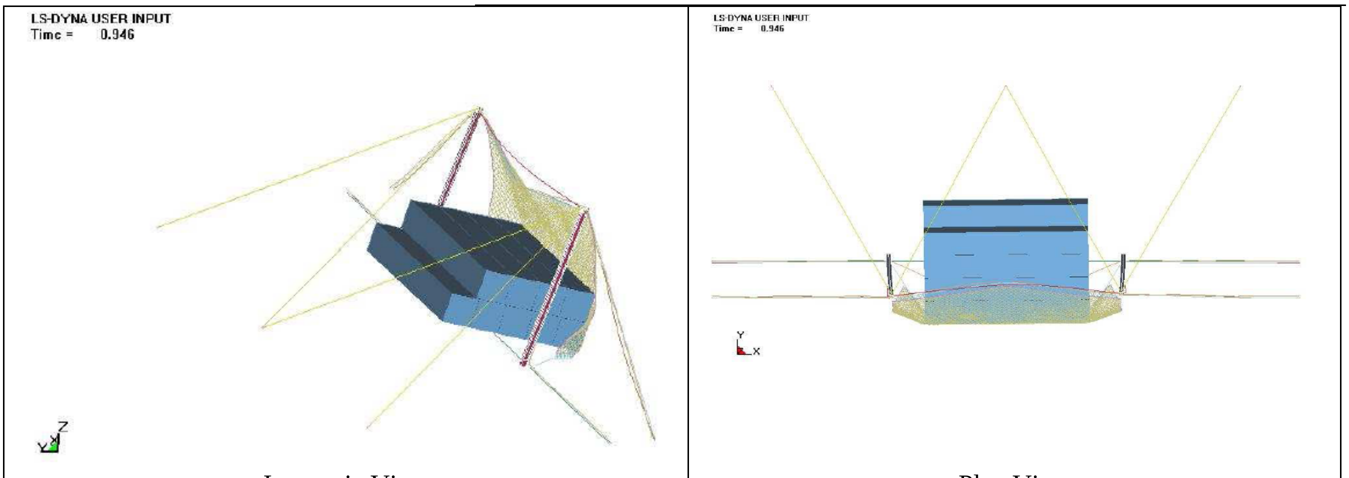
Deformation in rope brakes are shown in Table 2, and their locations shown in Figure 5.

(c) Load

Loading in all components are computed, down to the wire in the net. The maximum load in the main cables are shown in Table 3 and the variation of load with time shown in Figure 6.

Table 2: Maximum computed sliding distance of different brakes

Position Description			Sliding distance (m)
Group	Number	Position	
Main brakes	01	Lower lateral brake SX	1.05
	02	Central lower brake	0.00
	03	Lower lateral brake DX	0.94
	04	Upper lateral brake DX	0.00
	05	Central upper brake	0.00
	06	Upper lateral brake DX	0.00
Upstream Brakes	07	Upstream brake SX, pillar 1	0.28
	08	Upstream brake DX, pillar 1	0.17
	09	Upstream brake SX, pillar 2	0.11
	10	Upstream brake DX, pillar 2	0.20
Auxiliary brakes	11	Lower auxiliary brake SX	0.47
	12	Lower auxiliary brake DX	0.38
	13	Upper auxiliary brake SX	0.00
	14	Upper auxiliary brake DX	0.00



Isometric View

Plan View

Figure 3: Maximum deformation (2.57 m on the net at 0.946 seconds)

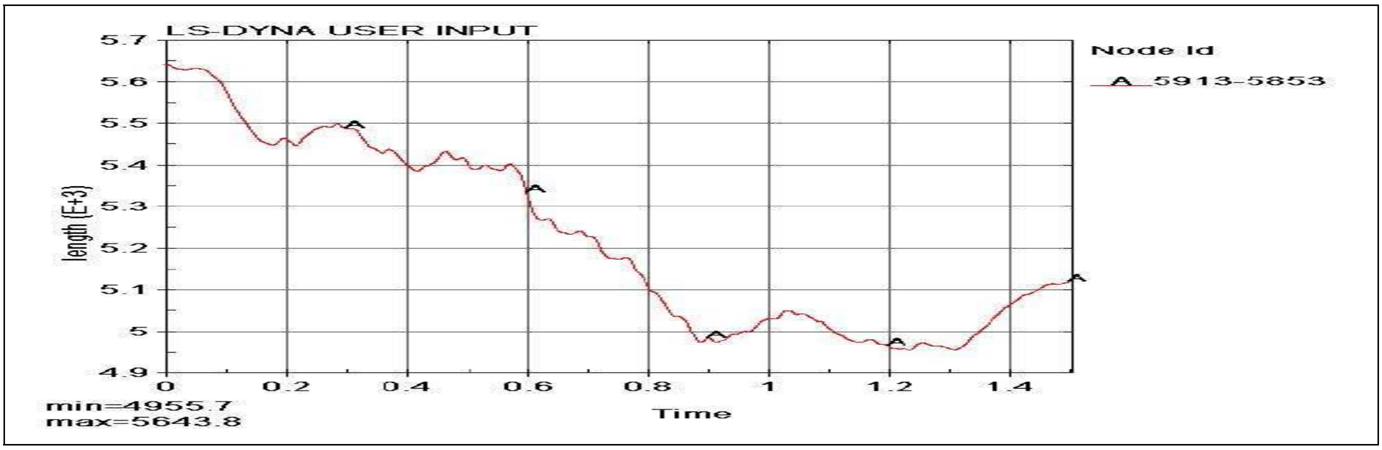


Figure 4: Residual height vs time

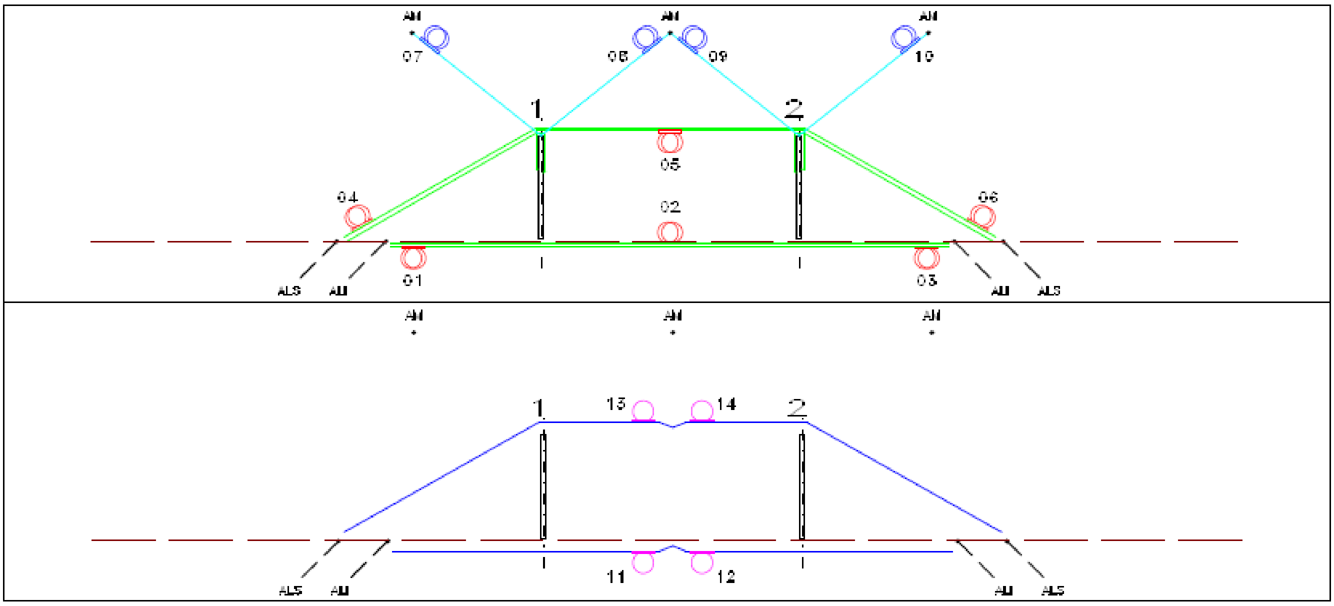


Figure 5: Locations of the rope brakes

Table 3: Maximum force computed at the upper and lower cable anchors

	Maximum Resultant Force
Cable	484 kN
Upper Cable Anchor	187 kN
Lower Cable Anchor	250 kN

4 FIRST PROJECT IN HONG KONG

A 3000 kJ debris flow flexible barrier will soon be installed for landslide mitigation work in Hong Kong. It has a total length of approximately 80m. The foundation works are in progress. The system is scheduled to be completed before May 2009.

5 CONCLUSIONS

The increasing importance of mitigation works on natural slopes leads to the increased use of barrier systems in Hong Kong. The Isostop barrier system from Switzerland is described in this paper together with the results of recent research and development work. The system has certain claimed advantages as described in this paper, in particular the ability to optimise design and the ease of maintenance after an impact event. This system will soon be installed in Hong Kong and local geotechnical engineers will be able to judge the performance of the system first hand then.

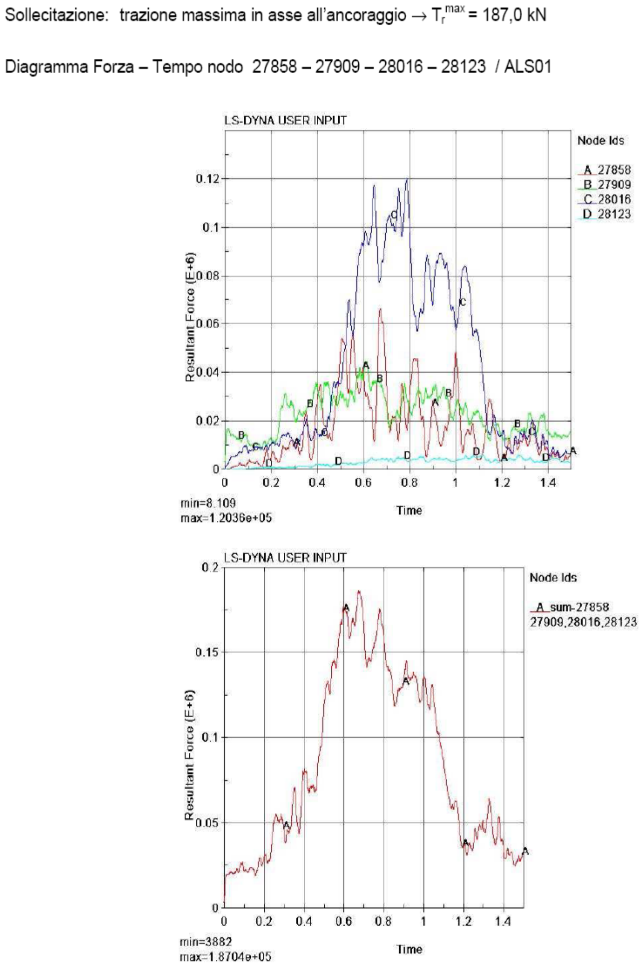


Figure 6: Computed forces at the upper cable anchor

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RTHK 2008. Television programme *Landslides in the Hong Kong Connection Series* dated 9 October 2008 (viewing available from the archive of the Radio Television Hong Kong website).

Reducing Uncertainty in Natural Terrain Hazard Studies: the Role of the Engineering Geologist

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ABSTRACT

Natural terrain hazard studies can appear deceptively simple. A limited Aerial Photograph Interpretation can be undertaken and a hazard model generated from this plus existing landslide datasets, such as the ENTLI. Geographical Information Systems can be used to interrogate the data and a series of hazard maps can be generated. However, without high quality engineering geological input, in particular field mapping, such an approach could underestimate the hazards affecting the site with potentially serious consequences. This paper illustrates how engineering geological input throughout the assessment process can reduce uncertainty with respect to critical issues such as hazard identification, magnitude, frequency and debris mobility.

1 INTRODUCTION

Unlike many regions in the world, where landslide assessments on natural slopes are carried out at a regional scale and relatively large degrees of uncertainty may be acceptable, the majority of Hong Kong's landslide assessments are of relatively small catchments or sub-catchments from which even a relatively small natural terrain landslide could impact on the site being assessed. As a result, any uncertainty could have potentially serious consequences and therefore assessments of a very high quality are necessary to ensure that all the hazards are identified and assessed.

2 NTHS vs. LPM

There are considerable differences between the assessment of foundations or man-made cut and fill slopes and the assessment of natural terrain for landslide hazards (Table 1). A Natural Terrain Hazard Study (NTHS) will result in limited "facts" and is instead dependent upon site observations, assessment and interpretation. A further key difference is that whilst "hard" engineering in the form of soil nailing may overcome limitations inherent in the LPM programme for man-made slopes, such a "safety net" may be neither economically nor environmentally suitable for widespread use on natural slopes.

Table 1: Comparison of LPM vs. NTHS

Man–made Slope Assessment	Natural Slope Assessment
Site of limited extent	Sites have a large extent, often comprising multiple catchments
Ground investigation (GI) stations are closely spaced	Limited scope for GI given large site and difficult access, means it is highly dependent on being located in critical areas
Exposures are available either before, during the GI, or during construction	Exposure limited to rock outcrop, landslide scars and drainage lines
Considerable amount of published data on geotechnical properties	Relatively limited data on the behaviour of natural landslides in Hong Kong
It may be appropriate to use simple classification of material types e.g. "colluvium"	Simple classifications are inappropriate. Classifying the superficial deposits requires an understanding of landscape evolution and geomorphological processes
Well developed software for slope stability analysis	Software programmes are not appropriate for catchment wide applications

3 DESIGN EVENT APPROACH

The most commonly adopted methodology for assessing natural terrain hazard in Hong Kong is the Design Event approach (Ng et al. 2003). These government guidelines present a framework for assessment based on notional susceptibility of the terrain to landsliding, the type of facility, and the steepness of the terrain. Based on this either a "conservative event" (generally corresponding to a reasonably conservative estimate based on the observable failures in the last 50-100 years, i.e. within the available aerial photograph record for Hong Kong) or a "worst credible event" (WCE) (generally corresponding to the largest credible event based on API interpretation of landslides as well as interpretation of the geomorphological characteristics of the site) is adopted for design purposes. The WCE is nominally taken as being approximate to a 1 in 1000 year event, with the view that "this is intended to exclude historical events that occurred in the geological past" (Ng et al. 2003).

The Design Event approach was developed to enable a rapid evaluation of the possible magnitudes of hazards a site may face and therefore allows potential cost implications and alternative layouts to be considered at the feasibility stage. In applying the Design Event Approach, particularly at a feasibility stage, conservative assumptions may have to be made to ensure that the design requirements are not underestimated due to the lack of information. However, these assumptions should be reviewed and modified as the study progresses as assessments are very sensitive to changes in potential volume, distance between source and facility, rates of deposition and entrainment, etc. The determination of a design event i.e. a particular magnitude hazard with a certain mobility which is used for the design of the mitigation works therefore requires sound engineering geological judgement.

4 KEY UNCERTAINTIES IN NTHS REQUIRING ENGINEERING GEOLOGICAL INPUT

The four key areas in a landside hazard assessment requiring engineering geological input are:

- the identification of the various hazards;
- determination of an appropriate volume for each identified hazard, including potential entrainment where applicable i.e. magnitude of the hazard;
- an indication of the frequency of such hazards; and
- the likelihood of such hazards affecting the facilities in question, i.e. an assessment of mobility.

The data required to assess the 4 key areas will become available at different stages of the NTHS and should reduce in uncertainty as the study progresses. The derivation of the design event using engineering geological input is illustrated in Figure1 and, in this respect, the authors consider engineering geological mapping to be the critical area for reducing uncertainty.

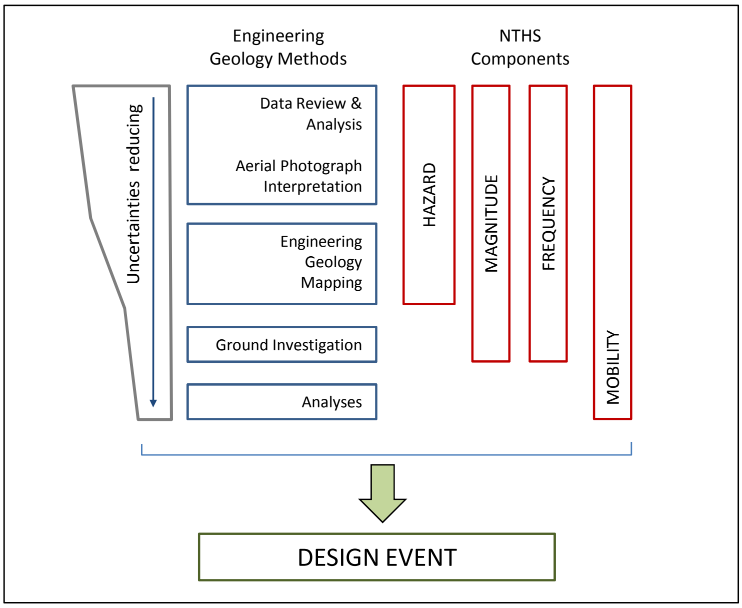

Figure 1: Engineering geological input within an NTHS

4.1 Hazard identification

In Hong Kong natural terrain hazards have been grouped into five main hazard types on the basis of transportation, the nature of the displaced material and the topographic location (Ng et al. 2003). These are: Open Hillslope Landslides (OHL); Channelised Debris Flows (CDF); Deep-seated Slide; Rock Fall; and, Boulder Fall.

As a consequence landslides are commonly classified as OHL or CDF. However, all landslides identified should be classified in accordance with their failure mechanism (e.g. Cruden & Varnes 1996) as these commonly reflect the geomorphological and geological setting.

The primary method of hazard identification is Aerial Photograph Interpretation (API). However, API has limitations which are well documented both overseas (Fookes et al. 1991; Hart et al. 2009) and in Hong Kong (Parry et al. 2006). As Fookes et al. (1991) note "all aerial photographic interpretation must be viewed with caution. When well done and confirmed by ground truthing exercises it can be a very valuable tool on any site. However, it should be borne in mind that it is an interpretation and hence subject to the interpreters' training, experience, skill and judgement as well as other factors such as the availability of additional information, type, scale and quality of the photographs, time available and the extent, if any of ground checking work". Despite these limitations, in Hong Kong there is often a misconception that aerial photographs display "facts" which can be extracted by staff inexperienced in geomorphological/geological mapping.

The identification of relict landslides require very careful judgement given that landslide scars progressively lose their morphological definition at rates that are dependent on factors such as vegetation, landslide size, material characteristics and location (Parry & Hart, in press). The evidence for associated landslide debris is rarely evident due to erosion or vegetation growth but maybe reflected in the presence of debris lobes. Identification of recent landslides where complete detachment does not occur (refer to the example in Figure 2a) from API is also problematic and these require careful field mapping. There is also a tendency to overestimate recent landslide run out from API due to post-landslide, fluvial re-working of the landslide debris and again field verification of such events is critical.

Alternative remote sensing approaches have been examined in Hong Kong but have not been widely adopted to date due to the excellent quality and coverage of the available aerial photographs. However, a trial of airborne LIDAR has recently been undertaken and this dataset is proving to be valuable especially in areas where dense vegetation is present in the aerial photographs. However, the processing of LIDAR data is not straightforward and specialist input is required to ensure that techniques such as DEM gridding and shaded relief maximise the relevant features, as well as ensuring that the data is not misinterpreted (Parry & Jonas 2007). That said, LIDAR has its limitations (refer to Figure 2a) like any remote sensing technique and at the site specific scale, field mapping is essential.

Landslides are extremely complicated processes, which can be associated with a variety of depositional environments. Unfortunately there is a tendency in Hong Kong to categorise all deposits associated with mass wasting processes as "colluvium", which can lead to considerable over estimations as to the size of previous landslide events. For example, detailed engineering geological field mapping located a debris lobe associated with a recorded 1966 landslide within dense vegetation. However, given the limited exposure in the field, trial pits were excavated to assess the depth of the debris. These revealed evidence of previous mass movements prior to 1966 (Figure 2b). The horizon beneath the 1966 debris comprised sand and well sorted, sub-rounded gravel with a well developed imbrication fabric, suggesting that it was deposited by fluvially dominated processes, possibly during a debris flood. The lowest layer comprised cobbles and boulders suggesting a relatively high energy landslide event. This layer is underlain by dense structureless colluvium. Each layer exhibited a topsoil horizon and the stratigraphy suggests that landslide events in this drainage line may have been decreasing in energy, and changing from debris flow to debris floods, with time.

Engineering geological mapping carried out by experienced personnel is necessary to evaluate landscape evolution, form, processes and materials, all of which are critical to identify, characterise, predict and mitigate landslide hazards.

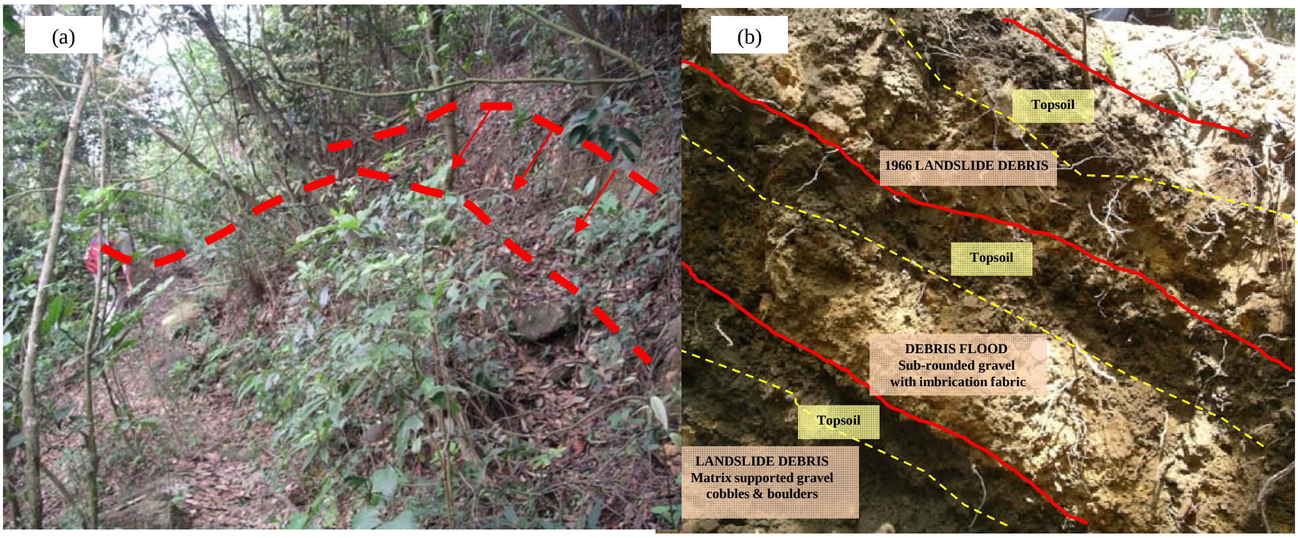

Figure 2: Identification and evaluation of landslide events during field mapping and ground investigation; (a) indicates extensive tension crack and minor displacement associated with an area of distressed ground which was not identified from API or LIDAR but only during field mapping; (b) differing landslide types and ages interpreted from a trial pit in a debris lobe identified from field mapping

4.2 Magnitude

The GEO have developed detailed territory-wide landslide inventories since the mid-1990s. The latest Enhanced Natural Terrain Landslide Inventory or ENTLI (Maunsell Fugro Joint Venture 2007) is a considerable improvement over the original NTLI, in particular the identification of "relict" landslides which are now based on the high quality territory-wide, low-level 1963 aerial photographs.

However, there are still limitations to this improved inventory. For example, whilst guidelines were produced to improve consistency with respect to the interpretation of relict landslides (Parry et al. 2006), the interpreters' classification varied with experience. Furthermore, given the project constraints (over 105,000 aerial photographs were reviewed in 16 months) the interpreters could only examine for relatively clear evidence of landslides e.g. obvious scarps, with very little time available to consider geomorphological settings, often critical with respect to the identification of older degraded and possibly landslide related features.

An example of the limitations with respect to degraded features is illustrated in Figure 3, which indicates the positions of landslides within the existing databases and an additional large landslide identified during the site specific API. The identified feature is considerably larger than any landslide within the existing database. Based on the field mapping it was interpreted that many of the lobate landforms identified by API were of differing origins. However, the morphology and materials of one lobe indicate that this could have been deposited as a single large rock avalanche.

Furthermore, when considering the possibility of high magnitude, low frequency landslide events, it is not only the initial landslide volume but the potential for both multiple source volumes and/or significant material entrainment that also have to be considered. Whilst in Hong Kong the most recent channelised debris flows have not involved significant entrainment, this could occur with the right combination of geological and geomorphological conditions. For example, the 1990 Tsing Shan debris flow comprised an initial landslide of 450 m^3, which induced a second landslide of 2500 m^3. The debris subsequently accelerated over a cliff resulting in extensive entrainment of the materials below with the final volume estimated at 19,000 m^3 (King 1999). Section 4 of this paper gives an example of the evaluation of such a scenario.

There is concern that the existing landslide inventories, which have been developed without field verification, may be used mechanically and without appreciation of their compilation methodology and limitations, and that critically, degraded landforms, that may represent high magnitude, low frequency and that are not identified in the datasets will be overlooked.

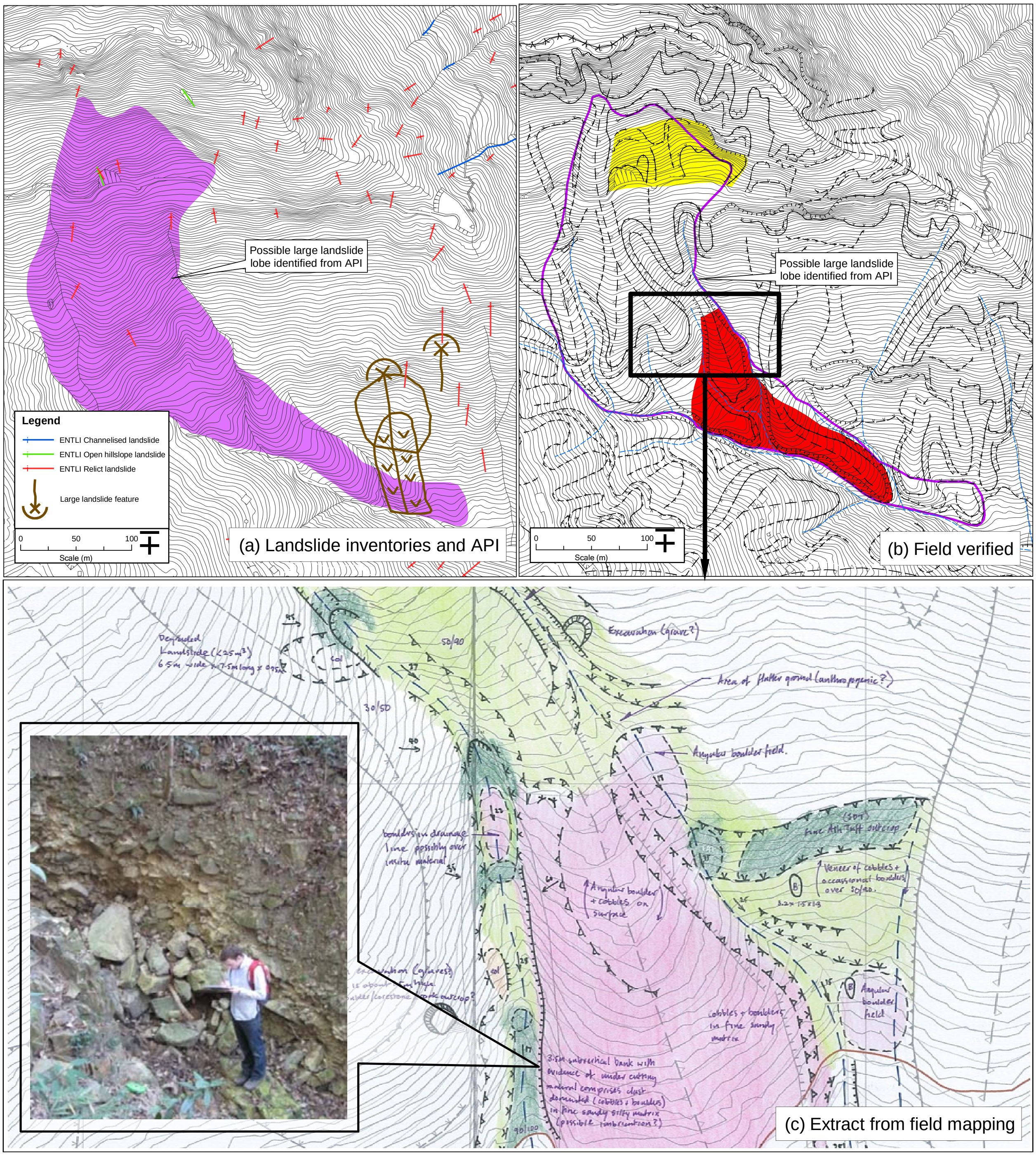

Figure 3: (a) Landslides identified in the various inventories and an additional possible large degraded landslide identified from site specific API. (b) Engineering geological mapping at 1:500-scale indicated that the lobate landforms identified from API have separate origins. For example, the feature identified in red consists of a distinct deposit comprising angular to sub-angular, slightly to moderately decomposed, clast-supported cobbles and boulders. A depression (yellow) is evident above this lobe. The field evidence suggests that the lobe may represent debris from a large rock avalanche

4.3 Frequency

The Design Event Approach is often over simplified to assume that a "conservative event" is equivalent to the largest landslides observed within the time period of the available photographic record and a "worse credible event" is equivalent to the largest morphological depression in the landscape. The lack of absolute age dates for landslides in Hong Kong makes the assignment of ages extremely problematic. In a limited age dating study of landslides in Hong Kong (Sewell et al. 2006) the upper bound age for the features dated was 34,000 years BP. As a result, determining whether a landslide occurred less than 1000 years BP (i.e. it is a WCE) or whether it occurred "in the geological past" is extremely challenging.

There has been great progress in the use of age dating techniques for the dating of landscape evolution (Sewell et al. 2006). However, the application of these techniques depends on the landscape evolution at a site and the processes involved being understood so that the samples selected are truly representative of the hazards being dated, i.e. any dating must be within the constraints of a geological model. For example C^{14} dating of the buried soil horizons shown in Figure 1(b) could have been underatken to provide an absolute age constraint for each landslide event.

Even without absolute age dating it is possible to provide relative age dating of landforms. Figure 4 shows an API which, in addition to identifying debris fans associated with channelised debris flows (levees and individual debris lobes are evident) also provides relative ages of the fans given their juxtaposition and morphology. Again such observations are critical in understanding hazard type, magnitude, frequency and mobility.

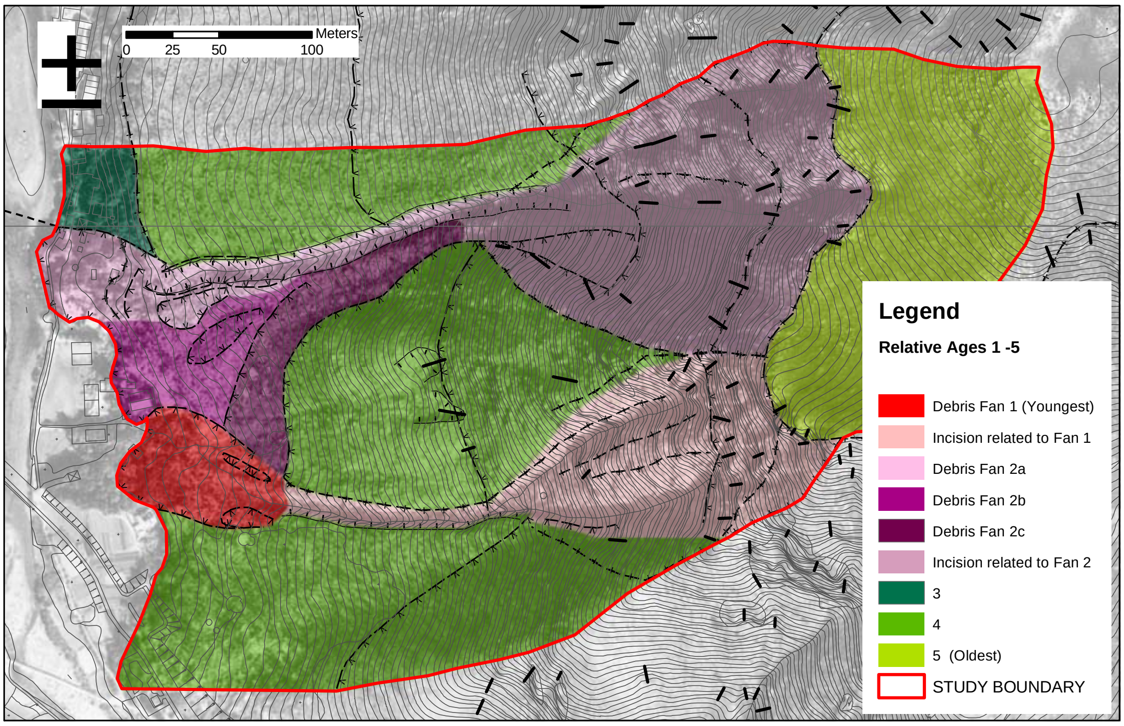

Figure 4: Relative ages of terrain units determined from API. Note that Debris Fan 2 has three distinct deposits, the ages of which are based on the sharpness of features identified. Debris Fan 1 is younger than Debris Fan 2b and 2c. The debris fans are being generated by channelised debris flows related to the incising drainage lines. Note that there are no recent landslides noted in the ENTLI (black lines). This figure also illustrates the importance of geomorphological setting with respect to landslide initiation and potential mobility

4.4 Mobility

Having determined the potential magnitude of the design event to be adopted, mobility analyses can be undertaken to estimate the volume and velocity of the potential debris reaching the facility at risk. This can be either empirical or, more increasingly, analytical. The main contributions of engineering geology to mobility include:

- providing site-specific data with respect to drainage line characteristics, in particular the most applicable channel geometry to adopt for channelised debris flows. Where drainage lines have incised into older and larger channel forms, the correct geometry to adopt is dependent upon the magnitude of the landslide being analysed ;

- determining the possibility of secondary induced failure and general entrainability of the substrate; and
- examining evidence from historical landslides, in particular recording field evidence for features such as debris height and super elevation (Figure 3), for mobility back analyses.

All of these factors are dependent upon careful field observations. For example, the failure to distinguish between run-out distances of remoulded landslide debris and outwash material could result in significant errors in the back analysis of the landslide event.

The contour maps generated from the LIDAR are a considerable improvement on the existing 1:1000 topographical maps, with respect to evaluating run out. However, relatively small scale topographical variations which can affect the mobility analysis may not be evident from LIDAR (Figure 5).

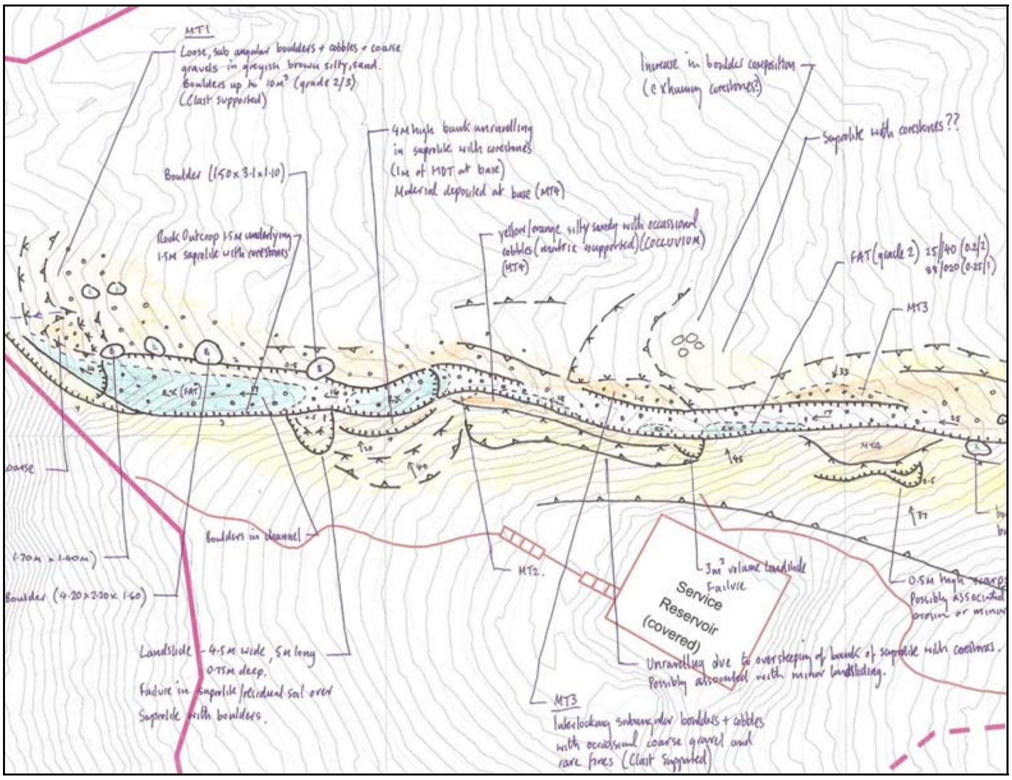

Figure 5: Engineering Geological mapping at 1:500-scale on LIDAR generated contours. The mapping identifies an incised drainage line with vertical banks up to 4m in height which are not evident from LIDAR. Such information is critical for mobility modelling. Also shown is fluvial incision resulting in over steep terrain and evidence of instability. The initial hazard models generated from API and existing data review should be re-evaluated based on such field observations

5 DERIVATION OF A DESIGN EVENT

The derivation of a design event is an iterative process. Figure 6 shows the development of an initial design event for a hillside in the New Territories, predominately based on API (Parry & Ng, in press). The hillside of concern rises from 90mPD to 295mPD and comprises two catchments with a total area of about 56,000m^2. Figure 6a is an engineering geological map from API which shows considerable areas of rock and intermittent rock outcrop with saprolite on the spurlines and thicker saprolite in the upper terrain. Superficial deposits, predominantly comprising taluvium, are associated with the drainage lines. Figure 6a also shows the location of ENTLI features which are predominantly located in the incising terrain. Figure 6b shows an initial hazard model for the site focusing on the likely magnitude of landslides. Based on the Design Event approach a "worse credible event" (WCE) is applicable. In order to generate a WCE, the largest section of steep terrain adjacent to the boundary of the Upper Terrain unit was selected, whereby a source landslide would result in the debris impacting upon the greatest extent of taluvium within the Incising Terrain unit below. Where the taluvium is considered to be underlain by rock or intermittent rock outcrop (typically steeper terrain), it has been assumed that all the taluvium over the width of the source landslide would mobilise under the impact of the debris. The thickness of taluvium was conservatively assigned as 4m at this time. Such a WCE corresponds to an initial landslide source volume of 1500m^3 in the Upper Terrain saprolite combining with a secondary failure of 1400m^3 in the taluvium, resulting in a total volume of 2900m^3.

Obviously there are uncertainties associated with such preliminary models e.g. the potential source volumes, amount of entrainment etc, but critically such models provide focus for the subsequent engineering geological mapping and ground investigation. They also allow a preliminary evaluation of the types of mitigation works required and allow factors such as cost and environmental impact to be considered.

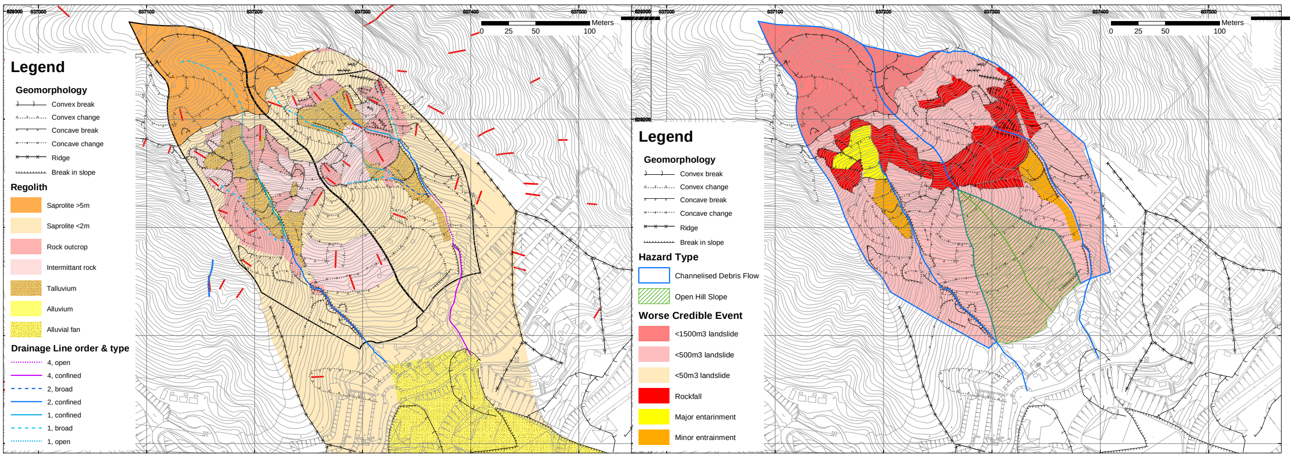

Figure 6: Initial Design Event derivation. (a) Engineering Geological map. (b) Initial Hazard Model used to generate the Design Event at review stage. Both are based on API and should be re-evaluated during field mapping

6 CONCLUSIONS

Natural terrain hazard studies can appear deceptively simple. A limited Aerial Photograph Interpretation can be undertaken and a hazard model generated from this plus existing landslide datasets, such as the ENTLI. Geographical Information Systems can be used to interrogate the data and a series of hazard maps can be generated. However, without high quality engineering geological input, in particular field mapping, such an approach could underestimate the hazards affecting the site with potentially serious consequences. An engineering geological model, focused on potential hazards, and developed from geological and geomorphological principals and with knowledge of the terrain being evaluated, provides the key to understanding the geological controls and geomorphological processes that have affected the site in the past and may affect it in the future. Such models allow “what if” scenarios to be generated and evaluated and enable areas of uncertainty to be recorded, investigated and reduced. Models should be constantly questioned and refined as additional data becomes available; particularly important are the observations from the field mapping. Once an appropriate model has been developed it can be analysed in terms of run out and impact. Such analysis must be grounded within the site specific observations. Only with continuous engineering geological input can robust and defensible mitigation works be constructed.

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The Enhanced Natural Terrain Landslide Inventory

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ABSTRACT

The Enhanced Natural Terrain Landslide Inventory (ENTLI) consists of a comprehensive record of historical natural terrain landslides in Hong Kong. Maunsell Fugro Joint Venture (MFJV) was commissioned by the Geotechnical Engineering Office to compile the Inventory.

The ENTLI has been compiled from interpretation of a comprehensive set of aerial photographs taken between 1924 and 2003. The GIS-based inventory is a valuable source of information to geotechnical practitioners for undertaking natural terrain landslide and slope studies in Hong Kong, in assessing the susceptibility of hillsides to failure and potential risks posed by natural terrain landslides to the community.

This paper outlines the approach and methodology adopted in compilation of the ENTLI; presents briefly the findings of the work; and, provides discussion on the limitations of the data.

1 INTRODUCTION

The Enhanced Natural Terrain Landslide Inventory (ENTLI) is a comprehensive record of historical natural terrain landslides in Hong Kong. The inventory was compiled from interpretation of available low (<8,000feet) and high-flight (≥8,000 feet) aerial photographs and was presented on a 1:5,000-scale mapsheet basis. The locations of all identified landslides and associated data were recorded directly into a geographical information system (GIS) for ease of input and subsequent analysis. A report was produced for each 1:5,000-scale mapsheet together with a summary table showing the basic data on each landslide and a list of aerial photographs used for interpretation.

2 BACKGROUND

In the mid-1990s, the Geotechnical Engineering Office of the then Civil Engineering and Development Department of the Hong Kong Government (hereafter referred to as GEO) commenced the first phase of the Natural Terrain Landslide Study with the creation of an inventory of historical landslides occurring on natural terrain in Hong Kong (known as the Natural Terrain Landslide Inventory or NTLI), based on aerial photograph interpretation (API) using high-flight aerial photographs (≥8,000 feet) taken in 1945 to 1994. The compilation methodology for the NTLI is reported in King (1999).

Since 1998, updates of the NTLI were undertaken as part of various natural terrain landslide studies: 1995 to 1997 (Scott Wilson 1999), 1998 to 2000 (MFJV 2003a), and 2001 to 2003 by both Maunsell Geotechnical Services Ltd. (MGSL) and a Fugro Scott-Wilson Joint Venture (FSW). Up to 2003, the NTLI contains about 30,000 features. The data has been widely used in natural terrain-related studies to assess susceptibility of hillsides to failure and potential risks posed by natural terrain landslides.

The limitations of the NTLI are broadly discussed in King (1999) and discrepancies between landslide findings from site-specific natural terrain studies and those features contained within the NTLI have been discussed by others, e.g. Parry (2001), Pinches and Smallwood (2000), Pinches et al. (2002) and MFJV (2003b). In particular, the high-flight API, on the basis of which the NTLI was compiled, has a limited resolution, with many smaller natural terrain landslides or those occurring some time before the aerial photographs were taken, being difficult to identify. Identification of such landslides using low-flight aerial photographs was considered necessary to provide additional information and on a comprehensive basis, has been instrumental in the study of natural terrain landslide hazards and in detection of hillslope catchments with historical landslide activities potentially posing a risk to the community.

In early 2004, it was decided to establish an updated natural terrain landslide catalogue for Hong Kong, using both high and low-flight aerial photographs (ranging from 1,800 to 20,000 feet flight-height), in addition to years of coverage excluded from the NTLI. To develop a methodology and to evaluate required resources, a Pilot Study for 28, 1:5,000-scale mapsheets covering Hong Kong Island, Kowloon and Sha Tin foothills was undertaken by MGSL and FSW under two separate consultancy agreements. In July 2005, Maunsell Geotechnical Services Limited and Fugro (Hong Kong) Limited Joint Venture (MFJV) were commissioned to compile the catalogue for the remaining 155, 1:5,000-scale mapsheets.

The main ENTLI study was finished in March 2007. Validation of both the API mapping and the relevant digital data continued until July 2007. Updating of the ENTLI for 2004 to 2006 has subsequently also been undertaken by MGSL and FSW in 2008, under two separate consultancy agreements.

3 COMPILATION METHODOLOGY

The ENTLI compilation methodology was developed on the basis of the NTLI compilation methodology reported in GEO Report No. 74 and the GIS data-generation procedures adopted as part of the NTLI update for year 1998 to 2000 (MFJV, 2003a), with the following modifications and refinements.

The first key difference from the original NTLI methodology is that a much greater coverage of aerial photographs has been used during preparation of the ENTLI than in the NTLI (approximately 20,000 aerial photographs were reviewed during compilation of the NTLI as compared to approximately 105,000 reviewed as part of ENTLI compilation). The second is the greater detail in the treatment of relict landslides (see Section 3.1.2 below).

The third major difference is the interpretation of what construes ‘development’ and ‘natural terrain’. In the previous NTLI study and updates, each sequential period of aerial photographic coverage was used to generate and update a plan showing development lines, separating natural terrain from development. As described in GEO Report No. 74, natural terrain was defined as “*Terrain that has not been modified substantially by human activity but including areas where grazing, hill fires and deforestation may have occurred.*” Interpretation for the NTLI was restricted to the area defined as natural terrain.

Under the present ENTLI compilation procedure, the interpretation was not restricted to a tight definition of natural terrain as constrained by a development line. Instead, the interpreter was required to evaluate the origin and nature of a targeted feature with respect to the aerial photographs reviewed, evaluating if the feature was clearly a man-made slope feature (e.g. cut or fill slope failure), or a natural terrain feature.

The ENTLI compilation methodology comprises two major components: Firstly, the interpretation of recent and relict landslides from available aerial photographs and secondly, the creation of the associated landslide dataset. Existing features in the NTLI (up to year 2003) were checked and validated, with positional amendment and deletion made where appropriate. Additional landslides interpreted and not previously recorded by the NTLI were then digitised, with the complete catalogue compiled using GIS.

3.1 Definition of recent and relict landslides

During compilation of the ENTLI, landslides were classified into two groups: recent if they occurred within the time scale of the available aerial photographs and relict if they occurred earlier.

3.1.1 Recent landslide classification

Recent landslides are defined as those features that clearly occurred within the time scale of the available aerial photographs. GEO Report No. 74 elaborated that “*The scars of recent landslides have a distinctive light tone on aerial photographs and are generally bare of vegetation, being in vegetation cover Classes A or B* [completely or partially bare of vegetation]. *The time period in which they occurred can be confirmed by reference to earlier aerial photography"*. The ENTLI definition of recent landslides follows that used by the NTLI study.

Recent landslides recorded by the ENTLI were classified into three principal types: open hillslope, channelised or coastal.

- Open hillslope landslides are those where landslide debris could be observed extending directly downslope, with no evidence of deviation or redirection caused by localised channelisation along topographic depressions.
- Channelised landslides are those where the landslide debris could be observed to deviate from a vertical trajectory due to the influence of localised topographic depressions (e.g. stream courses).
- Coastal landslides are those where the landslide was considered to have been caused by under-cutting from wave erosion. Where the landslide source was adjacent to the coast but did not intersect the shoreline or was protected from wave erosion (e.g. at the crest of a rock cliff), it was not classified as a coastal landslide.

3.1.2 Relict landslide classification

Relict landslides are those that occurred earlier than the time scale of the available aerial photographs. GEO Report No. 74 stated that "*Relict landslides are covered in grass, shrubs or trees (vegetation classes C or D)* [completely covered by grasses or by shrubs and trees] *but the ground still shows some clear characteristics of a landslide scar. They were mapped when a spoon-shaped depression with a sharp main and/or lateral scarps was either visible or could be reliably inferred from vegetation characteristics.*"

Under the ENTLI study, the relict landslide definition was modified to provide a more comprehensive record of relict features than the NTLI and to include an expression of the relative certainty behind the interpretation. ENTLI relict landslides were further classified as A, B or C-Class features with an indicative probability of interpretation of 80%, 50% and 10% respectively, together with additional S-Class records for coastal landslide features caused directly by under-cutting from wave erosion (Figure 1).

3.2 Aerial photograph interpretation procedures

3.2.1 Aerial photographs used

Recent landslides were identified from sequential interpretation of 1924 to 2003 aerial photographs. These included both high-flight and low-flight photographs taken at 1,500 to 20,000 feet, giving nominal scales of 1:3,000 to 1:40,000.

Relict landslides were principally identified by interpretation of 1963 aerial photographs, in order to facilitate a consistent baseline review process. The 1963 aerial photographs comprise the earliest high-resolution, HKSAR-wide aerial photographic coverage taken at 2,500 to 8,000 feet. The reduced post-war vegetation density also facilitates improved observation and interpretation of underlying hillslope terrain.

Additional years of aerial photographs were only reviewed as part of the relict landslide mapping process under two scenarios. The first was where relict NTLI landslides were recommended for deletion following 1963 aerial photograph review. For such cases, the year of aerial photography recorded for the NTLI feature (typically 1945 or 1964) was also reviewed to further validate the proposed deletion. The second was where no 1963 aerial photographic coverage existed for a particular mapsheet. For such cases, the highest resolution, lowest flight-height aerial photographs nearest to 1963 were reviewed.

3.2.2 Aerial photographs selection

A GIS solution was developed to facilitate aerial photograph selection. The system converted the aerial photograph centroid data into coverage footprints using flight-height, camera focal length and aerial photograph print dimensions. Algorithms were also developed to determine flight path vectors so as to ensure correct orientation of generated aerial photograph footprints. Aerial photographs relevant to particular 1:5,000-scale mapsheets were automatically selected and aerial photograph lists then generated.

Class A1	Class A2

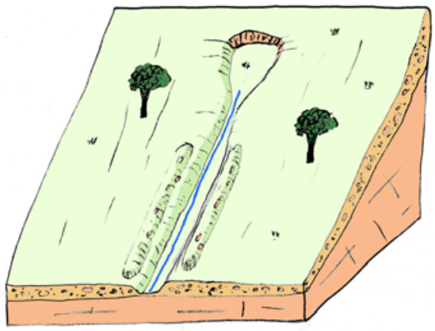
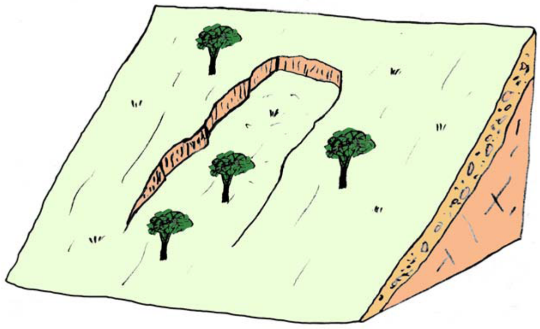
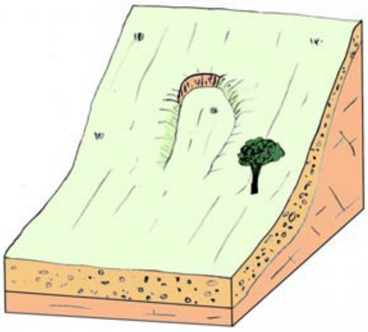
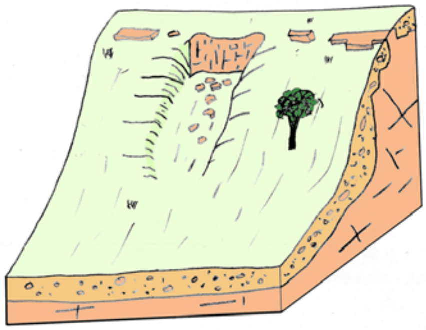
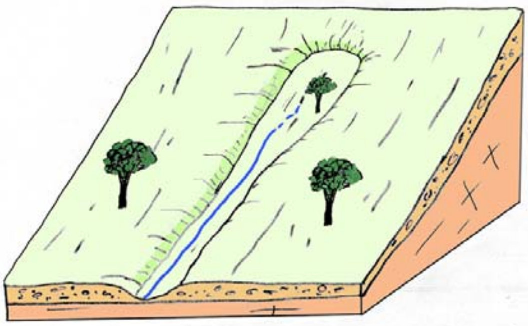
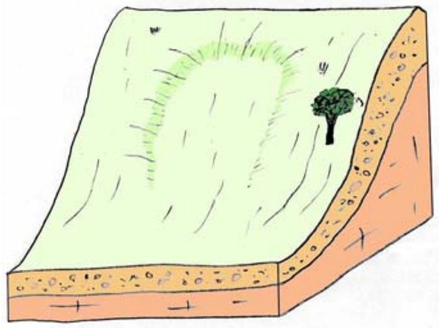
	
• Debris evident (levees), which are clearly related to source. • Source area scarp mainly rounded. • Vegetated source and debris. • Well-defined drainage line. • Assigned certainty 80%	• No debris evident. • Source area scarp mainly sharp. • Vegetated source. • No drainage line present. • Assigned certainty 80%
Class B1	Class B2
	
• No debris evident. • Source area mainly rounded. • Vegetated source. • No drainage line present. • A depression inconsistent with adjacent slope morphology. • Assigned certainty 50%	• Possible subsequent rockfall debris. • Source area scarp mainly rounded. • Vegetated source. • No drainage line. • Rock outcrop forms part of scarp. • Assigned certainty 50%
Class C1	Class C2
	
• No debris evident. • Source area mainly rounded. • Vegetated source. • Head of drainage line. • No clear evidence of mass movement. • Possibly result of fluvial erosion. • Assigned certainty 10%	• No debris evident. • Broad depression bounded by gentle break in slope. • Vegetated source. • No drainage line present. • No clear evidence of mass movement. • Assigned certainty 10%

Figure 1: Schematic examples of landslide classifications

3.2.3 Aerial photograph review procedures

Available aerial photographs were collected for review on the basis of three principal criteria. These were:

(i) Low-flight height. As the aim of the ENTLI was to improve and update the preceding NTLI, review of low-flight aerial photographs was imperative. Where low-flight aerial photographs were unavailable or inexistent, relevant high-flight aerial photographs were reviewed.

(ii) Fullest coverage. Full mapsheet coverage of reviewed aerial photographs was required to ensure that mapsheets had been effectively surveyed. Where low-flight coverage was incomplete, high-flight aerial photographs were reviewed.

(iii) Post wet-season. Review of post wet-season aerial photographs (August to December) was prioritized in order to reduce the number of aerial photographs requiring review. It was also considered that such photographs would more likely contain landslide-related features warranting incorporation into the ENTLI. Where post wet-season aerial photographs were unavailable, inexistent or incomplete, pre wet-season aerial photographs were reviewed.

The review of 1963 aerial photographs for relict landslides identification was solely undertaken by senior API personnel to ensure consistency of interpretation and judgment. Internal cross-checking was arranged to confirm aerial photograph review procedures and ensure product consistency.

4 DATA COMPILATION & DIGITISATION

Data, particularly feature positioning, was input directly into a GIS, facilitated by digital orthophotographs and 1:1,000-scale topographical baseplans. The availability of 1963, 1973, 1982, 1993, 2000 and 2002 orthophotographs and 1:1,000-scale topographic data, provided greatly improved positional accuracy against the NTLI wherein data was manually drawn onto 1:5,000-scale baseplans and was subsequently digitised.

5 RESULTS

The ENTLI identified a total of 105,364 landslide features up to year 2003. These landslides included 15,794 recent and 89,570 relict records.

As part of the ENTLI compilation, 29,669 NTLI features up to year 2003 were reviewed. Of these, 7,954 features (26.8%) were deleted; locations for 20,559 (69.3%) features amended and only 1,156 features (3.9%) retained in their original locations. Most of the features recommended for deletion were determined to be non-landslide features such as gullies, rock outcrop and erosion, or man-made features with appearances similar to small-scale landslides (e.g. graves and ground investigation locations). Such features were likely to have previously been misinterpreted due to only high-flight aerial photographs having been reviewed in compiling the NTLI.

Of the 15,794 recent landslide features recorded by the ENTLI, the majority were classified as open-hillslope failures (10,551 records) with fewer channelised landslides recorded (5,020) and only very few coastal landslides considered to have been caused by coastal undercutting (223). These figures appear to identify the prevailing Hong Kong failure mechanism, although difficulty in differentiating channelisation from aerial photographs suggests that such judgments should be reserved and that the capacity of open hillslope failures in becoming channelised (given the presence of specific ground form characteristics), not be discounted.

Relict landslides represent the largest proportion of the ENTLI (89,570 records). Of these records, 19,624 represented landslides with 80% indicative probability of interpretation (Class A1 and A2), with only 1,207 of these records interpreted to demonstrate trails definitively associated with downslope debris accumulations (Class A1 only). A further 35,527 records represented landslides with 50% indicative probabilities (Class B1 and B2), with the majority of these records relating to the presence of degraded depressions interpreted to have been probably formed due to historic landsliding (Class B1). Only 9,269 of these records were interpreted to relate to failures emanating from rock-based areas with rocky backscarp features (Class B2). The remaining 34,407 ENTLI records comprised landslides with 10% indicative probability of interpretation (Class C1 and C2, 30,115 records) and coastal landslides interpreted to have been initiated due to coastal undercutting (4,292). These results are summarised for ease of reference in Table 1.

Table 1: Final ENTLI summary results

Slide Type	No. of Recents
C	5,020
O	10,551
S	223
Total	15,794

Class	No. of Relicts
A1	1,207
A2	18,417
B1	26,268
B2	9,269
C1	19,674
C2	10,441
S	4,292
Total	89,570

6 LIMITATIONS

6.1 Recognition

The ENTLI retains some of the intrinsic limitations of the NTLI that may lead to omitted or misinterpreted landslides. These limitations can be caused by the relative angle of the photograph and the observed feature, the presence of shadows, cloud and vegetation cover, the low and poor resolution of earlier photographs, and the possibility of an incomplete set of photographs for a particular area. It is also possible that some of the identified features are in fact not landslides, but had the attributes of landslides when viewed from aerial photographs. Furthermore, none of the landslides were examined or verified by site assessment and field mapping.

The ENTLI should only be used to provide a general indication of the distribution of landsliding on natural terrain and should not be relied upon for specific assessments of individual sites. Such assessments should include, amongst other things (c.f. Ng et al. 2003), a thorough review of all available high- and low-flight aerial photographs, backed up by detailed field mapping by experienced practitioners.

6.2 Date of occurrence

The year of photography on which recent landslides were first identified and the year of preceding aerial photography on which the landslide was not present, have been recorded. These years simply bracket the time period within which the landslide is considered to have occurred. Precise dating of the landslide event cannot be determined.

The date of occurrence of relict landslides also cannot be determined, as these features may relate to events that occurred hundreds or thousands of years ago.

6.3 Classification

Most of the landslides recorded in the ENTLI consist of channelised and open hillslope debris slides, debris flows, complex debris slide-flows, or composite slide-flow falls (Cruden and Varnes 1996).

Despite these variations, all ENTLI landslides are represented in the same way by a point representing the landslide crown and a line depicting the debris trail or the length of the landslide source area. The ENTLI feature will not reflect variations in the significant change of hazard that this represents. A small volume landslide of little consequence has been recorded in the same way as larger events, although this has been partially redressed through the incorporation of source area width and length data together with the recent and relict landslide classifications.

6.4 Relict landslides

Many limitations in identifying and recording the relict landslide dataset exist, given that they are very old, overgrown and often severely degraded topographical features, requiring considerable skills to interpret. In addition, the methodology of relict landslide interpretation and digitisation targeted the identification of single landslide features rather than large composite features.

It must be emphasized that the debris trail length has only been recorded for Class A1 features, with the digitized trails for the remaining relict landslide classes (A2, B1, B2, C1, C2 and S) only referring to the interpreted length of the landslide source area. Thus, the trail length recorded for these relict landslide classes differs considerably to that of recent landslides.

The above limitations pertaining to recorded ENTLI relict landslides further highlights the need for detailed site-specific API and desk study, backed up by comprehensive field studies.

6.5 Relevant aerial photograph selection

Given the tight programme for the ENTLI, selection of relevant aerial photographs was undertaken in accordance with the methodology detailed in Section 3.2. This methodology considerably reduced the number of aerial photographs requiring review. It is likely that additional ENTLI features would have been identified had all relevant aerial photographs of all flight heights been reviewed.

7 CONCLUSION

The Enhanced Natural Terrain Landslide Inventory (ENTLI) consists of a comprehensive record of historical natural terrain landslides in Hong Kong. The inventory was compiled from interpretation of available low and high-flight aerial photographs with records presented on 1:5000 scale map sheets and a geographical information system (GIS) for ease of input and subsequent analysis of the results. The basic information with the derived recent and relict status of the landslide has been incorporated into the GEO Slope Information System (SIS) for easy reference.

The ENTLI provides a useful indicator of the presence of past landsliding near a site but should always be supplemented with detailed site-specific interpretation of low-flight aerial photographs and detailed field mapping by experienced engineering geologists and engineering geomorphologists to bring it up to date, better classify the landslides and understand the history and geomorphology of slopes adjacent to a site.

With regular updating and ongoing correction of the ENTLI, it will form an up to date listing of all identified natural terrain landslides in Hong Kong.

ACKNOWLEDGEMENTS

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A Unique Fault Controlled Debris Slide near Shek Pik Reservoir Associated with the June 7th 2008, Black Rainstorm

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ABSTRACT

The intense rainfall during the Black Rainstorm of June 7, 2008 produced about 1600 natural terrain landslides on Lantau Island. Most of those landslides were failures of thin surficial deposits on sloping sites. This paper describes the detail of a relatively deep-seated, debris slide where failure appears to be controlled by an inactive bedrock fault plane intersecting a slope in a way that favored slope failure. Debris flows adjacent to this slide may have been a triggering mechanism but otherwise the rainstorm does not appear responsible for this anomalous failure. Although such bedrock failures are not common in this area, the likelihood of such large failures (up to about 16,000m^3 from a preliminary estimate) and potential consequences should be considered for developments on or near bedrock slopes. It is particularly important to consider the orientation and condition of bedrock discontinuities, such as faults and persistent joint sets in relation to site topography.

1 INTRODUCTION

The Black Rainstorm of 7 June, 2008 was an extraordinary event for the landslide prone slopes of Hong Kong in its intensity and geologic consequences. A rain gauge located at Keung Shan recorded a peak rainfall of 384mm in a four hour period on the morning of 7 June while the total rainfall for the 24 hour period was 622mm. Those rains were preceded by rainfall of up to 130mm the day before the Black Rainstorm. The Geotechnical Engineering Office of CEDD later identified over 1600 landslides on Lantau Island (unpublished data, GEO/CEDD). Photos taken by aerial reconnaissance and field mapping teams confirm that most of those recent landslides formed during or shortly after the rains on June 7.

The predominant landslide failure mode on Lantau Island involved the shallow translational sliding of residual soil and colluvium over saprolite of weathered rhyolite and dacite tuff. Where the debris had sufficient momentum to travel into existing channels, many debris avalanches transitioned into channelized debris flows. However, at least one slide did not follow this prescribed pattern of failure. A relatively deep-seated landslide, located in a complex of slides near Shek Pik Reservoir, failed along an existing, steeply-dipping fault rupture plane. As a preliminary estimate, the volume of the displaced mass could be up to about 16,000 m3, which was larger than the volume of any of the shallower slides of the more common failure mode. The slope failure moved as a debris slide composed of a mixture of intact bedrock blocks and disaggregated debris and, as such, represented an apparently unique mechanism of failure among the slides associated with the intense rainfall of 7 June 2008.

2 SETTING

The slopes above Shek Pik Reservoir are composed of Jurassic volcanic rocks of the Lantau Volcanic Group, overlain by surficial deposits of colluvium ranging in age from Pleistocene to recent. The Lantau Volcanic Group is composed of largely massive fine-ash vitric tuff, flow-banded rhyolite, and eutaxitic coarse ash lithic tuff. The rocks of the Lantau Volcanic Group are rarely exposed at the surface except along stream channels or in prominent outcrops, like the cliffs of flow banded rhyolite forming the ridge above Shek Pik. The climate accelerates chemical weathering of rocks exposed at the ground surface and they are commonly weathered to Grade IV to VI.

The Quaternary slopes above Shek Pik Reservoir are mostly covered by colluvium including both recent deposits and older Pleistocene deposits. The older colluvium, which is moderately to highly weathered, forms a colluvial apron on the middle and lower slopes above Shek Pik Reservoir. The weathered bedrock and older colluvium are incised by perennial and ephemeral streams that drain from the slopes. Older colluvium, ranging between 1 to 3 m thick, covered the lower slope where the failure occurred. Several large shallow failures and resulting debris flows occurred in the area adjacent to the more deep-seated failure, as a result of the June 7th rainstorm. The debris flows in this area were high velocity, high volume landslides depositing approximately 10,000 m^3 of debris into Shek Pik reservoir.

The bedrock along the north side of Shek Pik Reservoir is cut by numerous, inactive faults of different scales related to the regional-scale Tung Chung–Shek Pik Fault, which is part of the Sha Tau Kok-Pui O Wan Fault Zone (Figure 1). The age of the most recent movement of this fault is not known but samples from the Tai O–Tai Long Wan Fault, located approximately 3 km to the west, yielded a thermoluminescence age of 278700 ($\pm$23000) years on exposed fault gouge (Lee et al. 1998). The fault at the site of the slide, striking 070^o, is subparallel to the Tung Chung–Shek Pik Fault, which strikes 050^o where it lies within a kilometer of the slide.

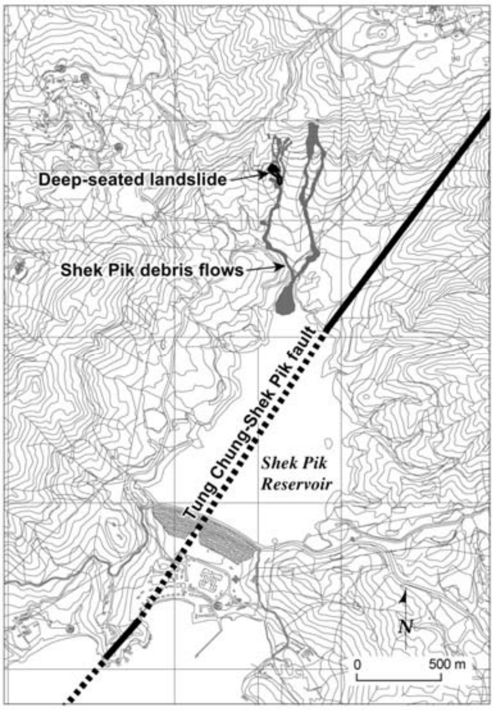

Figure 1: Location map showing the location of the deep-seated landslide relative to Shek Pik Reservoir, the Shek Pik debris flows and the Tung Chung-Shek Pik Fault

Several minor faults were observed in the stream channel that runs adjacent to the slide where channelized debris flows had scoured the channel to fresh bedrock. These faults typically had well formed subplanar fault planes with associated quartz veins, silica-cemented breccias, slickensides, and siliceous fault gouge. The fault plane upon which the deep-seated slide occurred was well-exposed over a large area nearly 35 m wide and 20 m high and contained similar features with easily identifiable spatial relations. The most prominent features on the fault were meter-scale mullions plunging 30 degrees to the west. The mullions were decorated with subparallel striae in siliceous fault gouge. Superimposed on the west-plunging lineations were dip-slip slickensides and quartz fibers in siliceous gouge indicative of normal motion on the fault and indicating possibly two directions of slip on the fault. Superimposed on these were a set of dip-slip striae formed where the surface had been scored by entrained mineral grains during the landslide event.

There were no recognizable surface expressions observed of the fault on pre-landslide topography, suggesting that the tectonic movements on the fault where much older that the present land surface. Indeed, the fault is not even indicated by any degree of differential weathering or topographic expression along strike. Where the fault projected across the adjacent channel it is buried by recent alluvium, although other faults were exposed along the scoured parts of the channel.

3 SLOPE STABILITY

The deep-seated failure described here appears to be due to the coincidence of slope angle and bedrock fault orientation. The failure occurred on a south-sloping spur ridge where bedrock was cut by the moderately high-angle fault dipping the same direction as the slope. The rounded spur ridge was approximately 50 meters wide with a crown that sloped 30^{o} to the south-southwest (Figures 2 and 3, and Plate 1). The fault, striking 070^{o} and dipping 50^{o} to the south-southeast, became the upper basal rupture surface and the scarp of the failure. The lower basal rupture surface was not exposed. It is possible that the fault has a listric geometry where the steeply inclined fault curves into a low-angle fault that forms the basal slip plane. Translation and tilt of large blocks in the slide suggest it is a curvi-planar failure surface (Figure 3). If the basal rupture surface dips 20^{o} the resulting volume of the slide would be up to about 16,000m^{3}.

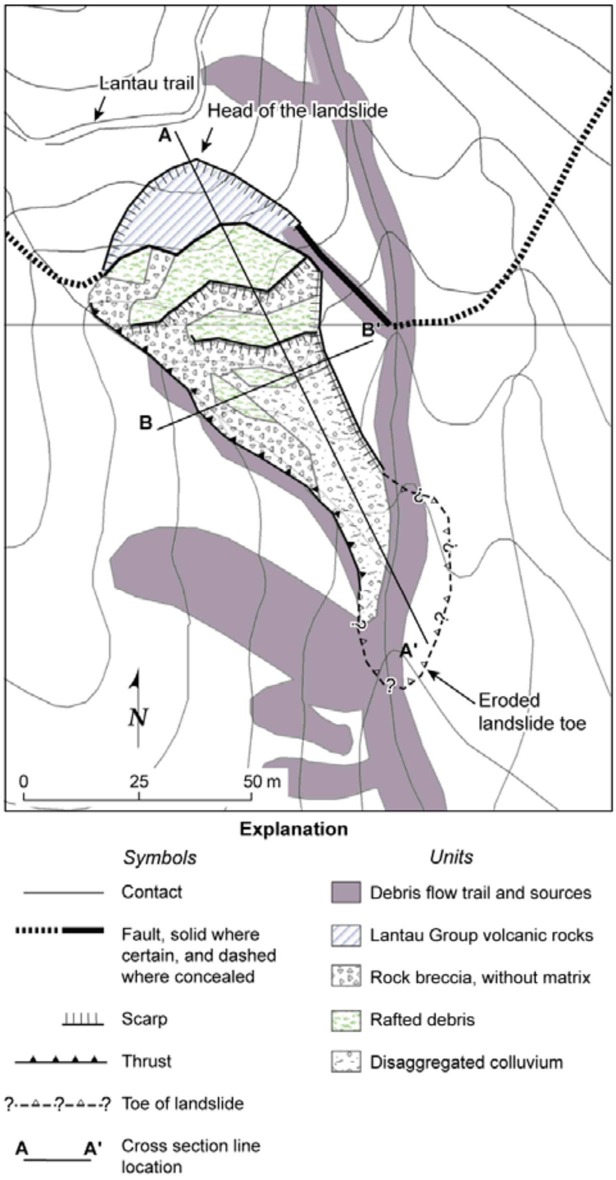

Figure 2: Generalized geologic mapping showing the architecture of the deep-seated slide and its relation to the adjacent debris flows

The debris slide contains bedrock tuff in the upper part and older colluvium toward the toe of the slide. The distribution of rocks influenced the architecture of the debris slide and includes large rafted blocks (Figure 2) and masses of jumbled fracture-bounded blocks (Plate 2) of the well lithified tuff. In some places, the jumbled fractured bedrock blocks appear to have moved like a rock avalanche deposit with a stream of blocks spread uphill on the western flank of the slide and individual blocks thrown into the vegetation 2 to 3 meters beyond the limit of the slide. The colluvium, which is well-consolidated where it is exposed in an undisturbed outcrop, formed a disaggregated mass of rounded cobbles in a matrix of silty sand (Plate 3). The disaggregated colluvium failed in a more fluid manner flowing as a granular mass into the toe of the slide where it filled the adjacent debris flow channel to depth of approximately 5 meters.

The angular bedrock breccia, extending from the head to middle part of the slide, is well drained and apparently was not significantly modified by the subsequent rains, which persisted through the month of June. By contrast the disaggregated colluvium in the toe of the landslide is deeply eroded and may have been a source of secondary debris flows. The portion of the landslide toe that flowed into the main channel, also composed of disaggregated colluvium, was entirely removed by subsequent erosion, except for a small remnant of debris preserved on the opposite slope 5 meters above the channel bottom (Plate 3). Erosion along the main debris channel removed only a small proportion of the displaced materials, possibly about 500 m^3 of debris that blocked the channel.

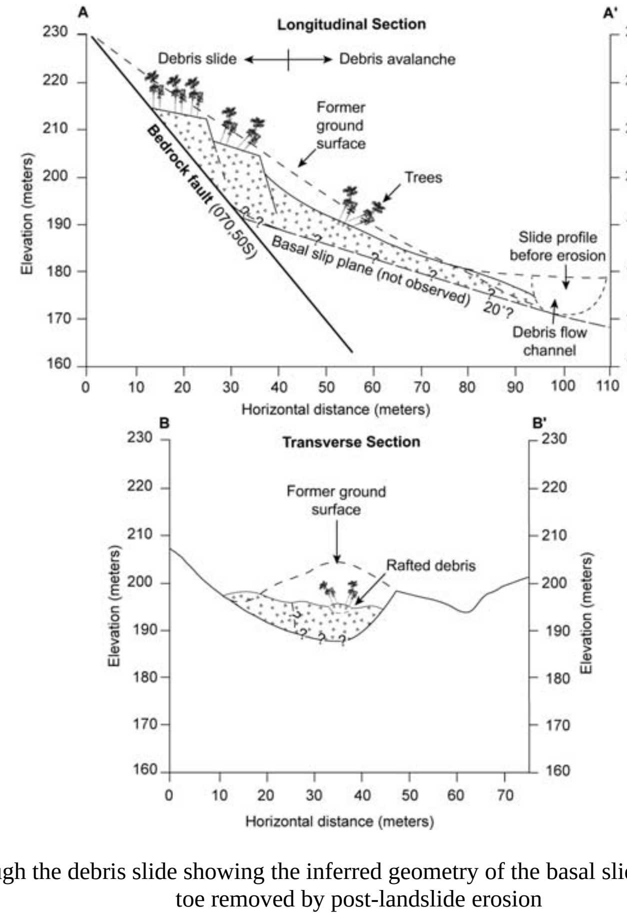

Figure 3: Cross-sections through the debris slide showing the inferred geometry of the basal sliding surface and the portion of the slide toe removed by post-landslide erosion

4 DISCUSSION

While it seems likely that the location of the landslide was controlled by a combination of topography and bedrock fault orientation, the triggering mechanism remains an open question. The large number of shallow landslides that occurred on 7 June 2008 can be reasonably attributable to the influence of heavy rains, particularly the period of high intensity rains. In the case of the deep-seated landslide described here it is unlikely that rain water could have quickly permeated to the depth of 20 meters where the basal rupture plane lay. The fault plane is largely unweathered, polished smooth, and there is no evidence of highly weathered rock or soil washed deep into the slide as might be expected if water entered the slide along pipe structures in the soil or bedrock. The possibility of increased loading by saturation of the overlying colluvium and soils is unlikely based on the relatively thin depths of those deposits (< 1m). However, the fault plane itself is a very weak surface and it is plausible that thin saturated superficial deposits acted as the "straw that broke the camel's back," adding enough additional mass to trigger failure.

Perhaps a possible triggering mechanism for slope failure could be the under-cutting of the basal slopes due to the entrainment of debris into passing debris flows. A significant debris flow with an active volume of about $1000m^3$ passed through the channel on the east side of the deep-seated landslide, where it appears to have entrained debris from the channel and sloping sides of the channel. A smaller debris avalanche formed in the drainage to the west of the slide but the details of that slide are not well understood because it was largely buried by the deep seated landslide debris. It is also possible that the impact of the debris flow and debris avalanches striking the basal slopes were contributing factors by transfer of momentum and vibration into the slopes that failed as the deep-seated slide.

5 CONCLUSIONS

Although the impact of a landslide of this type can be quite severe, it is fortunate that they are not common or widespread hazard. Nevertheless the depth of disturbance and the volume of debris moved suggest that slides of this type could have significant consequences for structures and human. Construction investigations on bedrock hill slopes need to evaluate the potential hazards resulting from persistent zones of weakness and discontinuities for the potential to cause deep-seated slope failures. This is particularly true for construction sites where excavation or borings reveal even minor fault planes. Ensuring adequate safety of the site should include detailed structural analysis and use of inclined borings to determine that unsafe discontinuities are not present.

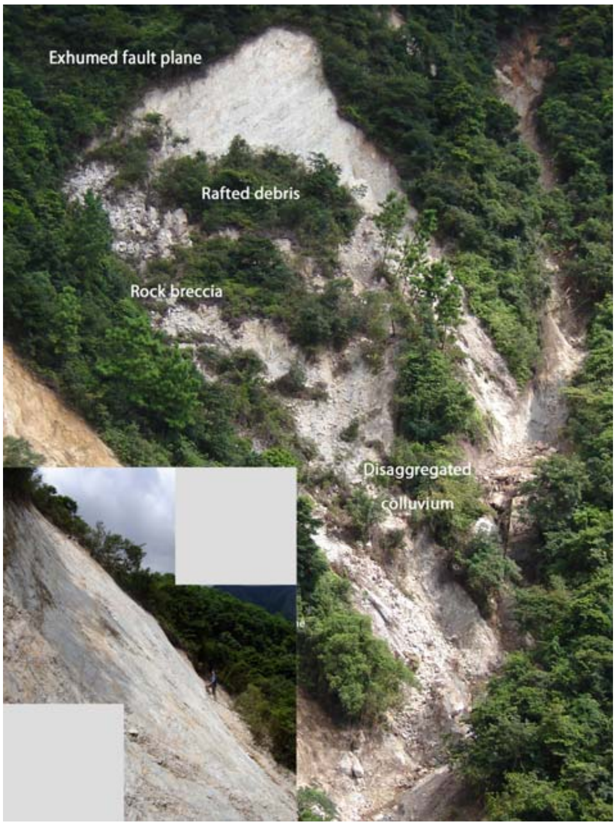

Plate 1: Oblique aerial view to the north of the deep-seated debris slide showing the exhumed fault plane, and the distribution of the debris. Lineations are visible on the fault plane dipping to the west. The inset photograph is a ground level view showing the exhumed fault plane

Plate 2: Rock breccia formed in translated bedrock within the debris slide

Plate 3: Disaggregated colluvium in the foreground is located in the middle part of the slide. Distal debris below the far trees are remnants of the landslide toe on the far side of the debris flow channel. The remainder of the landslide toe has been removed by post-landslide erosion

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Destabilisation of Natural Slopes Relating to Soil Density Loss – Strength Reduction

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ABSTRACT

The natural phenomenon of rock decomposing through chemical weathering processes leads to major strength loss with stress reduction accompanied by soil volume increase and movements relating to stress release, influences of gravity and adverse climate affects. The soils decrease in both density and strength towards the ground surface. The setting for geomorphological processes governs the composition of much of the overlying loose under-compacted soils. The soils may reach critical densities or less which allows a collapse mechanism to exist within the soil structure promoting liquefaction debris flow failures during heavy rainstorms. The soil density - strength relationship relates to particle interlock and the critical state framework enabling relationships to be established that are dependent on the density of the soil and for critical densities to be determined. Through fieldwork measurements the density profile can be related to strength and therefore the assessment of slope stability for potential planar modes of failure. Theoretical determinations of slope factor of safety relate soil strength, density, degree of compaction and slope angle together. Further assessment of natural slope stability needs to consider deep seated non-planar failure surfaces, groundwater regime affects and adverse geology such as relict jointing and clay mineralogy.

1 INTRODUCTION

1.1 Formation of natural slopes

The Hong Kong natural landscape is high and steep sided with the underlying geology comprising mainly granitic or volcanic rocks from the Jurassic / cretaceous periods. The formation of natural slopes on hillsides is a combination of decomposing underlying rock and processes of geomorphological origin. With the decomposition of rocks covering geological time periods, the hillsides slowly loose mass by the removal of thin layers of saprolite and/or colluium that had previously formed. The colluvial stratum is normally in a highly disturbed state overlying decomposed rock. The natural phenomenon of rock decomposing through chemical weathering processes leads to major strength loss of the rock over time to that of a residual soil. The breakdown of the crystalline structure of rock forms the clay / silt and granular content of the saprolitic soils. Associated with weathering is a change in soil properties including strength, density, permeability and voids ratio. Combined with the long term stress reduction is an accompanied volume increase and particle movements relating to the influences of the force of gravity.

1.2 Geomorphological influences

As rock decomposes with the gradual formation of soils, a general decrease in strength towards the ground surface gives a reducing density profile until soil from geomorphological processes are encountered as erosional products making up the overlying soil strata. The active geomorphological processes include toppling of rock outcrops producing talus deposits but mainly involve surface erosion of soil particles, washouts causing debris floods, infiltration causing landslides or debris flows linked to liquefaction of low density soils.

With the deposition of colluvium on the slope surface, the geomorphologic deposits comprise loose under-compacted material. With the surface soils of hillsides prone to the leaching out of fine material through seepage other severe climatic actions, particularly associated with adverse weather conditions provide the destabilising trigger force necessary to initiate slope failures or further erosional deposition. The groundwater response during periods of heavy rainfall is usually considered but the relationship of soil properties within a degrading soil profile requires further examination.

Whilst slope soils remain in position without noticeable disturbance during dry seasons, external surface water flow and internal actions of water seepage in the wet season will promote density loss through hydrogeological leeching actions, i.e. as density reduces with fines removal, void ratio and permeability increases further promoting subsurface seepage and soil pipe formation. Soils that reach a critical density are likely to fail during heavy rainstorms through a collapse mechanism of the soil structure and the promotion of liquefaction debris flow failures. The formation of soil pipes and a porous soil matrix therefore allows for a rapid absorption of water prior to failure and liquefaction, typically in the presence of increasing pore water pressure corresponds to the rise in the water table during periods of heavy rainfall percolation . The identification of soils with low density with a porous nature are therefore an important step in understanding the localised behaviour of destabilising soils.

1.3 Soil properties

The properties of saprolite soils can be examined for variation of dry density, relative density and strength relating to structure, particularly within the rock grade V and VI profile. Associated with the decomposition of the rock mass are the long term changes in soil properties most notable being density reduction and strength reduction during the conversion from highly to completely decomposed rock and finally residual soil. Further reduction caused by the removal of the finer clay or silt particles with an increase in water or air filled voids. The impact of density reduction was noted by Ng & Lumb (1980) who showed that soil density affected soil strength along with a decrease in the rate of soil dilation during shearing. Ng & Lumb (1980) further showed that at a critical soil density, no dilation was evident during shearing i.e. a constant volume existed. Also soil displayed contractile behaviour for very loose soils with a collapsing mechanism evident with the soil densifying instead to the critical density during strength testing.

1.4 Soil density measurements

Through fieldwork measurements, the density profile can be related to the strength profile as obtained from a series of laboratory strength tests at different densities. Density changes continuously over small depth increments therefore in-situ density tests from mazier or U-100 samples need to be closely spaced provide a reasonable profile. A GCO probe or cone penetration test (Figure 1) enables a continuous profile to be obtained which can be related to strength and soil classification tests.

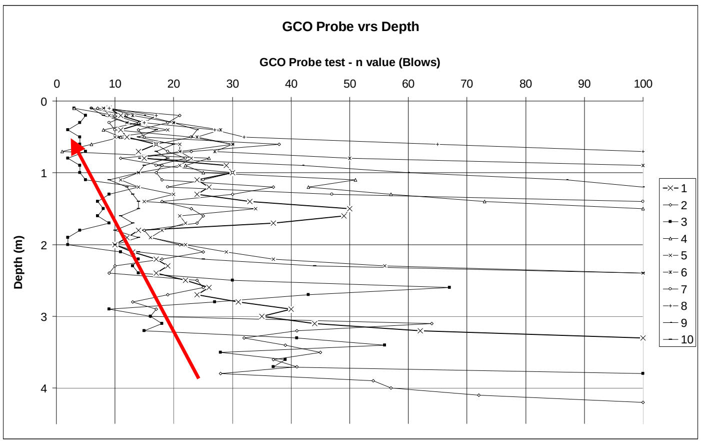

Figure 1: Result from GCO probe test indicates loss of density with decreasing depth

1.5 Factors affecting slope stability

The factor of safety of slopes can be assessed for potential planar modes of failure in relation to the slope angle and strength of the soils. Theoretical determinations of the potential for slopes to fail relate soil strength - density and slope angle which enable a quick assessment of potential slope instability. The analysis of larger events that are deep seated circular or other non planar failures requires the density – strength relationships to be considered along with other critical factors affecting the assessment on natural slope stability, with respect to groundwater profiles, rapid increases in pore water pressure e.g. through soil pipes, adverse relict jointing in rock and clay mineral composition that infill apertures within the underlying rock mass or relict soils.

2 SOIL PROPERTIES AND RELATIONSHIPS

2.1 Grading of soils with depth

The classification of soils is one of the important identifiers of soil type. Descriptions based only on observation are subjective and require good judgment whereas soil testing quantifies the soil objectively and without bias. One soil type may comprise a range of different properties due to density variation not readily identified unless a number of test results are compared to provide relationships and trends of results. Soil particle size grading provides a plot of soil type without reference to other soil properties such as moisture content, density or permeability which are provided by other tests. Therefore one soil type, e.g. CDG (Figure 2) can display a range of gradings and densities with different moisture contents with increasing depth. Particle Size envelopes need careful consideration (Wightman & Cheung 2008) for any soil stratigraphy or type due to changes in properties of a soil from say a low to a high fines content.

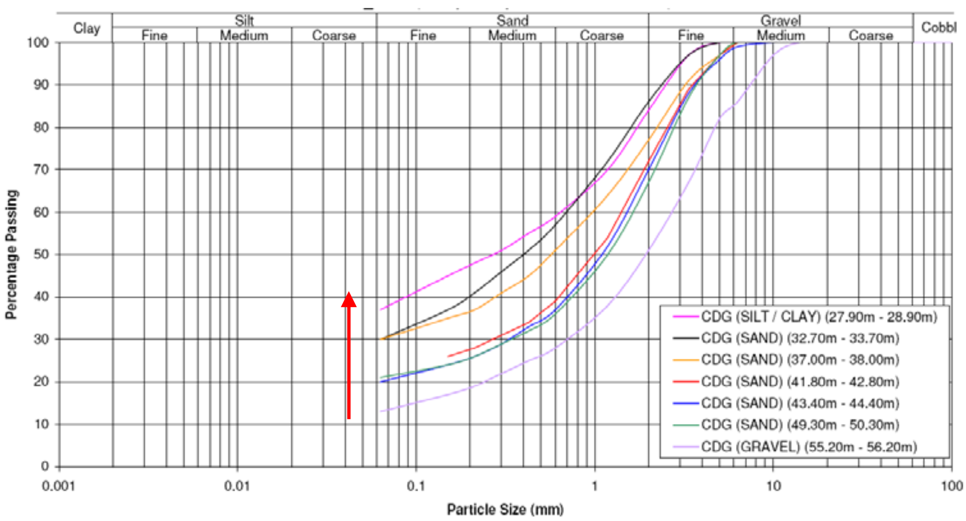

Figure 2: Decreasing coarseness of CDG from depths 55.2 m to 27.9 m from a sand / gravelly soil to silt/clay soil

2.2 Soil behaviour in shear strength tests: soil density – strength relationship

Soil density – strength relationships were determined by Brady et al. (1983) in controlled shear box tests (Figure 3). The lowest strength of a soil was obtained when the soil was shearing at a constant volume i.e. at the soil's critical density. Conversely the highest strength was achieved with maximum interlock between soil particles at the highest density (Wightman 2008). Research into soil critical state properties indicated that the soil interlock saw toothed model was a prime factor in the strength of soils (Taylor 1948) with the behaviour of soils in relation to the critical state described by Schofield & Wroth (1968). The measurement of the rate of dilation is related to the angle of interlock (Figure 4), a function of particle shape, mineralogy and initial density, Brady et al. (1983) and later by Bolton (1986). Both fine and coarse grained soils display this dilatancy phenomenon. Slope stability of compacted fill showed the influence of compaction of soil strength and therefore the factor of safety of fill slopes (Wightman 2008).

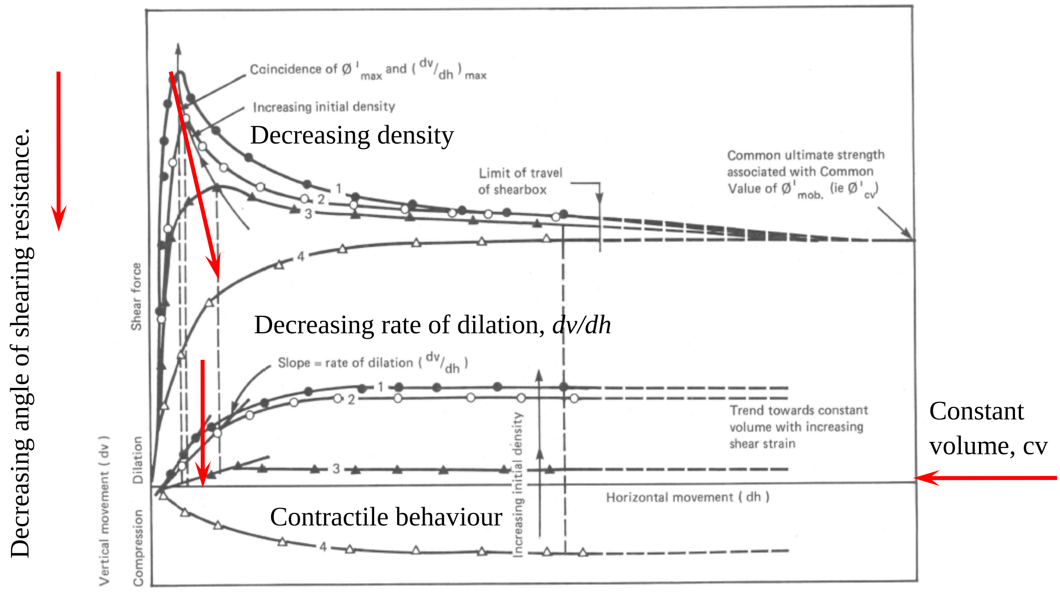

Figure 3: Effect of decreasing density on rate of dilation and angle of shearing resistance: general behaviour of granular soil in a shear box test (after Brady *et al.* 1983).

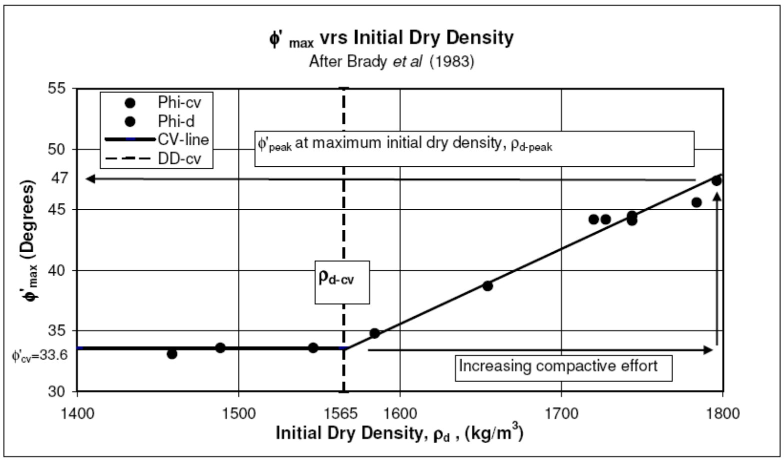

Figure 4: Effect of increasing compactive effort on angle of shearing resistance: relationship between ϕ'_{max} and ρ_d for a vertical stress of 120 kPa (after Brady *et al.* 1983)

3 SLOPE STABILITY AND THE DESTABILISATION PROCESS

The stability of natural slope is related to a combination of slope angle, relative density and soil shear strength which depends on the physical interaction from soil particle interlock. As the density of a natural slope decreases so the particle interlock decreases and the angle of dilation will tend to the critical state i.e. able to shear without contractile or dilative volume change but a very loose soil will shear with contractile behaviour. It is evident that as soil degrades the density loss will affect the strength value and the strength of the natural soil will be density dependent. The soil dry density-strength relationship is used to relate soil particle interlock, which is unlikely to be constant in the soil profile, due to the variable soil density particularly with the density loss phenomenon with decreasing depth. The soil shear strength is related to the critical state framework enabling relationships to be established that are dependant on the density of the soil and enable critical densities which occur at low density where shearing takes place with no volume change (constant volume), to be recognised. At the critical density the soil was deemed just stable. Figure 5 indicates that as the degree of compaction reduces (relative density reduction) so does the angle of shearing resistance until for a particular slope angle the factor of safety is reduced to 1. Overall slope stability relates to the slope angle as well as other forces acting such as water pressures. The Compaction Shear Strength Factor (C_{SSF}), the slope of the graph in Figure 4, is used to determine the soil strength from the measured soil dry density (Wightman 2008).

Table 1: Upper table shows factor of safety in relation to slope angle and soil angle of shearing resistance. The relation between dry density and angle of shearing resistance from use of the Compaction Shear Strength Factor, C_{SSF}

Design Density (kg/m³)																							
	Angle of Shearing Resistance (Design)																						
	24.0	25.0	26.0	27.0	28.0	29.0	30.0	31.0	32.0	33.0	34.0	35.0	36.0	37.0	38.0	39.0	38.2	37.1	35.2	33.3	31.4	29.5	27.6
Design Dry Density (kg/m³)	1196	1248	1300	1352	1404	1456	1508	1560	1612	1663	1715	1767	1819	1871	1923	1975	1936	1876	1778	1679	1580	1481	1383
Degree of Compaction C_d (%)	61	63	66	68	71	74	76	79	82	84	87	89	92	95	97	100	98	95	90	85	80	75	70
	Range of Angle of Shearing Resistance															Peak Max	Degree of Compaction values						

Using the Compaction Shear Strength Factor

Compaction Shear Force Factor	C_{SSF} =	0.01926
Parameter	ϕ'	ρ
Constant Volume, cv	26.0	1300
Peak Max Values	39.0	1975

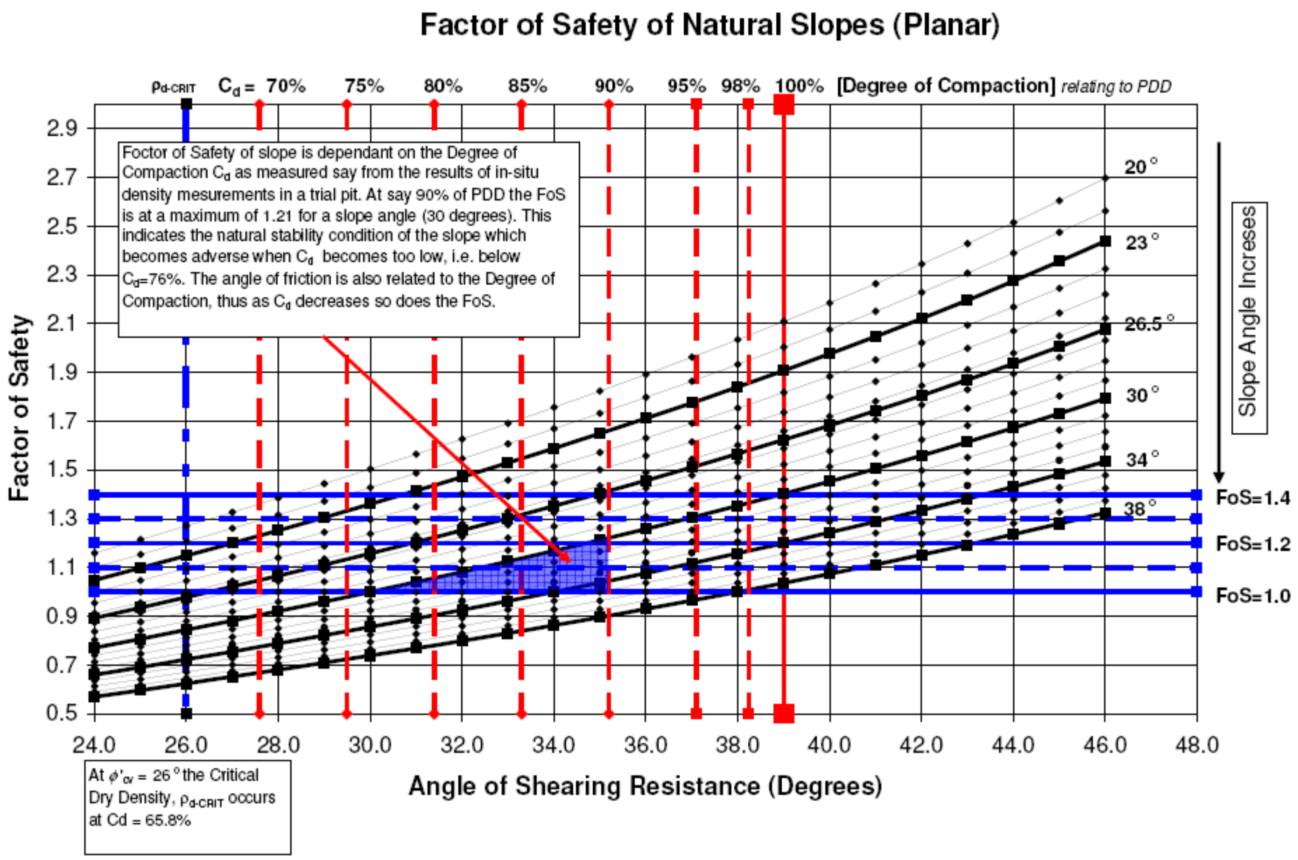

Figure 5: Graph showing the relationship between Factor of Safety, angle of slope, angle of shearing resistance and the degree of compaction of the soil. 100% C_d indicates a highly compacted soil whereas $\rho_{d\text{-}crit}$ is the degree of compaction of a soil at the critical state below which the soil becomes subject to a collapse mechanism leading to potential liquefaction

Slope stability issues of sudden collapse of loose soils can be identified by monitoring soil densities. Long term density loss is therefore one aspect of soil strength that leads to the long term destabilisation of natural slopes. A characteristic set of soil parameters relating to the density / soil profile, would include variations of strength, soil density, grading and moisture content that can be used for slope stability assessment.

4 CONCLUSIONS

Soil properties are affected by long term decompositional processes in saprolites. One main indicator of strength reduction is the low density / high voids ratio caused by density loss that occurs over time as fines are leached from the saprolite or loose soil collect leading to eventual strata weakening and slope instability. Colluival deposits change continuously through geomorphological erosion and depositional processes. The soil density and the relative density increase with depth tends to produce a profile of soil strengths. The derivation of the shear strength / dry density relationships aid the interpretation of natural slope factor of safety and provides a means for assessing the general stability condition of a slope. At critical soil densities, contractile behaviour is displayed linked to collapse mechanisms and debris flow where water infiltration triggers a failure. A dilative soil would strain harden depending on the rate of strain, which can rupture only once full shear strength has developed. Further investigative work in the area of soil properties would extend the understanding of the soil behaviour at failure.

5 RECOMMENDATIONS

Geomorphological field studies with soil density measurement from natural slopes would provide insight to the range of properties in relation to the topographical settings of colluvium with the identification of potentially unsafe hillsides. Hydrogeological models would also further enhance failure models.

Continuous cores logged for a continuous set of soil properties for the entire soil profile could include a range of classification tests and other observations such as colour (Munsell Soil colour chart), PSD, mineral composition, shell or organic content (roots, etc.), sedimentary structures, banding and other macroscopic features such as the presence of fossils. The PSD of smaller particles passing 841um size (sediment faction of samples) could use a Coulter laser diffraction analyzer model LS100 to determine the relative proportion of sand, silt and clay. Identification of clay minerals using XRD methods would enable the identification and quantification of the clay minerals present. The decomposition of rock minerals into clays and further chemical changes of clay minerals may indicate when leaching would become more prevalent. Kaolin deposits on relict joints in the saprolite and present in residual soils would also affect shear strength.

Density profiles, low density zones and variation of density within a sample tube can be logged by using a multi core logging system (gamma density) which is able to determine the location / areas of reduced density non-destructively. Methods to determine the density profile and fines content within slope profiles could related to the shear strength from a series of soil strength tests with confirming classification tests carried out on the test samples. To further understand the stability state of natural slopes, the density – strength relationship is required for each type of soil and existing relative density of the slope needs to be measured and related to geological model for further analyses of the potential for destabilisation of the natural slope.

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Study of Landslides with Long Travel Distances across Natural Terrain, Lantau Island

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ABSTRACT

The Hong Kong Special Administrative Region is located in a sub-tropical region prone to intense rainstorms during its wet season, which is typically the triggering mechanism for landslides in the hilly terrain typical of Hong Kong. During the rainstorm of June 7th 2008, the rainfall intensity exceeded 700 mm over a 24 hour period, which was the highest recorded intensity with a return period of about 1 in 1000 years for this area of Hong Kong. This activated numerous landslides across the territory and, at numerous locations over the hilly and relatively remote terrain of Lantau Island, initiated landslide flows unimpeded by anthropogenic influences with travel distances often exceeding 500 m. As landslides of this travel distance could have severe impact on facilities, particularly in Hong Kong, their physical characteristics through detailed field mapping to systematically collect data for assessing landslide susceptibility in locations, for which such long run-out landslides may have more adverse effects. This paper outlines the findings of two such landslides located at Sham Wat, immediately northeast of the Ngong Ping "Big Buddha" Cultural village. The landslides were initiated within colluvium overlying tuff with varying degrees of weathering. The joint sets within the underlying in-situ material were typically orientated sub-vertically, running parallel to the faults and intrusions traversing the area, and sub-parallel to the ground surface. The landslide initiating factors were considered to be saturation of the colluvium overlying the relatively impermeable saprolite and bedrock. The paper emphasizes the use of thorough and accurate field data collection in order to identify the landslide triggering mechanisms and improve the understanding of long run out debris flows.

1 INTRODUCTION

As referenced by Glossop (1968), "If you don't know what you are looking for in a site investigation you are not likely to find much of value". This is particularly relevant for landslide inspections carried out in Hong Kong, which require detailed inspections to be carried out by experienced engineering geologists soon after the failure to ensure important and often transient information is collected for interpretation. The failure located on the southwest facing hillside, west of the Ngong Ping Cultural Village, Sham Wat Valley, Lantau, is an example of a natural terrain landslide with a long travel distance, extending over 500m in plan length. The landslide source occurred on an open hillside and immediately ran into an ephemeral streamcourse, after which the landslide entrained and deposited debris along its route, before intercepting the Sham Wat streamcourse at its toe. Refer to Plate 1 for the general location of the landslides. A second landslide, also an open hillside failure occurred to the northeast of the main landslide, followed a tributary stream course intercepting the main landslide towards its lower levels. This paper outlines the findings of the detailed site inspection and preliminary literature search and provides estimates for the landslide failure volume, entrainment and deposition volumes, with reference to the approximations inferred from the field mapping data.

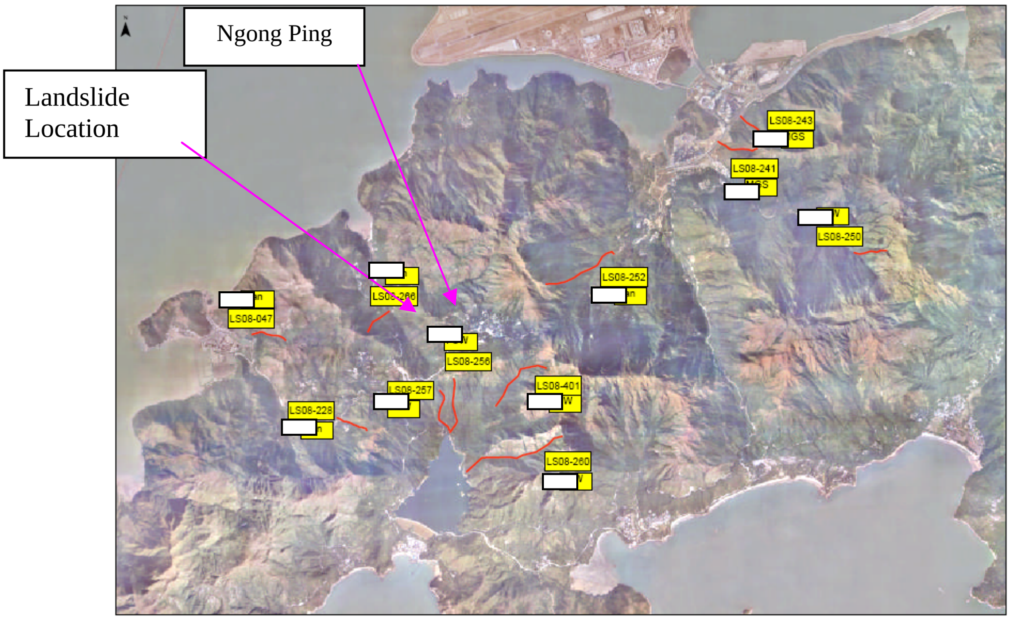
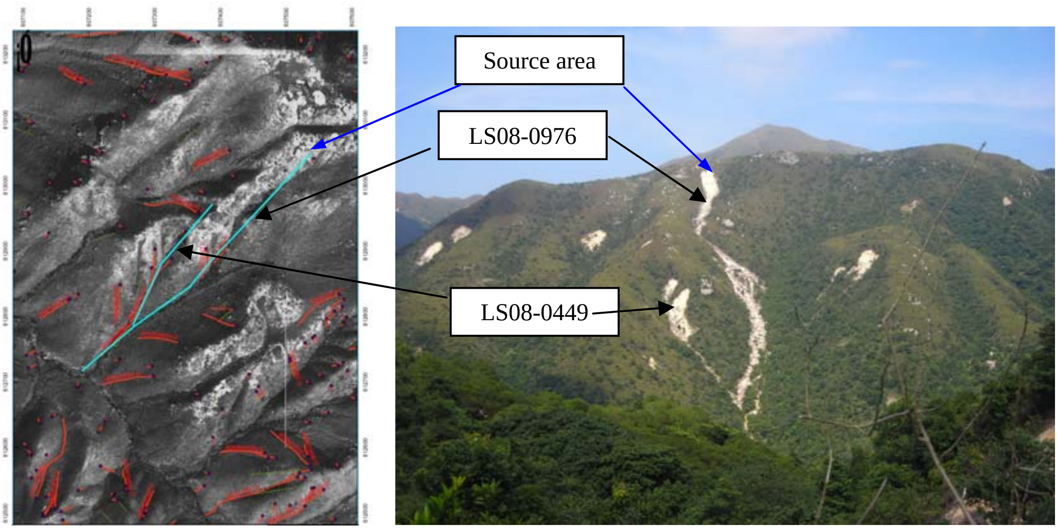


Plate 1: Landslide locations with travel distances greater than 500 m following the rainstorm of June 7th 2008, Lantau Island

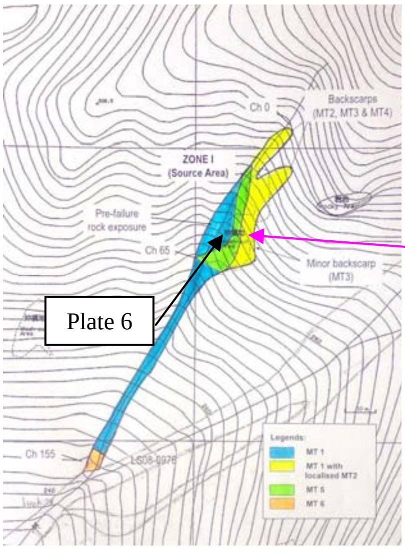


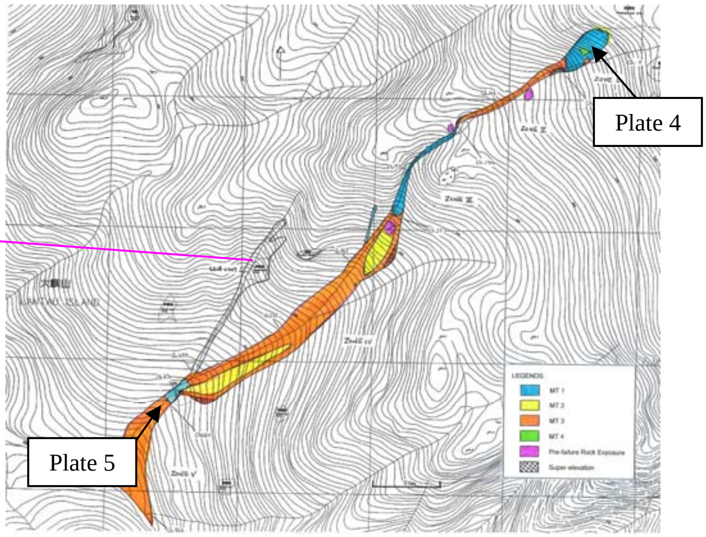
Plates 2 and 3: Location of the Landslides, near Ngong Ping, Lantau Island

2 SITE SETTING

An overview of the landslide site setting was obtained from a review of available topographic data, historical and recent aerial photographs and was further verified during the site inspection. The uppermost level of the landslide source, referenced LS08-0976, is located at level +432 metres Principal Datum (mPD), about 20 m vertically below a ridgeline bounding a broadly concave depression and trending approximately NNW. The landslide and debris trail extending from the source was bounded by ENE trending ridgelines. Refer to Plates 1 to 3 for the site location. The source failed into an ephemeral stream-course located at about 414 mPD, after which the debris trail followed this stream-course for about 650 m plan length before reaching a perennial stream-course at its toe at 215mPD.

A smaller landslide, referenced LS08-0449, had a source located at a lower level and northwest of the main landslide, at about 310mPD and joining the main landslide at 246mPD. This landslide followed a relatively straight alignment, trending northeast to southwest, intercepting the main landslide, as it realigns slightly towards the south. The source of the smaller landslide is also located in a broadly concave depression separated from the main landslide by a rock promentry rising abruptly from the surrounding ground surface and located at a similar level to the head of the landslide source.





Figures 1.and 2: Site Inspection findings for Landslide Nos. LS08-0449 and LS08-0976

3 GEOLOGICAL SETTING

The geological setting of the landslide is outlined in the available literature and maps (Fyfe et al. 2000; GCO 1994). Refer to Figure 3 for the location of the main geological features. The solid geology underlying the landslide site is referred to as metamorphosed and undivided rhyolite lava and tuff of the Lantau Volcanic Group, comprising welded and non-welded rhyolite tuffs with tuffaceous and sedimentary rocks. Tuffite and tuff is located beyond the head of the landslide. The volcanic saprolite, overlying the majority of the bedrock varies in thickness, typical of volcanic formations, weathering to a fine grained soil, typically ranging from 30 to 90 per cent fines content, derived from the tuff's fine grained constituents. In addition colluvium is located along the ephemeral streamcourse. This is described as slope debris, formed during the last ice age of the Pleistocene and Holocene periods, making it relatively weathered, dense and stiffer than the recently deposited overlying colluvium.

The structural geology comprised a major fault generally trending north to south along the streamcourse, intercepting the landslide toe. The discontinuities measured for the area were aligned sub-parallel and perpendicular to the fault, dipping sub-vertically. An additional major joint set is also indicated to trend north to north-west at 20 to 30 degrees (Sewell et al. 2000).

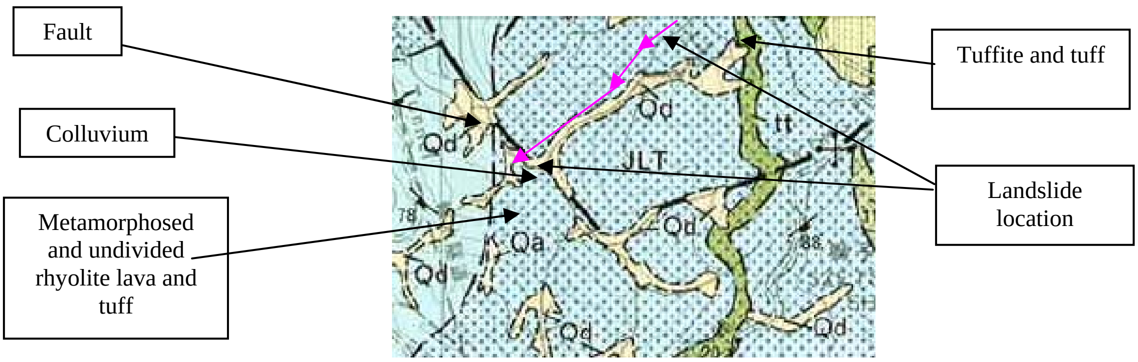


Figure 3: Geological Setting, GCO (1994)

4 SITE INSPECTION

Due to the extensive debris trail, over 650m in plan length, detailed landslide inspections were carried out to identify the landslide material, estimate the landslide volume, particularly the source, entrainment, deposition within the source and volume removed by the stream course at the toe, and gain a preliminary understanding of the failure mechanism. As topographic detail was unavailable before and after the landslide, the volume estimations were determined by topographic characteristics observed on site. The geology observed during the site inspection generally corresponded with the geological literature. The material types (MT) identified are summarized in Table 1 below and located in Figures 1 and 2.

Table 1: Ground identified during the detailed site inspection

Material Type (MT)	Description
1	Moderately strong, pale brownish grey, spotted grey, moderately to slightly decomposed, coarse ash tuff (BEDROCK Lantau Volcanic Group).
2	Weak to moderately weak, yellowish brown, highly to moderately decomposed, coarse ash tuff (Very sandy silt with much angular gravel to boulder sized moderately decomposed rock fragments, SAPROLITE).
3	Soft to firm, yellowish brown, very sandy silt with much sub-angular angular gravel, cobble and boulder sized fragments of rock, generally comprising fine ash tuff and generally exposed within the back and side scarps at the head of the failure (COLLUVIUM).
4	Soft to firm, yellowish brown, sandy clayey silt with occasional sub-angular angular gravels of rock fragments. Exposed locally within the back and side scarps at the head of the failure (RESIDUAL SOIL).
5	Firm, very sandy silt with much gravel to boulder sized rock fragments. Exposed mainly towards the lower portion of the landslide source. (DEBRIS FLOW DEPOSITS).
6	Gravel, cobble and boulder sized fragments of sub-angular rock, generally comprising coarse ash tuff, exposed mainly within the lowest levels of the landslide debris trail (LANDSLIDE DEBRIS).

The material located within the both landslide main scarps comprised colluvium and saprolite (MT 2 and 3) with the exposed rupture surface locally comprising bedrock (MT1). Refer to Plates 2 and 3 below for reference. Along the debris trails, running along the ephemeral streamcourses, both colluvium (MT 3) and saprolite (MT 2) had become exposed within the flanks. As the stream gradient for the main landslide reduced from 40 degrees to 20 degrees from the uppermost to lowest levels of the debris trail respectively, the amount of debris flow deposits and landslide deposits (MT 5 and 6) increased in volume as the gradient reduced towards the lower levels accordingly. Considerable increases in the volume of debris deposition had also occurred in changes in direction of the streamcourse, being deposited with a raised profile described as "super-elevated".

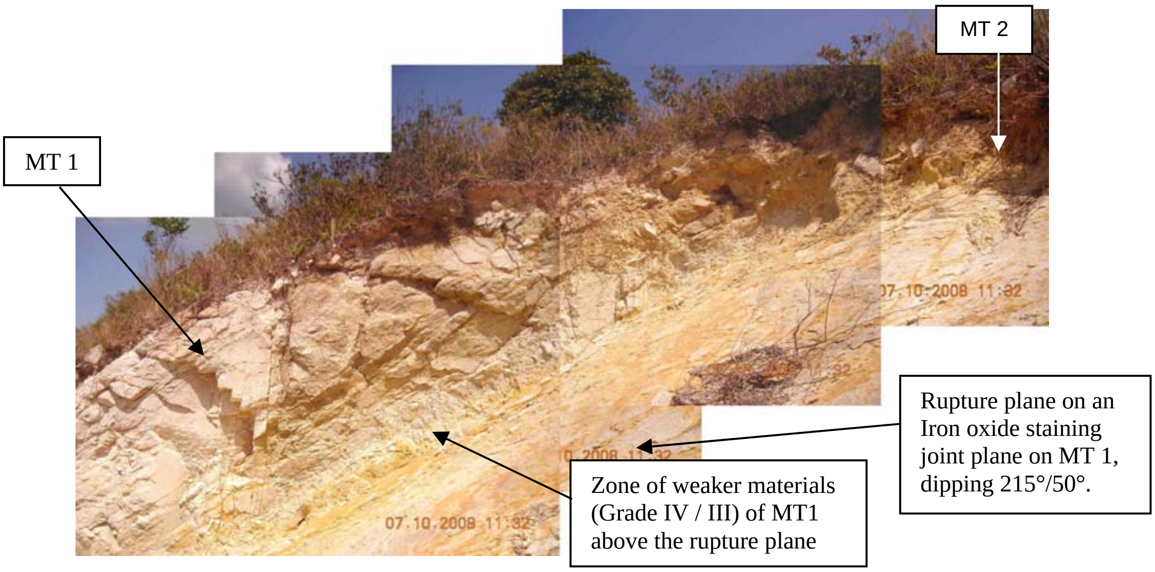


Plate 4: Landslide, LS08-0976

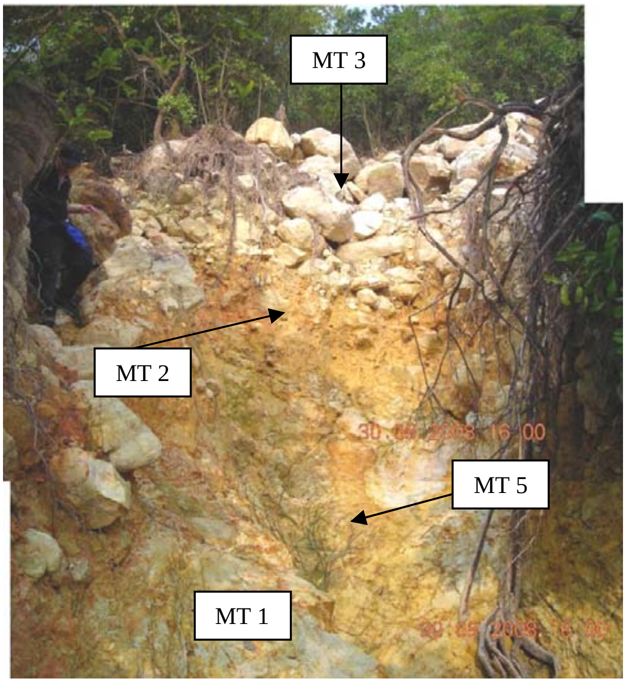


Plate 5: Debris Flow

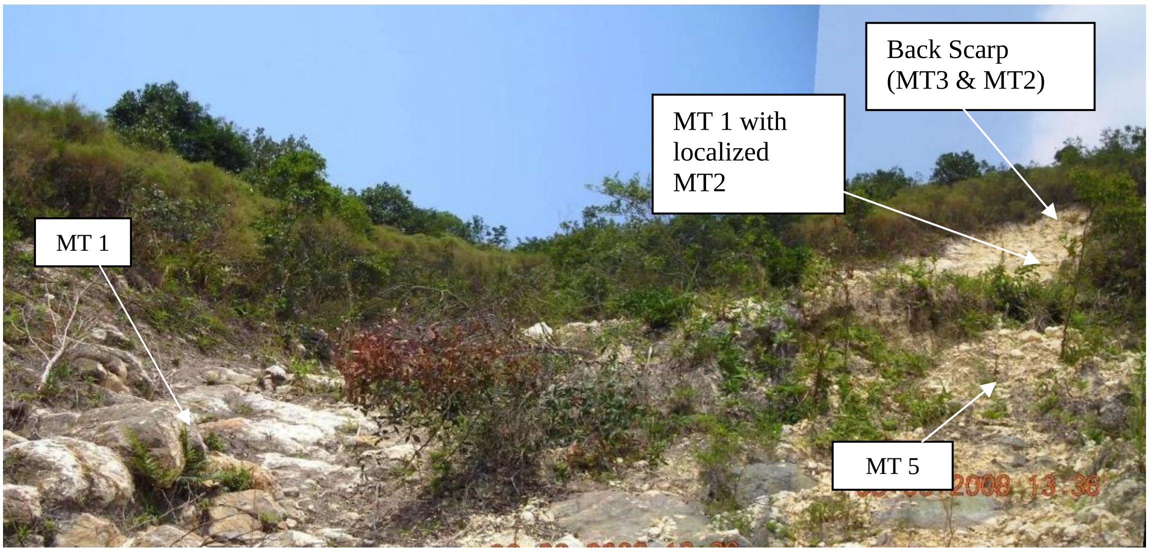


Plate 6: Landslide Back Scarp, LS08-0449

5 INTERPRETATION AND LANDSLIDE INITIATION

Both landslides were initiated by prolonged rainfall that occurred on the 7 June 2008 saturating the mantle of thin colluvium (MT2), up to a maximum 1 m thickness, located above the relatively impermeable bedrock (MT1). As the landslides were situated towards the centre of concave depressions, the surface and sub-surface water flows were concentrated towards these locations exacerbating saturation. Furthermore as rock was exposed locally with the surrounding ridgelines, this increased surface water run-off and suggested the rock head level decreased gradually below the ground surface towards the centre of the depressions. The presence of soil pipes located at the base of the landslide back-scarp suggested rapid and erosive groundwater flow prior to the landslide initiation.

The discontinuities within the underlying bedrock were considered to have adverse orientations, with joint sets, orientated sub-parallel to the ground surface, generally approximating perpendicular to the local trend outlined by Sewell et al. (2000), with a smooth planar surface allowing a basal failure surface to develop. The sub-vertical discontinuities, aligned parallel and perpendicular to the local fault, acted as side and rear release surface for rock to become detached and assimilate with the landslide debris (Mackay and Swales 2008).

Volume estimates for the landslide source, entrainment, deposition within the trail and source, and the volume unaccounted for and potentially transported by the streamcourse along the toe, are tabulated below for reference. The estimates were based on the topographic characteristics interpreted in the field so may therefore be highly approximate.

Table 2: Volume estimates from the landslide

Mass Balance	Volume (m^3)
Total landslide volume from the source	142
Volume deposited with the source	36
Volume entrained along the trail	1260
Volume deposited in the trail	1470
Material unaccounted for	104

6 CONCLUSION

The landslide studied at Lantau provided an insight into the mechanisms of landslide failures with a long run out distance. As landslides of this type are often altered by rainfall and erosion, occurring after failure and removing features indicating landslide initiation, it is imperative that a site inspection is carried out promptly by an experienced engineering geologist to ensure accurate information is obtained. It is considered that the landslide had a considerable run-out distance, resulting from the intensity and duration of the rainfall. The factors initiating the landslide may be similar to many other landslides located in less rural areas and which therefore may have a more severe impact on facilities.

ACKNOWLEDGEMENTS

This paper is published is published with the permission of the Head of the Geotechnical Engineering Office and the Director of the Civil Engineering and Development Government of the Hong Kong Special Administrative Region.

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Natural Terrain Hazard Assessment for Housing Development at Kwun Lung Lau, Kennedy Town, Hong Kong

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ABSTRACT

Due to the high demand and limited space for development in Hong Kong, there has been a necessity to place major facilities adjacent to natural terrain which may be prone to instability. An example of this is the recently completed multi-storey housing block at Kwun Lung Lau, Kennedy Town, west Hong Kong Island, which is placed along the toe of the northeastern flank of Mount Davis. The natural terrain overlooking the development is densely vegetated rising at an average gradient of about 30° and height of about 224m from the site formation level to the Mount Davis summit. To ensure that all potential hazards from the natural terrain were identified and mitigated prior to commencing the construction of the residential block, a detailed site investigation, hazard identification and assessment were carried out. This paper outlines these processes and concludes that a successful natural terrain hazard assessment can only be carried out following a detailed site investigation and thorough assessment of the findings.

1 INTRODUCTION

For many decades, there has been increased pressure on land for development in Hong Kong leading to encroachment into natural terrain which constitutes about 60% of the Hong Kong's land mass. Particularly, natural terrains which are susceptible to various forms of instability have been cut into in order to form a development platform, thereby exposing the development to threats from the hillsides. As a result the Hong Kong government has imposed stringent regulations that require a thorough investigation of potential threats from natural hillsides and mitigation of such threats Ng et al. (2002). The Kwun Lung Lau Housing Development was extensive, and there were potential hazards imposed by the overlooking terrain on the northeast flank of Mount Davis (refer to Figure 1 for the location). Owing to this, a detailed natural terrain hazard assessment, satisfying the Hong Kong government regulations was carried out. This comprised a literature search, aerial photographic interpretation, field mapping and a ground investigation followed by hazard identification, hazard modeling and characterization as well as design of mitigation measures. It is concluded that the site investigation of the terrain gave confidence in identifying the type and scale of the hazards and allowed suitable mitigation measures to be implemented.

2 LOCATION AND TOPOGRAPHY

As shown in Figures 1 and 2, the development is located towards the NW portion of Hong Kong Island, on the NE flank of Mount Davis. The natural terrain catchment, as delineated in Figure 2, runs from the summit of Mount Davis, +268m above Principal Datum (mPD) to the site formation level, +45mPD and measures about 5.2 ha. Man-made cut slopes bound the overlooking terrain from its mid to lower levels to the NE and NW and along the toe. The Kennedy Town Service Reservoir is located to the NE, at about +95mPD. A 600mm diameter freshwater pipe connected to the reservoir is at the toe of the Study Area.

3 INFORMATION SEARCH

Relevant data from various local and regional studies were collated and reviewed. The summary of a comprehensive information search with respect to the ground condition, rainfall and landslide potential of the Study Area is given in Table 1.

Table 1: Summary of desk study findings

Document	Findings
Natural Terrain Landslide Inventory (NTLI), Evans et al (1997), King (1997)	One Class 2 landslide (i.e. width >= 20m) and 12 Class 1 landslides (i.e. width < 20m) were identified within the Study Area.
Large Landslide Study (GEO Record)	Confirmed the findings of the NTLI. Two large landslides registered as 11SWAL002 and 11SWAL010 were identified in the vicinity of the Study Area near the peak of Mount Davis. 11SWAL002, about 75m to the west of the Study Area, was 160m wide whilst 11SWAL010, about 120m to the east, was 180m wide. Findings were based on air photo interpretation.
Boulder Field Inventory (MGSL, ERM (1999))	Based on the 1963 aerial photos, boulders are present in the Study Area but no boulder fall incidents have been recorded within the Study Area.
Debris Avalanche Susceptibility Study (GEO Record)	Study Area is categorized as having a "high susceptibility" to avalanching with a "very high susceptibility" towards the uppermost levels.
Geotechnical Terrain Classification System, (GEOTECS)	Map sheets 5, 8 and 9 describe terrain as vegetated by mixed broadleaf woodland that is undisturbed and locally stands at gradients greater than 30 degrees.
The Geotechnical Land Use Map (GLUM)	The Study Area has extreme geotechnical limitations and is considered to be unsuitable for development.
Geotechnical Area Studies Programme (GASP 1988)	No specific information as the Study Area is marginally covered.
Rainfall Data (GEO Record for rain gauge No. H02)	Rainfall intensity recorded on 9 June 1998 is 70mm over 1 hour and 375mm over 24 hours. During the second wettest (July 1994) two rainfall related failures located at the upper portion of the Study Area occurred. Adjacent to the development, another failure occurred (on 23 July 1994) resulting in five fatalities with three persons injured.

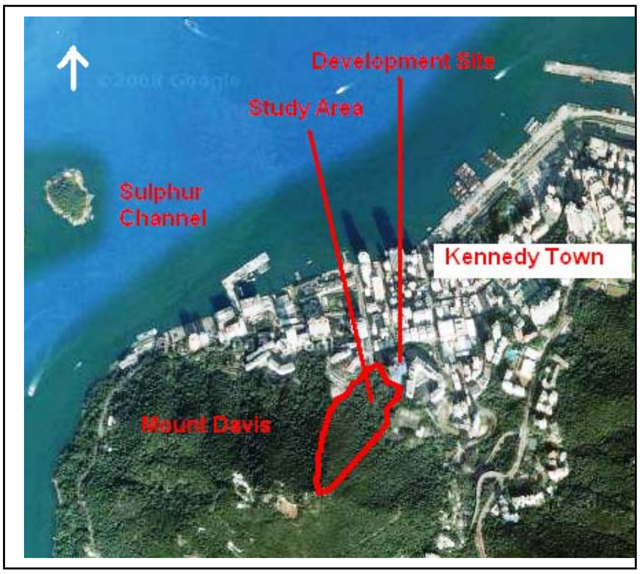
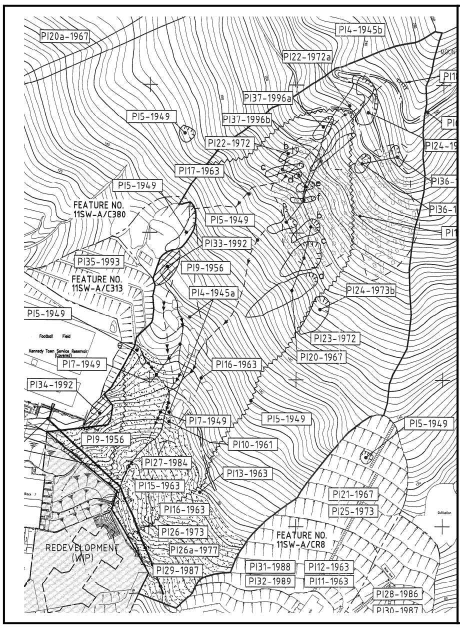
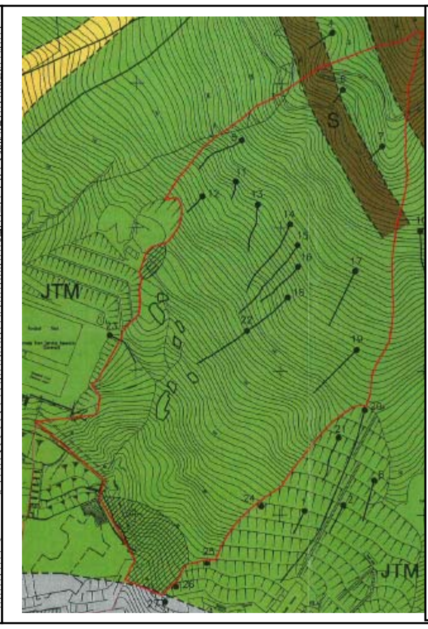


Figure 1: General location of the Study Area

Figure 2: Close up of the Study Area





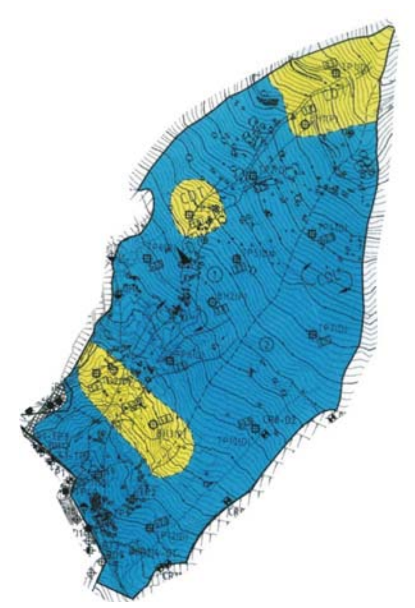


Figure: 3 Air photo interpretation

Figure: 4 Geology and NTLI data

Figure: 5 Geology (at 2m depth)

4 AERIAL PHOTOGRAPHIC INTERPRETATION

Dense vegetation generally covered the study area, however during periods of minimal rainfall, when sparse vegetation covered the slope, surface depressions inferred to be landslide scars were visible and linear depressions inferred to be ephemeral stream courses and vegetation along the linear depressions were identified. In addition, localized sheet, gully and surface erosion features, disturbed terrain due to the presence of squatters (near the lower levels of the Study Area), footpaths and boulder fields (mostly within the linear depressions and at upper levels) were observed. The observed landslide generally corresponded with the NTLI records.

5 GEOLOGY

The 1:20000 Hong Kong Geological Survey Map (Sheet 11) Study Area comprises mainly Mount Davis Formation coarse ash crystal tuff of Cretaceous period. Two bands of sandstone are present at the upper portion of the Study Area. GEO Report No. GR 5/2000 for Mount Davis confirms this geology. It also indicates the presence of perennial seepage. As shown in Figure 5, colluvium (COL) covers approximately 90% of the terrain. It comprises debris flow deposits described as silt with cobbles and boulders whilst saprolite (completely to highly decomposed tuffs (CDT and HDT)) underlie the colluvium. This is consistent with Figure 6 developed from site-specific ground investigation (GI).

Approx. Depth	RECENT GI STATION NUMBER														
	TP1(D)	TP2(D)	TP3(D)	TP4(D)	TP5(D)	TP6(D)	TP7(D)	TP8(D)	TP9(D)	TP10(D)	TP11(D)	TP12(D)	BH1(P)	BH2(P)	BH3(P)
0.0 - 0.3	COL	TSOIL	TSOIL	TSOIL	TSOIL	TSOIL	TSOIL	TSOIL	TSOIL	TSOIL	TSOIL	TSOIL	COL	TSOIL	COL
0.4 - 1.0	CDT	COL	COL	COL	COL	COL	COL	COL	COL	COL	COL	COL	COL	COL	COL
1.1 - 1.5	CDT	COL	CDT	COL	COL	COL	COL	COL	HDT	COL	COL	COL	CDT	COL	CDT
1.6 - 2.0	CDT	COL	CDT	COL	COL	COL	COL	COL	-	COL	COL	COL	CDT	COL	CDT
2.1 - 2.5	CDT	COL	CDT	COL	COL	COL	HDT	COL	-	COL	CDT	-	CDT	COL	CDT
2.6 - 3.0	CDT	-	CDT	-	COL	HDT	-	MDT	-	COL	CDT	-	CDT	COL	CDT
3.1 - 3.5	CDT	-	CDT	-	CDT		-	-	-	COL	-	-	CDT	CDT	CDT
3.6 - 4.0	-	-					-	-	-	-	-	-	CDT	CDT	CDT
4.1 - 4.5	-	-						-	-	-	-	-	CDT	CDT	CDT

Figure 6: Soil mantle stratigraphy of the Study Area

The saprolite had variable thickness, extending to depths exceeding 26m below ground level (m bgl). The groundwater monitoring records revealed basal groundwater levels ranging between 4 to 5m bgl, with occasional abrupt rises (up to 0.7m bgl) associated with rainstorms at the lower portions of the natural hillside.

6 FIELD MAPPING

Given the slope angle and the density of vegetation cover, detailed mapping of the Study Area was very challenging as there was need for thorough traversing. To ensure that all parts of the Study Area were inspected, 20m grids were established by topographic surveyors prior to the commencement of the field mapping which progressed with the aid of binoculars, electronic distance measuring device, global positioning system and digital cameras as well as the standard geological mapping tools.

This exercise revealed boulder concentrations corresponded with the API findings. Of the observed 207 boulders having at least one dimension greater than 1m, 98 boulders were considered to be potentially unstable. Based on their exposed dimensions, the boulder volumes ranged from < $2m^3$ (61%) to 20 to > $40m^3$ (4%). The larger diameter boulders generally being stable and appeared to be exposed bedrock. Table 2 shows summary of the distribution of the boulders. Defining shape factor, s, as the ratio of maximum dimension to the minimum dimension, up to 36% of the potentially unstable boulders have s = 2 to 3, 19% for s = 3 to 4 whilst 14% have s = 1.6 to 1.8 indicating that lesser percentage of the boulders are rounded.

Table 2: Distribution of observed boulders across the Study Area

Portion of Study Area (height is reference to slope toe at +44mPD)	Total No. of Boulders Observed	No. of Potentially Unstable (PU) Boulders	Percentage of PU Boulders within Zone	Percentage of PU Boulders across the Study Area
Upper third, from 150m to 230m	51	12 of 51	24%	12%
Middle third, from 70m to 150m	83	39 of 83	47%	40%
Lower third, from toe to 70m	73	47 of 73	64%	48%

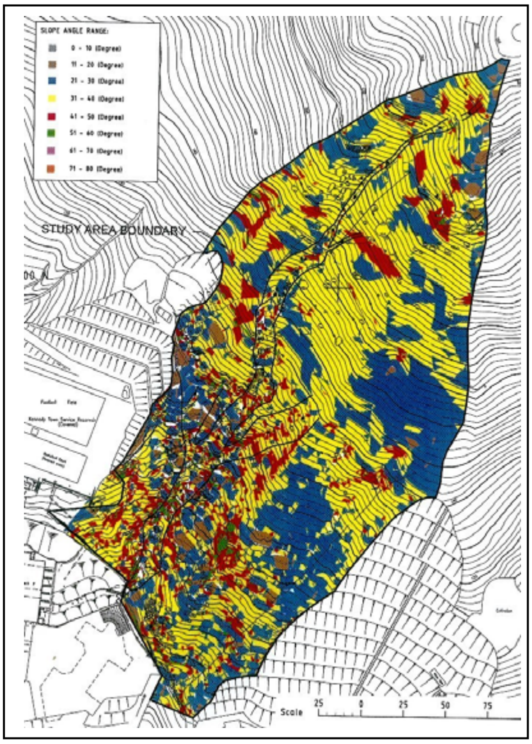


Figure 6: Slope angle distribution

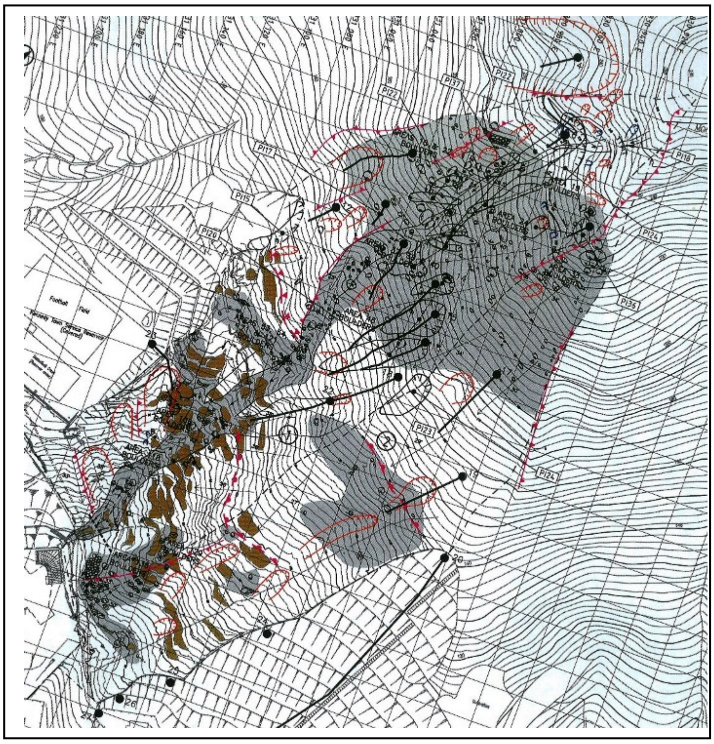


Figure 7: Engineering geological map

As shown in Figure 7, the terrain generally stands at 30° increasing to 35° towards the upper levels, with local over-steepening up to 50 degrees. Many landslide locations identified during the API were confirmed on site.

Similar to the boulder distribution, all portions of the Study Area showed evidence of previous instability varying from about $8m^3$ to about $384m^3$. In total, 39 failures, including 16 vegetation-depressions inferred to be relict landslides, and 23 failures classified as multiple and minor, were recorded. The observed failure scar widths measured from 3m to 12m, the depths rarely exceeded 1.5m. Five of the landslides were channelized, and perennial seepage over the lowest 50m of the terrain was observed along the stream courses, possibly due to a constant recharge from the Kennedy Town Reservoir situated about that level. Anthropogenic features, as indicated in the API, comprised buried pipelines, derelict squatter huts and cultivated areas towards the lower levels. Figure 7 shows the engineering geological map of the Study Area.

7 HAZARD ASSESSMENT

Site observations clearly show evidence of different scales of failures including boulder movement, relict and recent landslides. The presence of incised drainage paths will lead to channelised debris flow, potentially directing debris towards the development. There is only a limited buffer zone within the development site thereby increasing the concerns for potential threats from the overlooking natural hillside. Although tree canopies are well established, gaps under the canopies are sufficient to allow fairly unrestricted boulder movement. As indicated earlier, based on the boulder shape factors, up to 31% of the potentially unstable boulders are rounded and therefore can travel more in the event of a failure.

Ng et al. (2002) recommends three approaches to natural terrain hazard assessment, namely: design event (DE) approach, quantitative risk assessment (QRA) approach and factor of safety (FOS) approach. Given the known details of failures in the project area and the extent of the Study Area (over 5.2 ha), the FOS approach was limited to the assessment of the potential for a deep-seated failure. DE approach was preferred to QRA approach since it is cheaper and more common.

Various tools are available for characterizing debris flow and boulder fall. Lo (2000) presents various local and international design methodologies noting that factors such as geology, hydrogeology, geomorphology, characteristics of vegetation cover and basin terrain characteristics influence type and scale of landslide and the mechanism of failure. Characterizing debris in terms of solid concentration and water content is particularly important in determining the potential debris mobility, debris velocity and impact load as well as the run-up distance. Hungr et al. (1984) and VanDine (1985) identified channel debris yield rate to be pivotal in determining the eventual volume of a failure as this reflects the quantity of loose erodible materials within the channel. Based on the scale of failure within the Study Area ($384\ m^3$) and on the assumption that the yield rate will be $3.5m^3/m$ over a distance of about 200m along an existing incised natural drainage channel, the worst credible event was estimated to be $1000m^3$. Table 3 shows the summary of debris characterization exercise.

Table 3: Debris characterization

Velocity (m/s)		Run-out Distance (m) (adopting an average velocity of 11.64 m/s)		Run-up Distance (m) (adopting an average velocity of 11.64 m/s)		Debris Impact Load (kN) (adopting an average velocity of 11.64 m/s)	
Newtonian Laminar Flow	0.868	Sled Model	26.80	Sled Model	6.86	Hungr	538.35 kN (160 kPa)
Newtonian Turbulent Flow	**12.61**	Leading Edge	52.07	Sled Model (corrected)	1.22	Du et al	1615 kN **(479 kPa)**
Dilatant Flow	2.69	Constant Rate of Deposition	40.2	Leading Edge Model	2.69	Thurber Consultants	186.4 kN (55 kPa)
Du et al Empirical Formula	**11.69**	-	-	-	-	Scotton Deganutti	342.18 kN (101 kPa)
Basic Flow Eqn. v = Q/A	**11.11**	-	-	-	-	Lin et al	891 kN (264 kPa)

Clearly, landslide debris may constitute entrained boulders which may travel at a velocity similar to that of the general debris. Although, huge boulders have been entrained within landslide debris, for practical reasons, it was decided that all potentially unstable boulders greater than $2m^3$ would be stabilized in situ. As boulders having a shape factor greater than 2 could not travel as far as rounded boulders, further streamlining of the characterization of boulder falls was achieved. Table 4 summarizes boulder fall characteristics obtained by RocFall software along potential trajectories in the event of a failure.

Table 4: Boulder characterization

Boulder Character	Remarks
Travel Distance (m)	Boulders will reach slope toe with apposite travel paths.
Velocity (m/s)	12.1 - 14.8 (special case – 17 m/s)
Bounce Height (m)	0 - 5
Kinetic Energy (kJ)	65 - 2200

As would be expected, all parameters related to boulder fall vary with location, distance traveled and nature of slope surface defined. A plot of these parameters (such as the kinetic energy envelope) along the potential failure paths was useful in determining the most cost effective locations of the major mitigation measures, in this case two check dams at (A) and (B) and one flexible barrier at (C), where energy levels are relatively low and there is space for construction without having to fell any mature trees, see Figure 8.

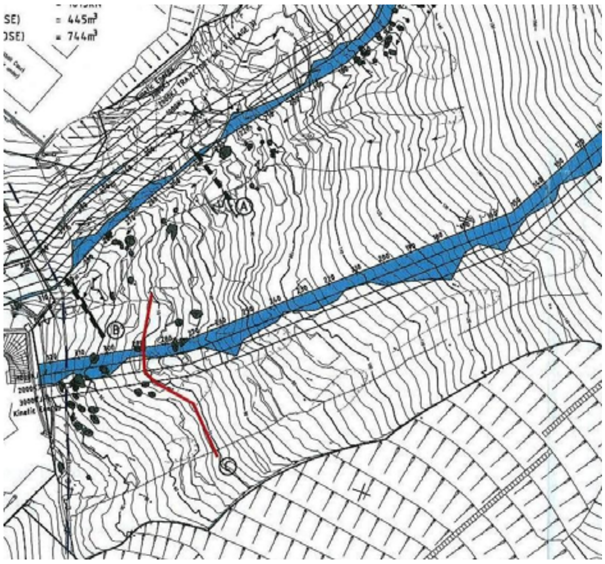


Table 8: Locations of major protective measures

8 CONCLUSIONS

The study was based on extensive information search and API, detailed fieldwork, and rigorous hazard assessments, which indicated that the terrain was prone to debris avalanching and boulder instability. Due to the many inherent inaccuracies associated with hazard modeling, detailed field mapping supplemented with adequate GI works and adoption of reasonably conservative parameters from various modeling tools should be sine qua non for a practical assessment of potential natural terrain hazards. Often, a cost-effective solution is possible if the overall hazard evaluation is underpinned by selected local treatment adopting both preventive and protective approaches.

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Mitigation Measures for Unstable Natural Terrain Adjacent to a Housing Development, Kwun Lung Lau, Kennedy Town, Hong Kong

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ABSTRACT

Due to the limited space available for construction on Hong Kong Island, a multi-storey housing development was proposed to be located in close proximity to the northwest flank of Mount Davis, west Hong Kong Island. This densely vegetated natural hillside overlooking the proposed development rises at an average gradient of about 30° reaching the height of 224m from the site formation level. The GEO's natural terrain landslide inventory has records of past instability within this hillside. Following an extensive field mapping (supplemented by ground investigation works) and hazard assessment, the potential hazards posed by the natural hillside to the proposed development comprised open hillside landslide, channelized debris flow and boulder instability. This paper outlines the mitigation measures considered which included two concrete check dams, a flexible barrier, as well as in situ stabilization using wire mesh, concrete matrix and concrete buttresses. Boulder removal was also undertaken. Due to the inherent inexactitude of natural terrain hazard assessment, it is important that mitigation measures should be undertaken only after a comprehensive field mapping and detailed assessment of identified potential hazards.

1 INTRODUCTION

The Hong Kong government typically imposes stringent regulations to investigate, assess and mitigate natural terrain hazards potentially impacting new developments (Ng et al. 2002). As a result, the recently completed redevelopment at Kwun Lung Lau could only commence after the initial natural terrain hazard assessment concluded that identified potential hazards from the overlooking hillside could be mitigated. Following an extensive field mapping (supplemented by ground investigation works) and detailed assessment of potential hazards from the overlooking hillside, preventive mitigation measures, comprising wire mesh fixing, concrete infilling, and concrete buttressing of boulders were provided in addition to the protective measures including two check dams and a flexible barrier. The hazard assessments leading to this included investigating the potential for open hillslope landslides, channelised debris flows and boulder falls (Onuselogu et al. 2008). Given the sloping ground profile and the dense vegetation cover, access was highly restricted requiring the use of helicopters in transporting ground investigation equipment and some construction materials. Other than access difficulties, high groundwater level at the lower levels of the hillside was particularly problematic during tieback installation.

2 BACKGROUND

The boundaries of the northeastern flank of the Mount Davis within which instabilities could pose a threat to the proposed residential tower were delineated and agreed with the Geotechnical Engineering Office (GEO). Past instabilities in the form of landslides have been recorded within this portion of Mount Davis, which has an area of 5.2 ha. Some regional studies including the GEO's Debris Avalanche Susceptibility Study categorized the site as having "highly susceptibility" to debris avalanching. The GEO's Geotechnical Land Use Map (GLUM) states that the site has extreme geotechnical limitations and is considered to be unsuitable for development. In 1994, two rainfall related failures occurred at the upper portion of the site. Adjacent to the proposed development, another failure which occurred on 23 July 1994 resulted in five fatalities with three persons injured. Against this background, and the stringent requirement by the Hong Kong government for natural terrain hazard assessment, the Hong Kong Housing Society proposed to redevelop the Kwun Lung Lau Estate. Plates 1 to 3 show the natural hillside and a broad timeline in the redevelopment.

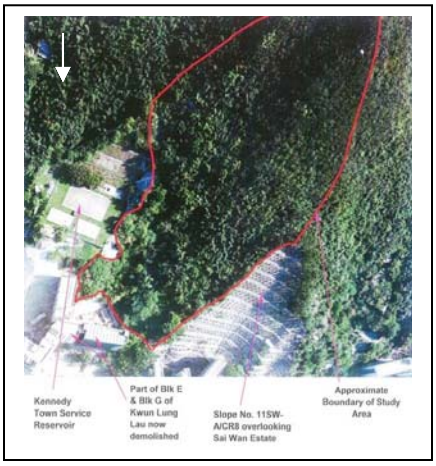

Plate 1: Site Boundary


Plate 2: Site formation works at toe

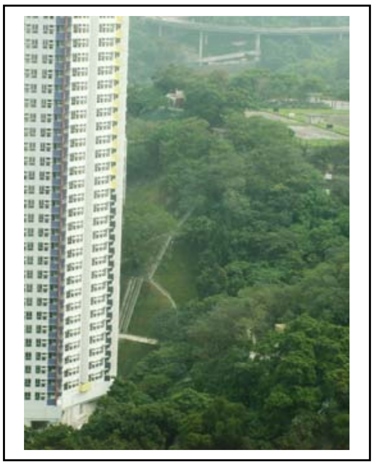
Plate 3: Completed building at toe

3 DATA SEARCH AND PRE-CONSTRUCTION FIELD WORKS

The implementation of the mitigation works was preceded by a comprehensive information search in form of both desk studies and aerial photograph interpretation (API). Debris flow deposits and landslides were observed in the 1963 aerial photographs of the site, one of which is shown as Plate 4. Ground investigation works (GI) revealed localized fill over colluvium, which covers the majority of the terrain, and saprolites derived from the underlying volcanics of the Mount Davis Formation of the Repulse Bay Volcanic Group (lapilli ash tuff with eutaxite). Extensive field mapping identified 39 landslides including 16 depressions covered with dense vegetation (relict landslides) and 23 multiple minor failures. The observed landslide scar widths measured from 3 to 12m, the depths rarely exceeded 1.5m. Five of the landslides were channelized, and perennial seepage over the lowest 50m of the terrain was observed along the main stream course, possibly due to a constant recharge from the Kennedy Town Freshwater Service Reservoir situated above that level.

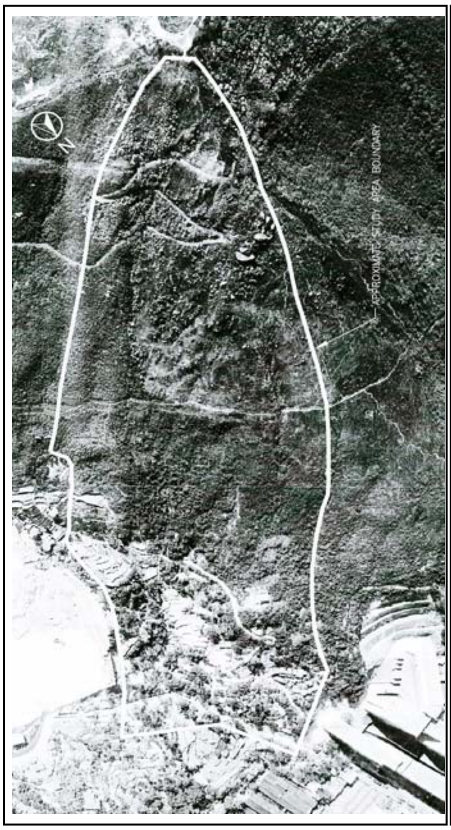
Plate 4: Aerial photo of site (1963)

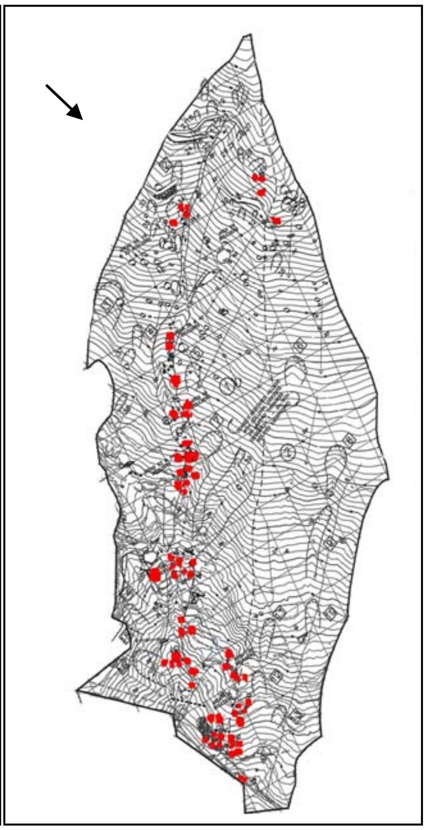
Plate 5: Location of Boulders and Landslides

Boulder survey confirmed the boulder concentrations identified in the API and revealed the presence of 207 boulders with at least one dimension greater than 1m. Of these, 98 boulders were observed to be potentially unstable out of which 30 boulders were rounded in shape. Based on their exposed dimensions, about 61% of all observed boulders were less than $2m^3$ in volume while about 4% had volumes in excess of $20m^3$. In general, about 48% of all the potentially unstable boulders were observed at the lower third portion of the site, whilst 40% and 12% were located at the middle third and upper third portions respectively. The locations of these boulders and landslides are indicated in Plate 5 (with locations of the potentially unstable boulders highlighted in red).

4 MITIGATION MEASURES

Preventive mitigation measures prescribed for the site were mostly against boulder falls and included wire meshing, buttressing over-hanging boulders and concrete infilling within boulder units (concrete matrix). These are shown in Figure 1 and Plate 6.

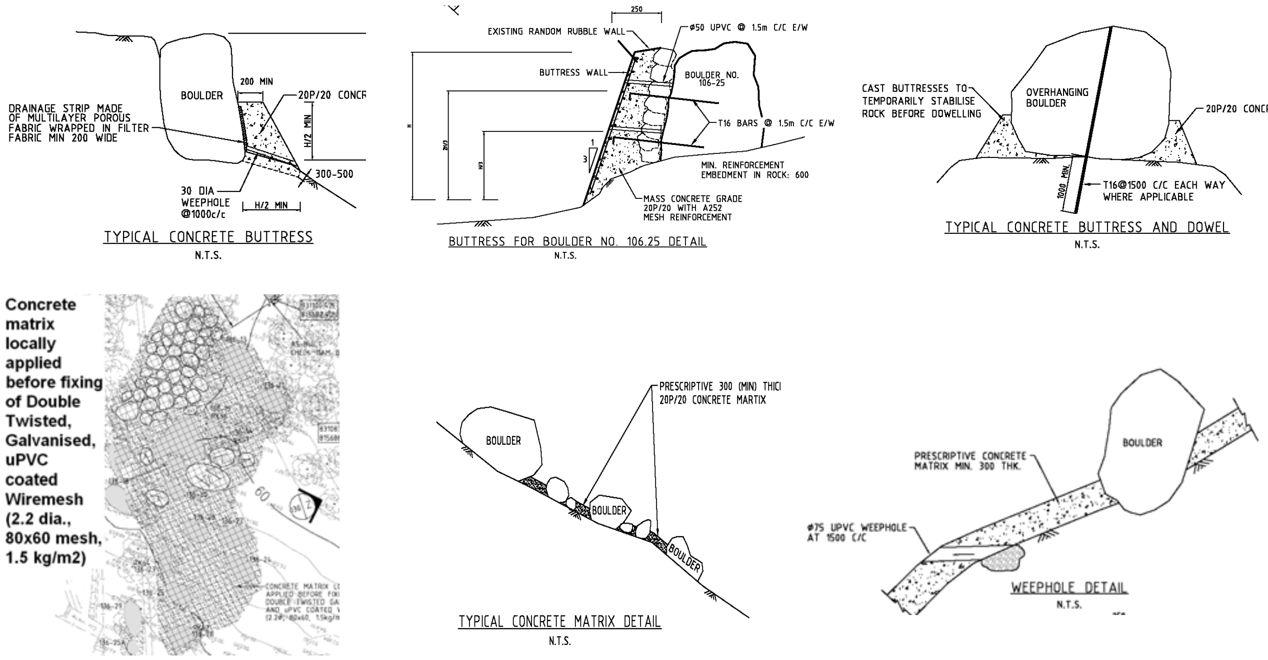

Figure 1: Preventive mitigation measures (in-situ stabilization of boulders)

Plate 6: Preventive mitigation measures (in situ stabilization of boulders – buttressing, concrete matrix and wire meshing)

It was uneconomic to stabilize all potentially unstable boulders. The boulder selection algorithm is such that only boulders larger than 1m in any one dimension were recorded during the boulder survey (Onuselogu et al. 2003). With respect to stabilization, only the boulders larger than 2 m^3 in volume were considered for in situ stabilization as this would minimize the required capacity of the protective mitigation measures. In all, 70 in situ stabilization works were carried out consisting mostly of buttressing and binding boulder units with concrete infill (concrete matrix).

Two concrete check dams and one flexible barrier were constructed and installed at the locations considered to be most cost-effective with respect to their required capacity as determined during debris characterization and boulder fall analysis (Onuselogu et al. 2003). Their dimensions were based on the design event (1,000 m^3) determined for the site, the estimated maximum boulder bounce height and the calculated maximum debris run-up distance. Both concrete check dams are 5.6m high, 1.2m thick at the top and 1.4m thick at the base. The upper check dam is 20m long and the lower check dam is 24m long. Both check dams have openings large enough to accommodate floodwater discharge as well as reduce debris impact on them in the event of a failure.

As shown in Figure 2, four rows of tiebacks comprising T40 steel bars and Grade 30 grout were installed in 150 mm diameter holes. The upper 2 rows of tiebacks were 25m long whilst the lower two rows were 23m long. Altogether 45 tiebacks were installed excluding 10 trial tiebacks. The tiebacks were installed at a vertical spacing of 1 m. To avoid clashing, they were declined at 24°, 27°, 33° and 36°, going from the uppermost row to the bottommost row. There were no incidents of hole collapse observed during the installation. To minimize erosion impact on the wing walls and the stems of the check dams, about 1.5m wide concrete apron was provided as shown Plate 7.

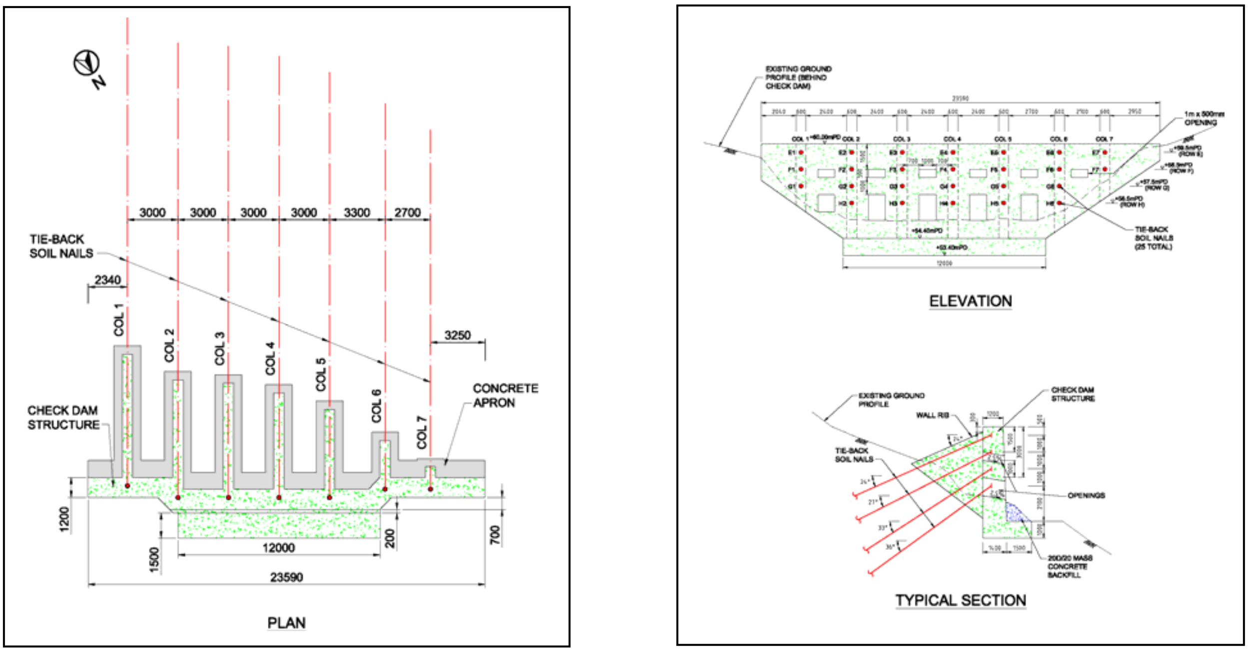

Figure 2: Protective mitigation measures (concrete check dam tied back with 23 – 25m long nails within a stream course)

Plate 7: Preventive mitigation measures (concrete check dam with tiebacks concealed in the concrete wing walls)

The installed flexible barrier is Geobrugg RX 075 having a capacity of 750 kJ. It is 3m high and 69m long. The barrier primarily consisted of 9 posts (installed at a spacing varying between 7m to 10m to avoid clashing with existing mature trees) and 300mm dia. ring nets. As shown in Figure 3, the flexible barrier system was supported by anchor blocks and 17m long ground anchors.

At a potential debris angle of repose of 10°, the upper and lower check dams and the flexible barrier would create a debris containment with estimated capacities of 500m³, 800m³ and 760m³ respectively. Figure 2 and Plate 7 give more details.

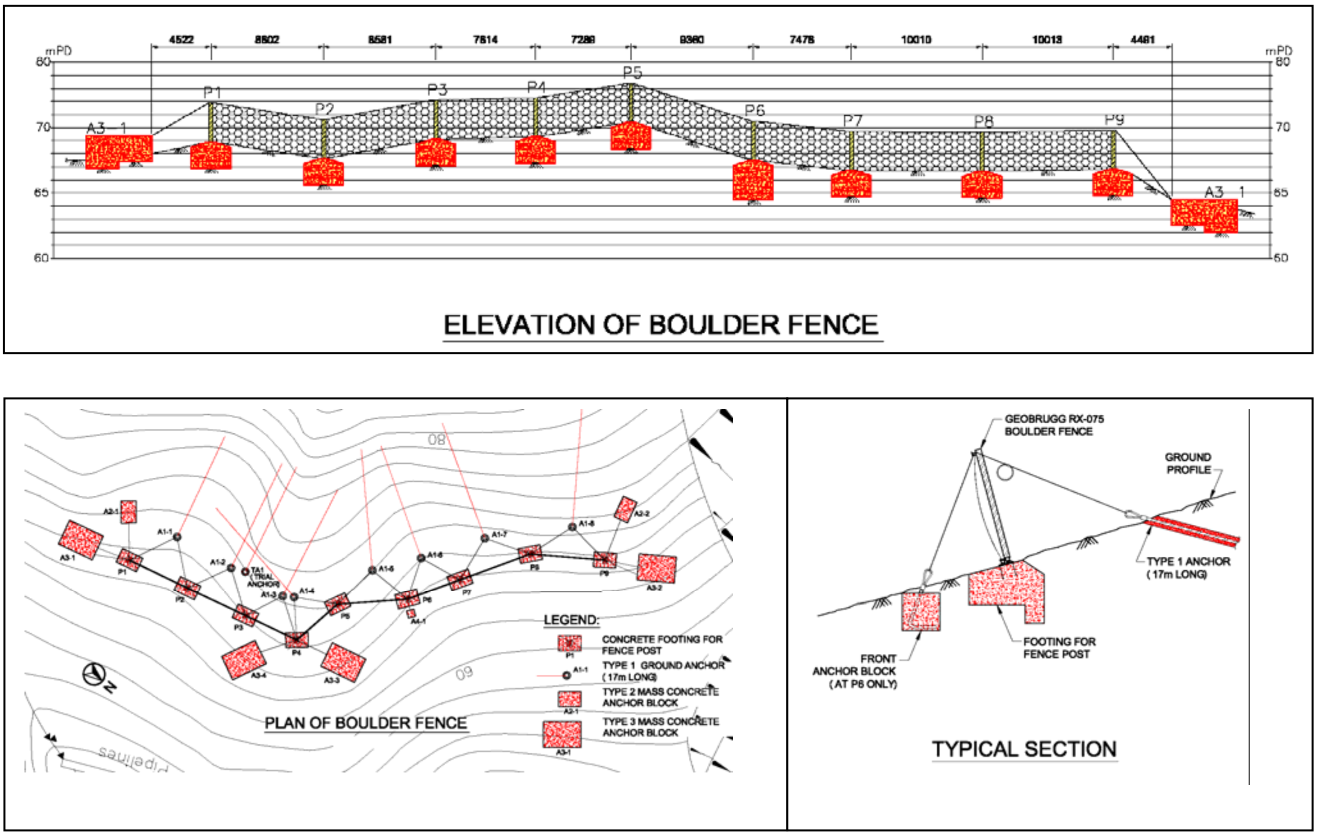

Figure 3: Protective mitigation measures (750 kJ flexible barrier – 3 m high, 69 m long)

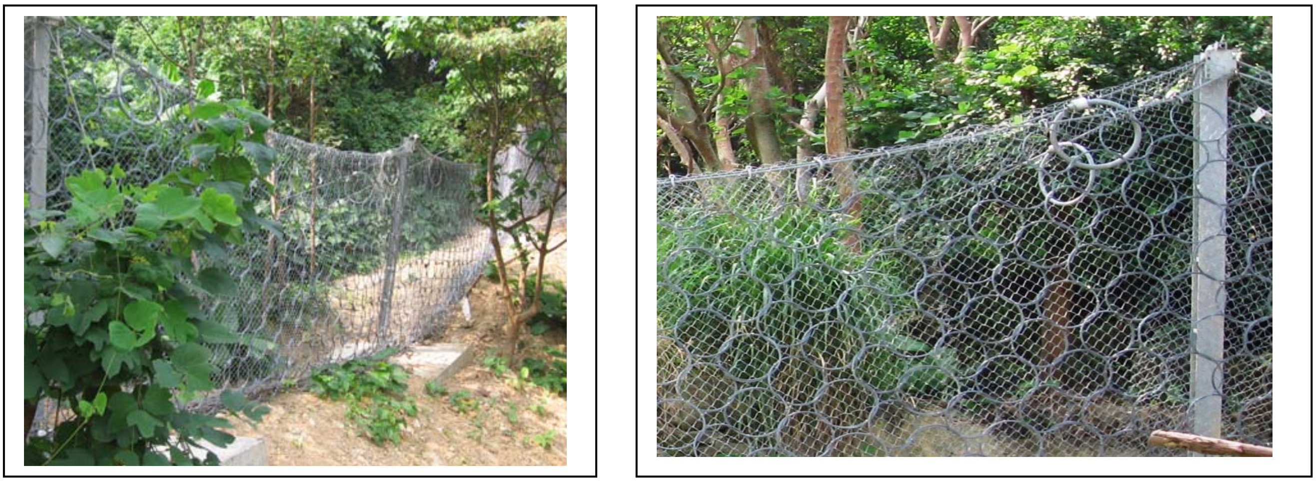
Plate 8: Preventive mitigation measures (flexible barrier with a layer of expanded metal to retain debris)

5 SITE CONSTRAINTS

The constraints encountered during the field works are summarized below:

Access - The obvious constraint during the pre-construction field works (ground investigation works, field mapping and boulder survey) was the limited access. The site is 224m high and has an average slope gradient of 30°. Neither the toe area nor the peak was directly accessible to vehicular traffic. Therefore, helicopters were used to transport the ground investigation equipment. During the construction stage, a tower crane located at the toe area of the site was used to transport the heavy equipment particularly the backhoe, the air compressor as well as the reinforcing bars. Concreting of the check dams was achieved through a concrete truck at the toe area. The construction of concrete buttresses and concrete infilling of boulder units were achieved using site mixed concrete, materials of which had to be carried upslope manually.

Construction Space - Dense vegetation cover of the site limited the space available for construction. Although there were gaps under the canopies of the mature trees, the sloping topography made construction personnel movement relatively difficult. Tree felling was not allowed so even when equipment transportation was undertaken by helicopters and tower cranes, there was still substantial manual handling as the “unloading bays” existed only at places where gaps existed within the tree canopies.

Disused Wells - There were anthropogenic features that posed major health and safety risks to the field mapping teams. The site was previously used for cultivation and this resulted in several shallow pits in the bushes but the presence of disused wells, which sometimes exceeded 3m in depth and containing water, was particularly troubling to the investigation teams during field mapping. These disused wells were backfilled during the construction as they could pose major health hazard to maintenance personnel.

High Groundwater Level - A bouldery perennial stream course is at the lower level of the site. Seepage was evident at various points near the toe of the site. As high groundwater level was considered problematic for the installation of check dam tiebacks, dewatering was originally intended. However, as ground movement in excess of 25mm was estimated for the Kennedy Town Service Reservoir due to the proposed dewatering, other alternatives were considered. The tiebacks were successfully installed through staged grouting, and their integrity was tested through additional trial and pull out tests. This method is encouraged in *Geoguide 7*.

Utility - A 600mm diameter pipe connected to the Kennedy Town Service Reservoir is present at the toe of the site where boulder stabilization works have been prescribed. Damage to this pressurized freshwater pipe will disrupt water supply to Kennedy Town.

Wildlife - Encounters with wildlife particularly snakes, created some anxiety among both the investigation team and the construction personnel. Due to Hong Kong government’s renewed warning on dengue fever and Japanese encephalitis, mosquito bites led to many instances of workers complaints.

6 CONCLUSION

The mitigation measures implemented on the natural terrain site were as a result of a comprehensive information search and detailed hazard assessment. They consisted of both the preventive and protective works that gave confidence that the proposed building would not be adversely affected by the hazards from the site. To ensure adequate maintenance, access footpath was formed and maintenance manuals prepared. All the mitigation measures have been registered by the GEO in accordance with the statutory requirements.

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The authors would like to thank the Hong Kong Housing Society for their consent to publish this paper. The views expressed in this paper are those of the authors and not of any other parties.

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Engineering Geological Assessment of the Natural Terrain Risks at Halong Bay, North Vietnam

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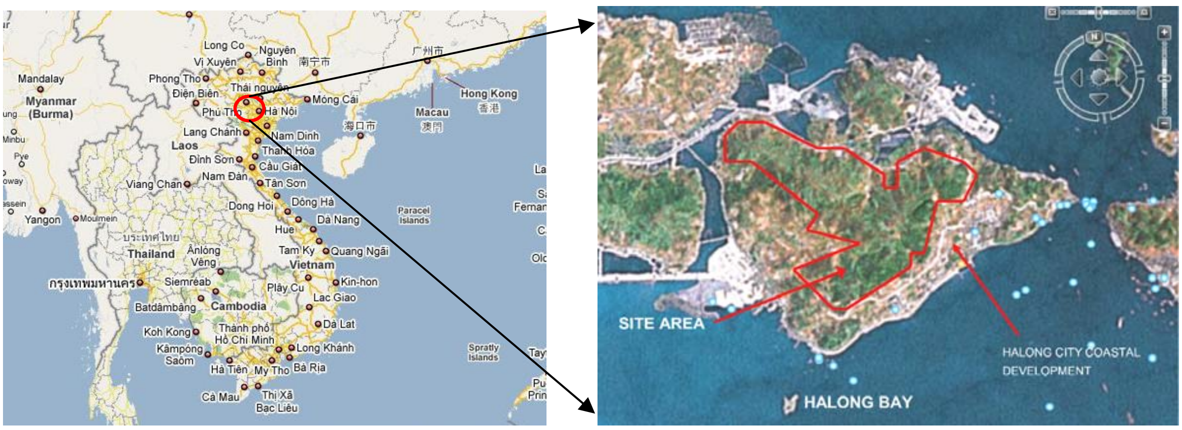
ABSTRACT

Halong Bay, north Vietnam is known for its drastic karstic topography forming island chains extending from the coastline into the bay. The islands have a unique geomorphology, rising precipitously from the sea, providing natural beauty and a scenic backdrop along the coast. As a result the area was designated a UNESCO World Heritage site. In contrast to this Bai Chai Island located across the bay to the north has a rolling topography rising gently from the coastline to levels overlooking the bay. Due to the rising demand to visit this area and the spectacular views provided from Bai Chai Island there are presently plans to develop high level resorts in this area. During a recent site investigation it was revealed that the site area has complicated geology with challenging geotechnical issues. To the north of the site the geology typically comprises weakly cemented residual soils (laterite), which become unstable when exposed to surface or subsurface water flow. The southern portion is dominated by coal measures overlain by colluvium, which has terrain stability controlled by its bedding. This paper outlines the different approaches adopted for the site investigation, which due to the early input required for design, assessed the past environmental and geological processes to derive the ground's physical properties. It is considered that the findings provided confidence with the geotechnical engineering solutions obtained and also emphasizes how a good walkover survey and desk study, with a particular emphasis on understanding the geology, can provide a relatively cheap and rapid method of site investigation.

1 INTRODUCTION

The "Dragon's Teeth", extending many kilometers offshore from the Vietnamese coastline, are formed from karst topography rising abruptly from the water surface to maximum elevations of about 200m above sea-level, providing spectacular scenery for visitors to admire. In contrast the Bai Chi Island has been offset from the karstic landscape of the Halong Bay by a fault running immediately offshore allowing the very different geology of coal measures and sandstones to be located in close proximity to the karst topography. Due to the different physical properties and the demand to accelerate development rapid feedback of data was required to commence the geotechnical design. An initial engineering geological appraisal was therefore carried out in advance.



Figures 1 and 2: General location of the Study Area (from Google Earth)

2 SITE SETTING

The Bai Chai Island development area is situated within the Bai Chi River delta, north east Vietnam. It is bounded by the Halong City coastal development to the south and areas of local settlements and cultivation to the north east and west. The coastline to the south is a low lying mud flat extending from the Ba Che River to the precipitous Peaks of the Cat Ba Island further offshore. The terrain to the west comprises undulating gently dipping terrain extending from the Haong Lien Mountain Chain.

The site has a maximum elevation of about +165m above sea level forming the highest point of an approximate north east to south west trending ridgeline. Dip slopes fall from this ridgeline at gentler and consistent gradients to the north and north-west and steeper scarp slopes fall to the south and south east. Many ephemeral stream courses run across the terrain generally flowing to the south-west. The area is covered by forest, mainly bamboo, locally removed for cultivation and settlement. More recently a large portion of the site has been altered by earthworks activities.

3 LITERATURE SEARCH

Due to Vietnam's recent past history, limited literature, aerial photographs and clear images from "Google Earth" are unavailable. Data was therefore restricted to topographic records and geological maps and publications. The 1:200,000 scale "geology and mineral resources map" (Vietnam Government 2001), showed productive coal extraction, to the NE of the site, and the site to be underlain by the Dong Ho Formation (conglomerate), coal measures (anthracite, siltstone and sandstone) and chert siltstone and limestone. An approximate WNW to ESE trending fault also traverses the central portion of the site. A geological study of the area (Waltham 2005), noted a fault, trending south west to north east, immediately south of the island and that the bedding dipped towards the west with a structure complicated by north and east south west trending faults. A morphotectonic map (Lung 2008), showed the site to be bounded by extensional rifting to the north and major east west trending faults to the north and south and "instrumental earthquakes" identified along the fault lines east and west of the site.

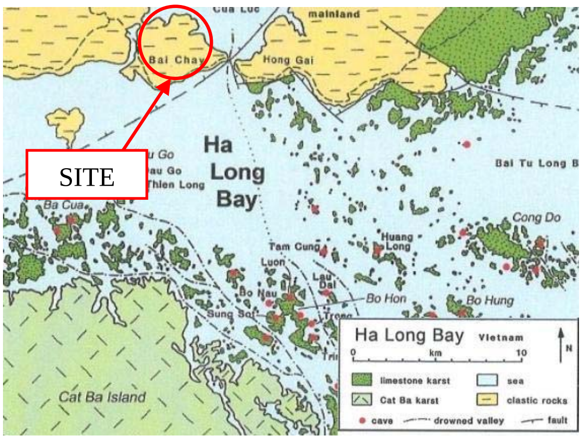


Figure 3: Outline geology of Halong Bay (Waltham 2005)

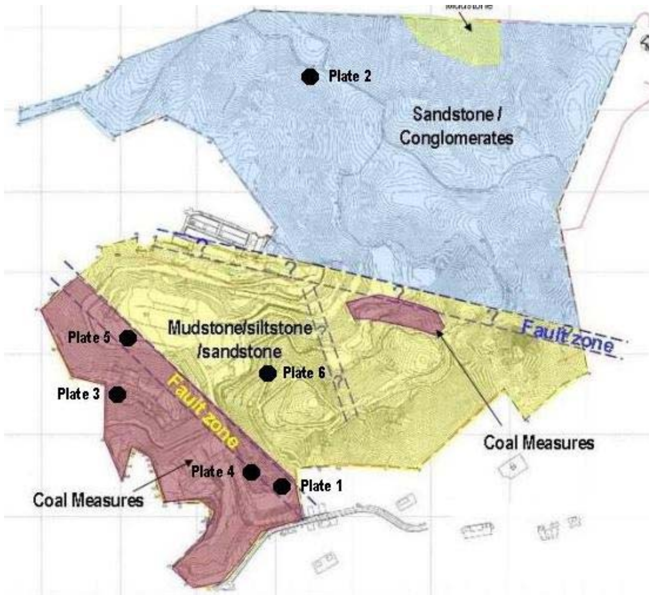


Figure 4: Ground conditions encountered

4 SITE INSPECTION

Due to the limited time and data available a site inspection was carried out to assess the engineering geology following the guidance given by Baynes et al. (2005), Fookes & Baynes (2008) and Mackay and Nord (2008). Based on this the ground was categorized into different "reference conditions" with unique physical properties derived from their past geological and environmental history. Generally the site investigation confirmed the findings of the available geological literature revealing the presence of coal measures (carbonaceous siltstone and sandstone); fault zones (swelling clay and cataclasite) and sedimentary rock (conglomerate, sandstone, siltstone and claystone). Refer to Figure 4 for the interpreted locations.

The near surface ground had either been weathered to varying degrees, altered by earthworks activities or covered by transported material (colluvium). Generally the colluvium covered the coal measures and mudstone, siltstone and sandstone south of the main WNW to ESE trending fault zone and laterite (cemented residual soil derived from weathering) covered the site north of the fault zone. Earthworks excavation and fill deposition had taken place locally over the site and the fault zones traversing the site formed topographic depressions due to their susceptibility to weathering and erosion upon exposure. A summary of the reference conditions assessed, their physical parameter and miscellaneous observations made during the site inspection are presented in Table 1 and examples of the reference conditions in Plates 1 to 4 below.

Table 1: Ground conditions encountered

Material type	*Reference Condition (1)*	*Reference Condition (1)*	*Physical Parameters (temp.) / stability control*	*Physical Parameters (Perm.) / stability control*	*Remarks*
Superficial	Fill		Phi = 25 - 30, Dens. = 18 – 19 kN/m^3 (soil)	Phi = 25 - 30, Dens. = 18 – 19 kN/m^3 (soil)	Unsuitable materials used, cavitatious, prone to catastrophic collapse.
	Colluvium	Cemented	125kPa (weak rock)	Phi = 30 - 35, Dens. = 18 – 20 kN/m^3 (soil)	Changes from rock to soil through groundwater leaching (covers the majority of the site)
		Uncemented	Phi = 30 - 35, Dens. = 18 – 20 kN/m^3 (soil)	Phi = 30 - 35, Dens. = 18 – 20 kN/m^3 (soil)	Has already undergone leaching, possibly from anthropogenic influences (covers small portion of site).
In-situ (weathered)	Laterite		125kPa (weak rock)	Phi = 30, Dens. = 19 – 20 kN/m^3 (soil)	Changes from rock to residual soil through groundwater leaching (covers the majority of the site)
	Residual Soil		Phi = 30, Dens. = 19 – 20 kN/m^3 (soil)	Phi = 30, Dens. = 19 – 20 kN/m^3 (soil)	Usually fine grained and associated with siltstone or fault zones.
	Partially Weathered Rock	30 / 70	Phi = 30, D ens. = 19 – 20 kN/m^3 (soil)	Phi = 30, Dens. = 19 – 20 kN/m^3 (soil)	Stability is controlled by the soil
		10 / 90	Phi = 30, Dens. = 20 kN/m^3 (rock, discontinuity control)	Phi = 30, Dens. = 20 kN/m^3 (rock, discontinuity control)	Stability is controlled by the rock discontinuities
In-site (un-weathered)	Coal Measures	Bedding adverse	Phi = 25 - 30, Dens. 20 kN/m^3	Phi = 25 - 30, Dens. 20 kN/m^3	Stability is controlled by the discontinuities
		Bedding Favourable	125kPa	125kPa	Stability is controlled by the rock mass
	Swelling Clay		C=20, Shrinkage about 20%	C=20, Shrinkage about 20%	Highly adverse, physical properties alter
	Sedimentary Rocks	Conglomerate / Sandstone	250kPa	250kPa	Very strong material
		Siltstone / Claystone	125kPa	125kPa	Good material / bedding controlled stability

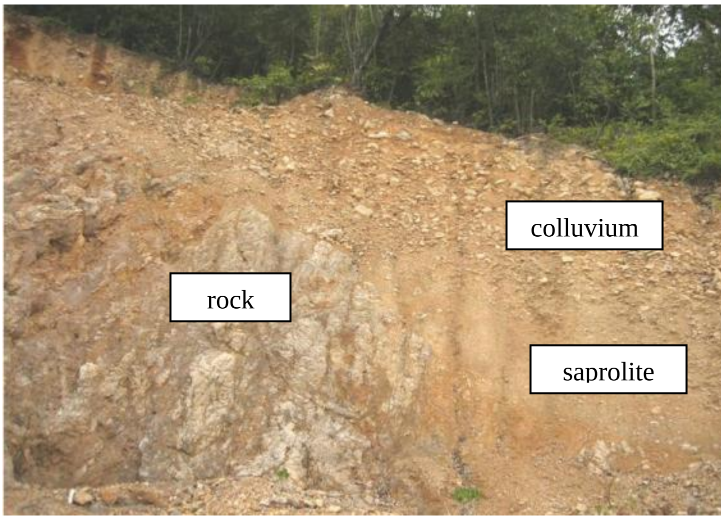


Plate 1: Colluvium

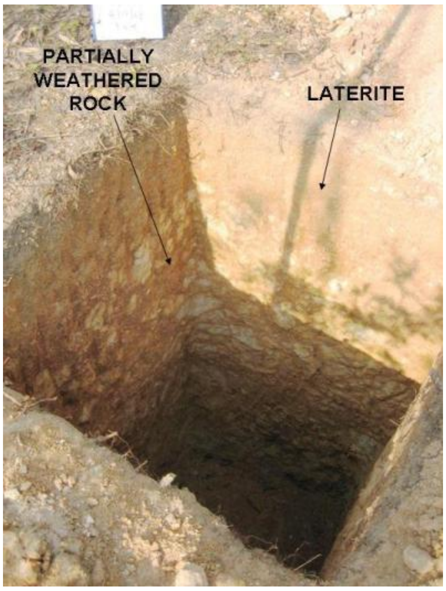


Plate 2: Partially weathered rock and laterite

Plate 3: Sedimentary rock

Plate 4: Coal measures

5 INTERPRETATION

The sub-tropical weathering effects, similar to that of North Vietnam, are shown in Figure 5.

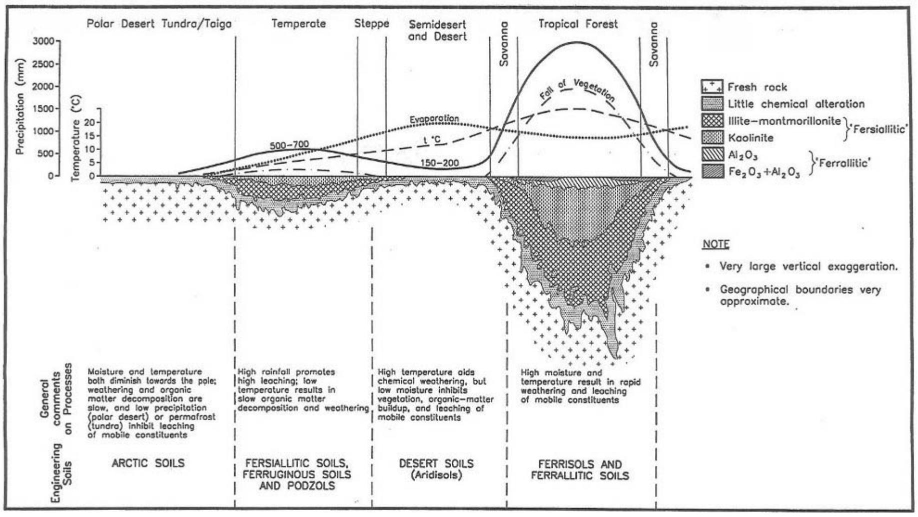


Figure 5: Tropical Weathering characteristics (Fookes 1997)

Features observed during the site investigation included the differential weathering effects resulting from steeply dipping faults and bedding (Plate 5 and Figure 6) and the sensitivity of the ground to leaching, occurring from variations in groundwater and surface water flow (Plates 5 and 6 and Figure 7).

Plate 5: Fault zone with differential weathering

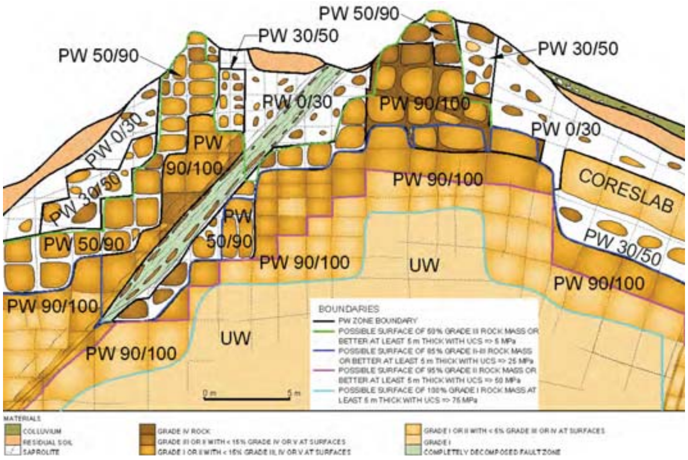


Figure 6: Schematic presentation of differential weathering (GEO 2007)

Plate 6: Soil piping

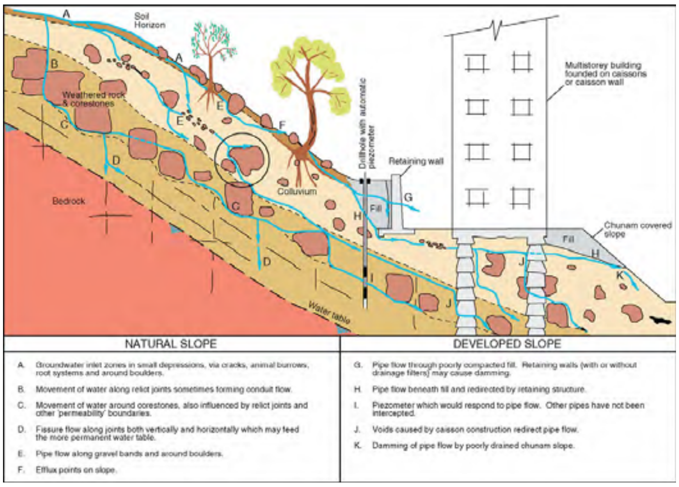


Figure 7: Schematic presentation of soil piping (GEO 2007)

As both the laterite and colluvium were located near surface over the majority of the site having the greatest impact to the development, it was considered vital to understand their past geological and environmental history in order to estimate geotechnical parameters. The findings from the inspection are summarized in Table 2.

Table 2: Summary of observations from the laterite and colluviums

Reference Conditions	*Description*	*Past environmental and geological history*	*Interpretation*	*Physical Considerations*
Colluvium	Sub-angular gravel to cobble sized rock fragments in a weakly cemented sand / silt matrix.	Change in climate from temperate (mechanical weathering effects), to sub-tropical (chemical weathering effects), following the last ice age, 20,000 years previously (GEO 2007).	The sub-angular rock fragments were derived from the mechanical weathering and fine grained weakly cemented matrix from chemical weathering	The material is stable in the temporary condition becoming unstable following groundwater leaching (See Figure 5).
Laterite	Silt and clay residual soil bound by ferruginous (reddish brown) cement.	The sea level has risen by about 120m elevating the groundwater level since the last ice age. Rainfall has also become intense and frequent, typical of a sub-tropical climate.	The laterite was formed when the groundwater table was lower allowing cement precipitation cement. As the groundwater lebvel rose, cement is leached causing destabilization.	The material is stable in the temporary condition becoming unstable following groundwater leaching (See Figure 5).

6 CONCLUSIONS

For any site with geotechnical risks it is imperative that these are understood and managed effectively from the outset. The site investigation method adopted to obtain data from a relatively remote location rapidly was considered to be highly effective. The site investigation also allowed engineering considerations to be made at an early stage, which due to the sensitive nature of the ground and its susceptibility to groundwater flow, was considered to be conventional, robust engineering techniques for slope support such as retaining structure.

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Application of Flexible Rock Fall Protection Barrier at Repulse Bay Road, Hong Kong

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ABSTRACT

Natural terrain hazard mitigation has become one of the major components of slope projects in Hong Kong. A slope identified to have potential rock fall problems was selected for upgrading under a Landslip Preventive Measures Contract. The slope, which consists of a rock fill man-made slope and natural hillside, is located immediately below a strategic route, Repulse Bay Road on Hong Kong Island. It has a maximum height of 20 m comprising a 1 to 2 m thick rock fill layer overlying moderately decomposed tuff. In view of the substantial amount of cobbles and boulders identified on the slope surface, traditional stabilization measures like buttressing, dowelling and boulder removal were not considered practical. A permanent high energy flexible rock fall protection barrier was adopted. This paper describes the application of the permanent high energy flexible rock fall protection barrier at the feature and possible future development for the design, construction and maintenance of permanent flexible rock fall protection barriers as an effective protection measure against natural hazard related to potential rock falls.

1 INTRODUCTION

As part of the ongoing Landslip Preventive Measures (LPM) Project, Ove Arup & Partners Hong Kong Ltd. were commissioned by the Geotechnical Engineering Office (GEO) of the Civil Engineering and Development Department (CEDD) to undertake the investigation, design and construction of Landslip Preventive Works for 60 numbers of government slope features, located on Hong Kong Island.

One of the features, Feature No. 15NE-A/F199 at Repulse Bay Road, was identified to have potential rock fall instability. Taking into account the substantial amount of cobbles and boulders identified on the slope surface, a permanent high energy flexible rock fall protection barrier at the slope toe was chosen in preference to the traditional stabilization measures such as buttressing, dowelling and boulder removal. The works were completed by a specialist contractor in 2008. This paper describes the application of the permanent high energy flexible rock fall protection barrier and its possible future development throughout various stages.

2 THE SLOPE FEATURE

2.1 Site description

The slope feature 15NE-A/F199 is located immediately below Repulse Bay Road, which is a strategic trunk road built before 1945 as a link between the southern and northern parts of Hong Kong Island. It consists of a rock fill man-made slope at the upper portion and natural hillside with an ephemeral stream passing through the slope. It has a maximum height of 20m with an average gradient of 35^o, comprising an upper 15m cobble-sized rock fill surface at the man-made portion and lower 5m soil, densely vegetated zone at the natural terrain portion. Its overall length is about 30m. A 2m wide public footpath and a high-rise residential development, namely Grand Garden, are located approximately 10m and 80m below its toe respectively, see Figure 1.

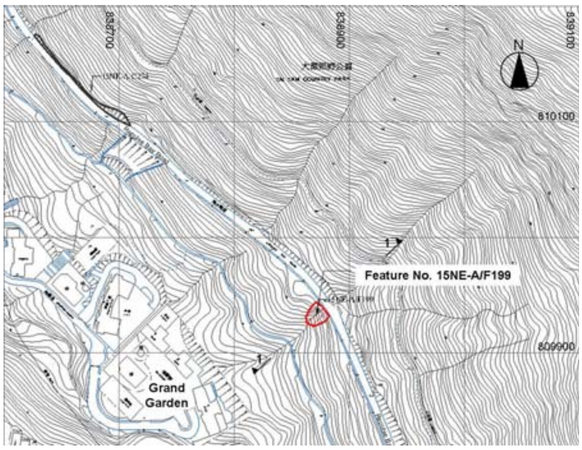

Figure 1: Site Location Plan

2.2 Geology

Based on the results of ground investigation and field testing (Arup 2007), the geology of the feature is generally a 1 to 2m thick rock fill layer comprising angular cobbles and boulders, overlying moderately decomposed tuff (MDT). This is consistent with the findings obtained from the desk study, aerial photo interpretation and site inspection. No significant fault or shear zone is observed to run across the site. A geological section with inferred geological information is shown in Figure 2.

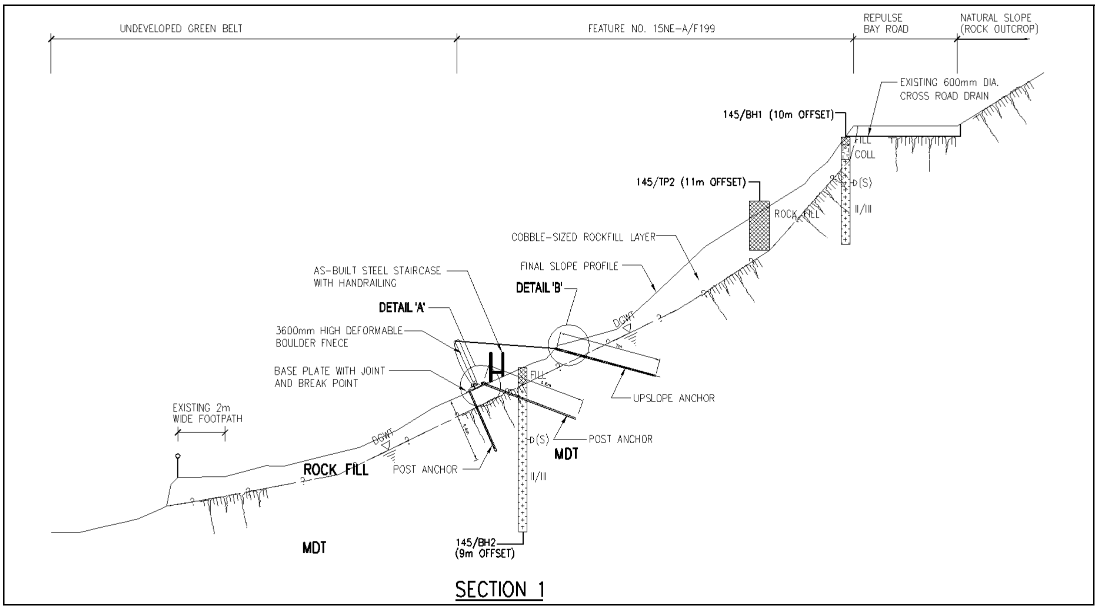

Figure 2: Geological section

Based on the site investigation, a set of geotechnical parameters were derived for the LPM design. The adopted parameters in Table 1 were assessed from field testing and comparison with Hong Kong typical values.

Table 1: Summary of geotechnical design parameters

Stratum	Unit Weight, γ (kN/m^3)	Cohesion, c' (kPa)	Friction Angle, ϕ' (Deg.)
Rock Fill	22	0	40
MDT	26	15	40

2.3 Groundwater

For the slope stability analysis, a single groundwater regime exists with groundwater level at the interface of the rock fill layer and the bedrock due to the highly permeable nature of the rock fill layer. Continuous groundwater monitoring on two standpipes with Halcrow buckets installed also indicates consistent groundwater table. As no existing surface or sub-surface drainage system was identified within the feature boundary, rainfall over the feature and surface runoff from upstream are expected to drain rapidly as subsurface underflow through the rock fill layer.

3 MITIGATION MEASURE DESIGN

In accordance with current practice (GEO 2003; WB 1999), a consequence-to-life category of "2" and economic consequence of "B" are designated for the feature due to Repulse Bay Road with moderate traffic density and footpath abutting the crest and toe of the feature respectively. Two major modes of failure, i.e. overall slope stability and potential rock falls, have been considered in the assessment.

The geotechnical computer program, *Oasys* Slope, was employed to evaluate the overall stability of the existing feature. Janbu's Method with variably-inclined interslice shear force was adopted. Stability analyses were carried out on the most critical scenario in terms of slope geometry, groundwater condition, geological profile and surcharge condition. The results indicate that all potential failure slips meet the current geotechnical standard of required minimum "Factor of Safety" equal to 1.2 except a case of minor localized failure due to adverse geometry. To cater for possible failure against minor localized slope instability, local trimming was therefore proposed and carried out at the construction stage.

Due to the presence of the substantial amount of unstable cobbles and boulders on the slope surface, mitigation measures are necessary to reduce the likelihood of rock fall hazard and associated threat to public safety. Having reviewed the latest technology available and experience gained through past applications both in Hong Kong and overseas (Lo 2000), a permanent high energy flexible rock fall protection barrier to be installed at the slope toe was adopted as a mitigation measure against the potential rock falls. In line with the observations from the site inspection and ground investigation (Arup 2007), relevant criteria for the rock fall protection barrier design were determined with reference to current local practice (Lo 2000; Sun & Lam 2006), see Table 2.

Figures 3 and 4 show an aerial view of the feature surroundings and typical layout of rock fall protection barrier respectively.

Table 2: Design criteria for rock fall protection barrier

Maximum Rock Mass	3000kg
Diameter of Boulder	1.3m
Speed of Boulder	26m/s
Minimum Height of Net without Sag	3.6m
Maximum Spacing of Posts	10m
Energy Absorption Capacity	1000kJ

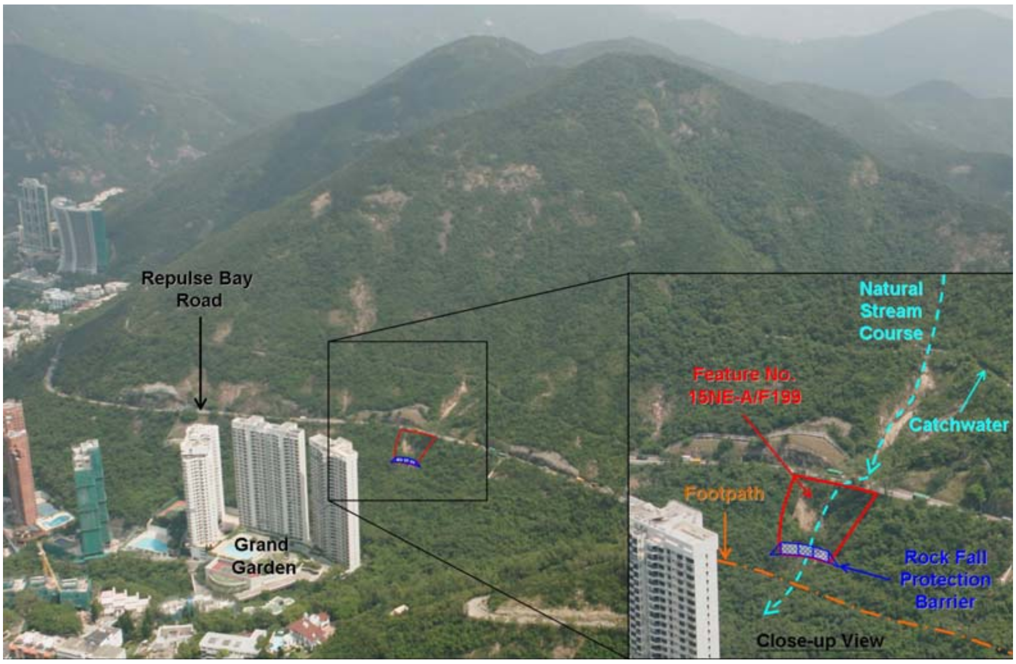

Figure 3: Aerial view of the feature surroundings

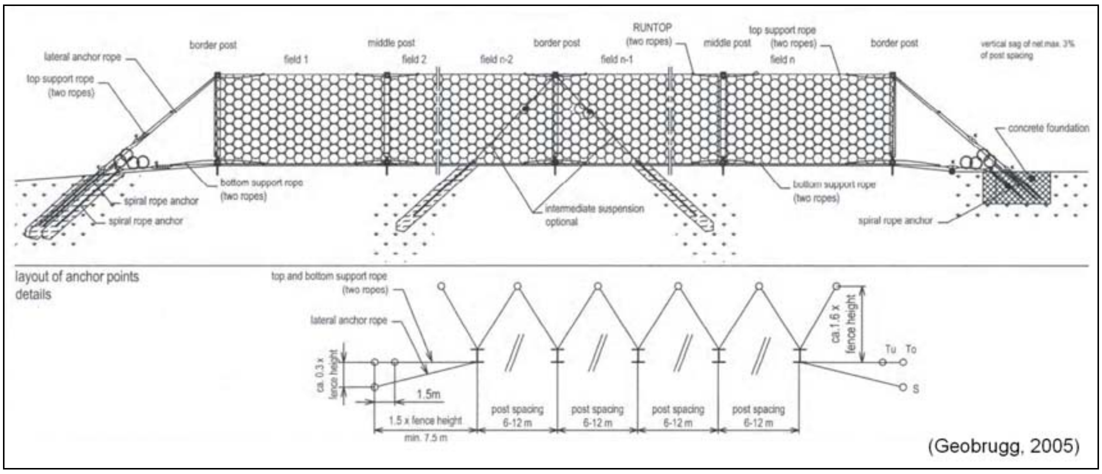

Figure 4: Typical layout of flexible rock fall protection barrier

3.1 Components

A proprietary flexible rock fall system was chosen and designed based on detailed investigation by the manufacturer's specialist. This system consists of steel wire ring net, brake rings, wire mesh, high-tensile ropes, hinged posts, support and upslope anchor ropes and foundations. It is designed to stop and retain the falling rocks in such load transfer mechanism that the energy from rock fall event is partially absorbed by the ring net, which is connected to the brake rings, and the remaining energy is transmitted through the posts and the ropes into the ground.

To protect the steel against corrosion, proper corrosion protection is necessary for each component of the barrier system. For the system adopted at Feature No. 15NE-A/F199, the steel posts, base plates and wire rope anchors are hot dip zinc galvanized while the wire mesh, wire ropes and ring net are hot dip with a zinc and aluminum coating. As proposed by the manufacturer's specialist, some essential but small and easily replaced component parts were sometimes produced in accordance with other production standards. For example, the wire rope clips are produced in accordance with DIN 1142 and the shackles are in accordance with the U.S. Federal Specification RR-C-271. In order to ensure the quality of the production of the critical components of the barrier system, relevant test reports and certificates of the quality assurance and acceptance tests related to the batch of the system components used are required. In general, the test reports and certificates provided should be issued on HOKLAS endorsement or equivalent to relevant international standards. In addition, corresponding types and quantity of acceptance tests on the material samples are required to demonstrate compliance of the main components of the barrier system with the material requirements as specified in local practice note (GEO 2007). Full-scale field tests on the barrier system to substantiate the impact energy capacity and the maximum allowable deformation are required. Sufficient time should be allowed for ordering, shipping and testing of a barrier system before fabrication on site.

4 CONSTRUCTION

There are two major construction phases for the proposed rock fall protection barrier, i.e. installation of anchors and foundation and fabrication of barrier components.

4.1 Installation of anchors and foundation

There are several installation methods for anchor installation, the selection is based on various factors such as cost, duration, safety, environment, site access, working space, ground conditions and groundwater regime. The drilling works are usually carried out by pneumatic drilling rig with down-the-hole hammer. The reinforcement bar or anchor rope is inserted into a pre-drilled hole, typically 100 to 150mm diameter, which is then in-filled with cement grout by using tremie method. Reinforced concrete foundation supporting the posts may be necessary when low bearing capacity is encountered at the post location. For that reason, extra care should be taken during selection of post locations.

Plate 1: Erecting hinged post

4.2 Fabrication of barrier components

To ensure the effectiveness of the rock fall protection barrier, sufficient numbers of hinged post are designed to support all major components, namely wire mesh, ring net, brake rings, high-tensile ropes, support and upslope anchor ropes, and hold them in correct positions. The hinge pins and post bases are specially designed not only to facilitate ease of post erection, but also to act as predetermined breaking points, which protect the anchorage from destruction in case the posts are directly impacted from falling objects.

Plate 2: Fixing brake rings

After erecting the post, it is fastened laterally and back to the ground with upslope anchor ropes by high-tensile ropes. The lateral anchor ropes and bottom support ropes are each equipped with brake rings, which are able to dissipate the residual energy out of the barrier. It is also important to use adequate wire rope clips to fasten the ropes. Lastly, a layer of ring nets together with wire mesh is mounted on the barrier. Shackles are used to fasten the ring nets to the bearing ropes. Typical installation sequence is shown in Plates 1 to 4.

Plate 3: Mounting ring net

Plate 4: Completed barrier and maintenance accessway

4.3 Maintenance

In order to ensure the designed service life of a flexible rock fall protection barrier, regular inspection, maintenance and repair works are required upon completion of the installation works. In most cases, little or even no maintenance is anticipated if the kinetic energy released in rock fall events does not exceed the designed load limits, which is guaranteed under a manufacturer's Warranty Certificate. However, the lifespan of some components are perhaps influenced by excessive rock fall events, atmospheric corrosion problem or combination of both. On occasion, reuse and recycle of components are allowed and encouraged by the manufacturer. Details of the works are illustrated in a comprehensive maintenance manual prepared by the manufacturer. In particular, the maintenance manual (Geobrugg 2006) should consist of lifespan of each component, inspection arrangement, inspection checklists, criteria for actions, assembly drawings and maintenance and repair procedures.

5 FUTURE DEVELOPMENT

5.1 Reviews on performance

Compared with traditional stabilization measures like buttressing and construction of rigid check dam, application of a permanent flexible rock fall protection barrier requires considerably less massive foundation supports with consequent reduction in excavation, concrete, reinforcement and formwork. It is simple to adjust to suit most conditions by selection of appropriate alignment and adjustment of various barrier components. Throughout the construction and maintenance stages, it has advantages in terms of environmental protection and sustainability like tree preservation, reuse/recycle of material and less energy/material consumption. It is considered as a cost-effective, efficient and environmental-friendly protection measure against natural hazard related to potential rock falls.

5.2 Suggestions for enhancement

In view of anticipated considerable demand for flexible rock fall protection barriers against potential natural terrain hazards in Hong Kong, it is necessary to equip or re-educate adequately qualified technical and professional personnel who are capable to design, supervise and maintain the works in connection with flexible rock protection barriers all the way through the service life. In addition, current specifications and conditions of contract for use of flexible rock protection barriers have room for enhancement in various aspects during the design, construction and maintenance stages. For example, legal liabilities for the works such as responsible Registered Geotechnical Engineer, Category I, II & III Personnel and Registered Specialized Contractor, should be clearly stated. More detailed method of measurement should be incorporated in the contract so that the uncertainties on the payment discrepancy could be significantly reduced.

Hong Kong's environment is generally urban and marine along with high levels of air pollution, particularly sulphur dioxide SO_2 which is a main determinant of zinc corrosion. This creates high to very high atmospheric corrosivity (ISO 9223:1992), which may differ from the manufacturer's overseas experience. This may cause an over-optimistic estimation on the long term durability of the barrier system. Further studies on the lifetime of flexible barriers in Hong Kong are therefore recommended so that the life of barrier components could be quoted with consideration of the local atmospheric corrosivity.

6 CONCLUSION

This case of application of a flexible rock fall protection barrier is considered to be an effective engineering option to provide appropriate protection measures against natural hazard related to potential rock falls. It also allows a high degree of flexibility and sustainability through adjustments of the pre-fabricated components so as to suit the various site conditions in natural terrains.

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Land Application and Landscaping Treatment for Debris Resisting Barriers in Discovery Bay

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ABSTRACT

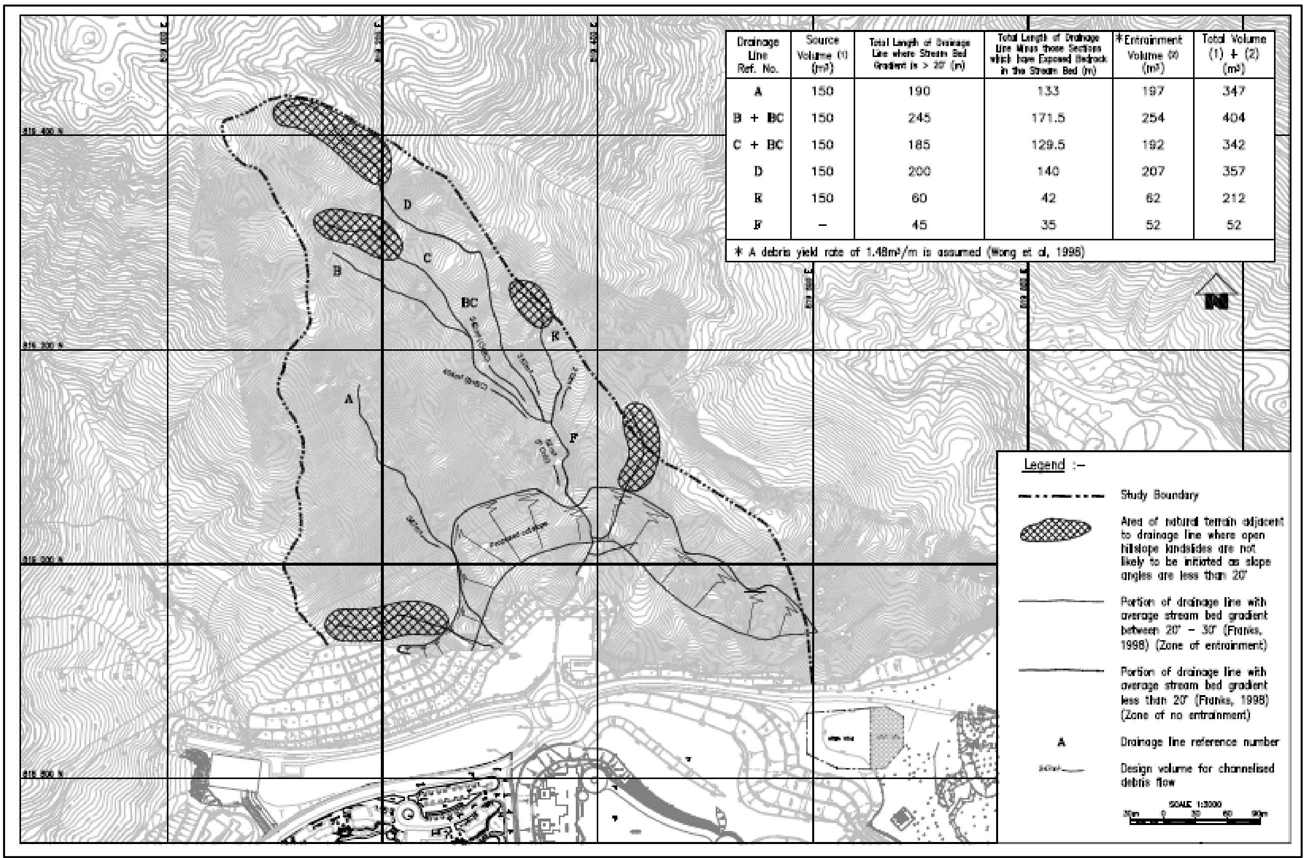
Based on the Natural Terrain Hazard Assessment findings, two debris resisting barriers (DRB) were required to safeguard the future development in Discovery Bay North against possible debris flow due to possible landslide of the natural hillside. Since the sites for DRBs were located within the Conservation Area zone and the Other Specified Uses zone designated for Amenity Area, in accordance with the Discovery Bay Outline Zoning Plan (OZP), the proposed barriers and associated site formation works would require permission/approval from the TOWN PLANNING BOARD (TPB) and would require application for Environmental Permit. In this paper, the design development of DRBs and the general process of application for permission under Section 16 and for Environmental Permit are described. In addition, the soft and hard landscaping treatments for these two DRBs, like surface texture and colouring, for reducing the visual impact will also be discussed.

1 INTRODUCTION

In accordance with GEO Circular No. 28, new developments situated adjacent to natural terrain require screening for natural terrain hazards. The proposed private development (Discovery Bay Master Plan 7.0) was located in the north of Discovery Bay, adjacent to the Discovery Bay Tunnel. The study area for the Natural Terrain Hazard Assessment (NTHA) incorporated all catchments that would affect the proposed development was bounded by the ridges of Tai Che Tung and Sam Pak Au. The study area comprised two major drainage lines orientated northwest-southeast flowing in a southward direction. An aerial view of the study area is given in Figure 1.

Based on the findings in the NTHA, the only natural terrain hazard considered to be significant to the proposed development site was channelised debris flow. The debris was all channelised into drainage line A and F and their corresponding estimated volumes are $347m^3$ and $456m^3$ respectively as shown in Figure 2.

Figure 1: Aerial view of Study Area for NTHS



Drainage Line Ref. No.	Source Volume (1) (m^3)	Total Length of Drainage Line where Stream Bed Gradient is > 20° (m)	Total Length of Drainage Line Minus those Sections which have Exposed Bedrock in the Stream Bed (m)	*Entrainment Volume (2) (m^3)	Total Volume (1) + (2) (m^3)
A	150	190	133	197	347
B + BC	150	245	171.5	254	404
C + BC	150	185	129.5	192	342
D	150	200	140	207	357
E	150	60	42	62	212
F	–	45	35	52	52

* A debris yield rate of 1.48m^3/m is assumed (Wong et al, 1998)

Figure 2: Design volume for channelised debris flow

2 MITIGATION MEASURE OPTIONS

Three schemes of mitigation measures to cope with the anticipated channelised debris flow had been developed. The general arrangement and pros & cons of those options are discussed below.

2.1 Option 1 - debris flow traps at development platform

Debris flow traps in the form of a pit at the edge of future development platform were proposed at stepped cascade that collected storm water drainage line A and F. The debris resulting from possible landslip at uphill would run through the drainage lines and eventually to the proposed debris flow traps via the stepped cascades.

In this option, all construction works would be confined within the development zone without touching into the Other Specified Uses (OU) zone designated as Amenity Area, hence Section 16 application was not required. However, there were concerns on the risk of blocking the stepped cascades and the storm water drainage systems. This option was therefore not considered feasible.

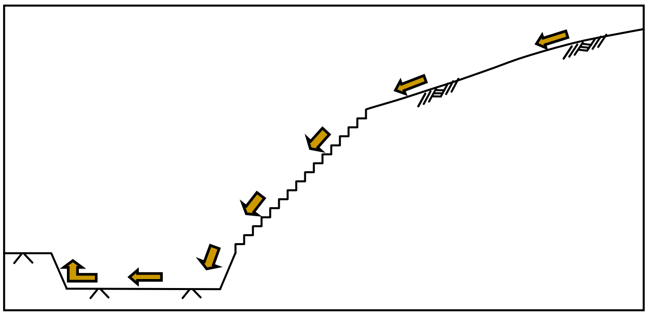


Figure 3: Option 1 – debris flow traps at development platform

2.2 Option 2 - L-shaped retaining structure at crest of cut slope

Another option proposed was to construct L-shaped retaining structures at drainage line A and F, before it entering stepped cascade. Those retaining structures would form a barrier to stop and collect the possible debris flow from upstream.

This option only involved minimal excavation into the natural hillside for constructing the retaining structures, mini-piles and soil nails were required to resist loadings from the debris flow, which included the impact force. However, the proposed retaining structure would encroach into the 'OU (Amenity Area)' zone, hence application for permission under Section 16 was required. Visual impact of this option was also of concern as it required a higher wall to cope with the possible debris run-up during impact and a longer wall to cope with the anticipated volume of debris (sloping ground behind) when compared with option 3.

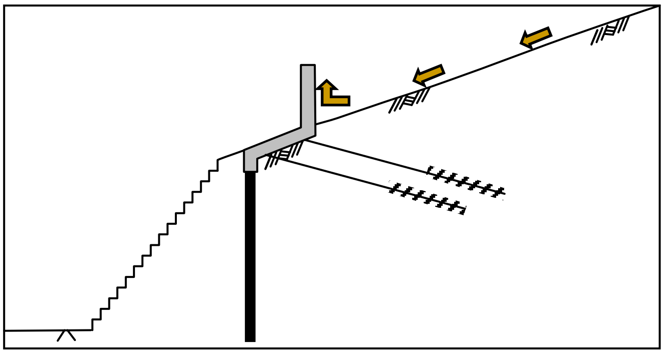


Figure 4: Option 2 - L-shaped retaining structure at crest of cut slope

2.3 Option 3 - debris resisting barrier at crest of cut slope

Debris-resisting barriers (DRBs) in form of a reservoir with concrete wall at all sides were proposed to replace the retaining structure in option 2. The DRBs would be formed by cut and fill method at the inlet area of stepped cascade, where the gradients of drainage line A and F were relatively gentle. The base slab of the DRBs would assist in slowing down the debris, which could reduce both the run-up and the impact force and could also increase the storage capacity. Hence, both height and length of the front wall would be reduced, when compare to option 2 and no pile was required.

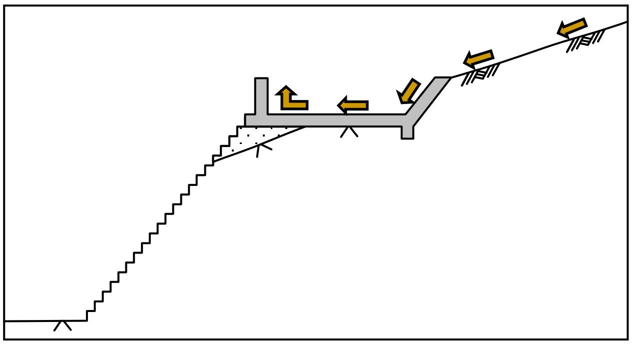


Figure 5: Option 3 – debris resisting barrier at crest of cut slope

Although the DRBs lied within the Conservation Area (CA) and 'OU (Amenity Area)' zone which required application for Environmental Permit (EP) and application for permission under Section 16, this option was selected as their visual impact and construction cost were much less when compared with option 2 and which was finally approved by Buildings Department (BD).

In order to further limit excavation requried and to maximize the storage volumes, DRBs were tailor made in different configurations to suit existing topography and were designed to be founded on insitu rock and mass concrete fill. See Figures 6 and 7 for general arrangement of these DRBs.

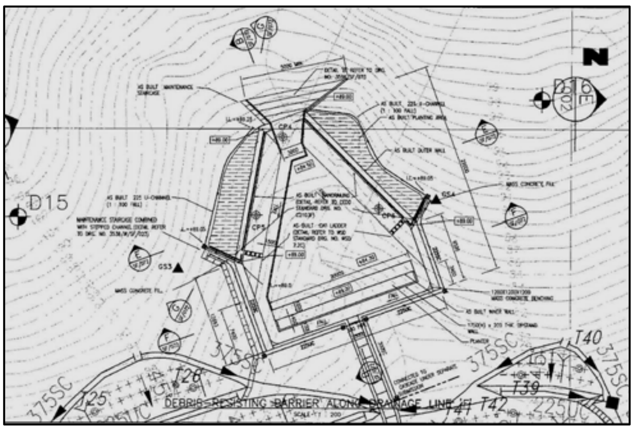


Figure 6: General layout of debris resisting barrier

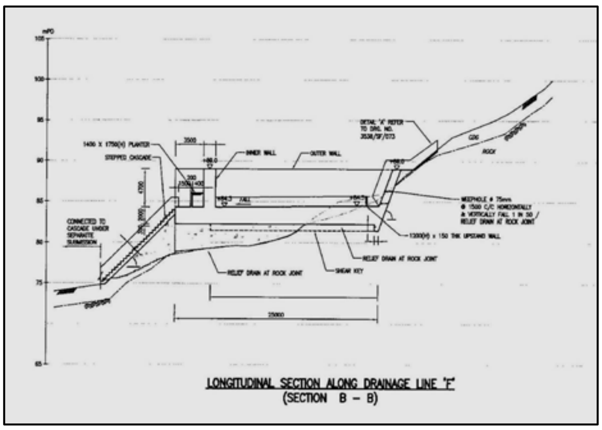


Figure 7: Typical section of debris resisting barrier

3 APPLICATION FOR PERMISSION UNDER SECTION 16 OF THE TOWN PLANNING ORDINANCE (CAP.131)

According to the Discovery Bay Outline Zoning Plan (OZP), the site upon which the DRBs were to be constructed lied within a 'CA' zone and an 'OU' zone designated as Amenity Area. According to the Notes relating to the 'CA' and 'OU (Amenity Area)' zones on the OZP, the proposed DRBs and associated site formation works would be regarded as a 'Utility Installation for Private Project'. This would require permission/approval from the TOWN PLANNING BOARD (TPB) under the provisions of Section 16 of the Town Planning Ordinance.

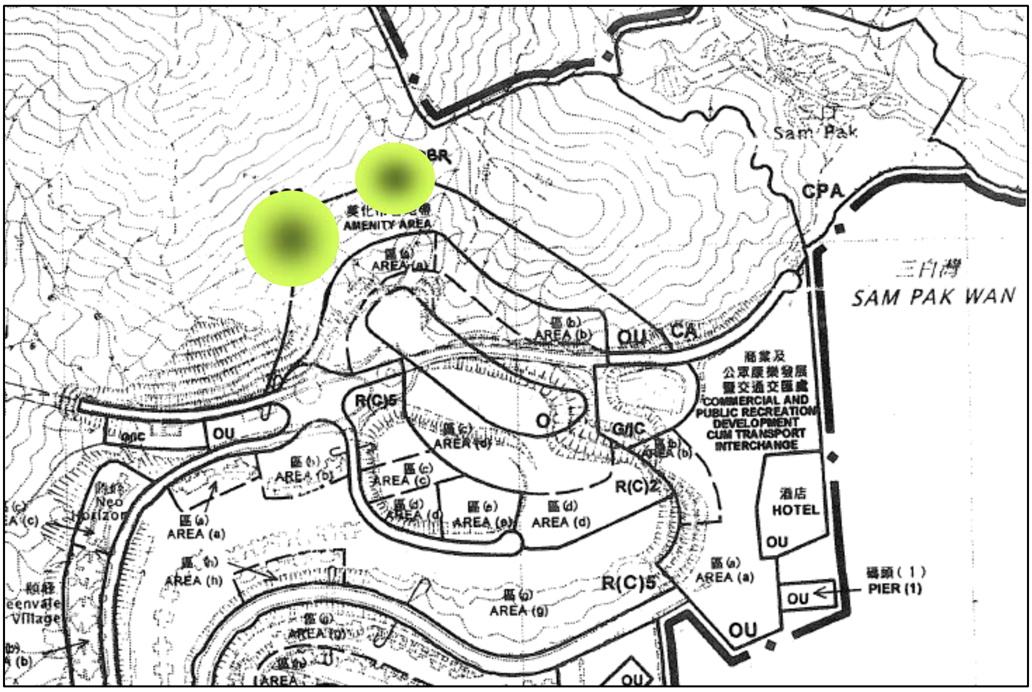


Figure 8: Part plan of OZP for Discovery Bay

In order to furnish the Board Members of the TPB with details of the proposed DRBs and justifications in support of the application, the following information was submitted in the Planning Statement to facilitate their consideration. The application was finally approved subjected to the implementation of a landscaping master plan and visual mitigation measures

- Background information such as site location, land status and statutory planning content;
- Justification for proposed works such as any other alternatives, why it needs, comments from other government departments:
- Technical Considerations such as civil engineering issues on temporary works and permanent works, landscape and visual impact issues, environmental impact issues during temporary and permanent works and methods of maintenance: and
- Figures including layout and sections of DRB, photomontage of DRB, arrangements of other alternatives, zone of visual influence, photomontage from representing Visual Sensitive Receiver, Landscape mitigation measures, etc.

4 LANDSCAPING MASTER PLAN & VISUAL MITIGATION MEASURES

Mitigating visual and physical impacts by the two DRBs had been considered during the design stage. The barriers had been designed in trapezoid shapes to match the geography and had been located at existing rock outcrop to reduce the extent of cutting required. Both soft and hard landscaping were proposed in the Landscaping Master Plan to mitigate the possible visual impact from the DRBs.

4.1 Soft landscaping works

For soft landscaping works, the planting around the DRBs was divided into two zones; an inner zone containing concentrated planting to hide the wall, and an outer zone with scattered planting which blended into the surroundings as shown in Figure 9 and 10. In addition, planter was also provided at the front wall to facilitate the growth of creepers to cover the wall up.

Planting around the structures incorporated mostly species that were commonly found in the area and would be able to establish themselves easily. A mixture of species (none with striking forms or colours) was recommended to emulate a natural layout while not attracting undue attention.

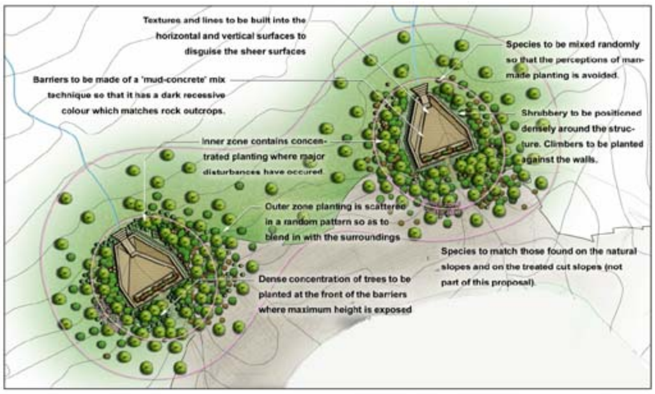


Figure 9: Landscaping plan for DBR (extracted from Figure 5 of Urbis (2004))

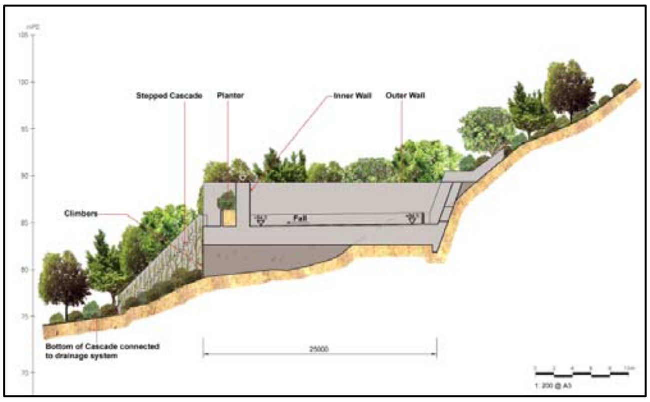


Figure 10: Landscaping section for DBR (extracted from Figure 6 Urbis (2004))

4.2 Hard landscaping works

The proposed hard landscape treatments comprised finishes to the concrete structures which were designed to minimise their visual intrusion. 'Mud-concrete' was proposed in the Landscaping Master Plan to emulate the earthy tones of rock outcrops. The rough surface created was also conducive to creeper growth, which would further enhance the appearance. It was also proposed that the sheer surface of the barriers be interrupted with random lines and tones which broke the massive structure up. Photo of mud-concrete sample is shown in Plate 1.

Plate 1: Photo of mud-concrete sample

However, although there was a successfully case of applying "mud-concrete" in other landscaping project in Discovery Bay, the mud-concrete mix applying to such a large vertical walls was found unsuitable after trial had been performed on site. The "mud-concrete" sample could not be securely bonded to the vertical surface of the concrete wall and its strength to support the dead weight of itself was also of question.

Shotcrete with irregular surface and recessive in color was therefore proposed as an alternative on-site. Trials were conducted to judge the performance of different application methods and to decide the texture and color that could blend in well with the surrounding. Both application methods, 1) manual control and 2) rock fragment control were found to be acceptable. The manual control method was therefore chosen because it is relatively easy to apply. Steel wire mesh fixed on the wall surface was also proposed to further enhance the durability of shotcrete and its bonding with concrete surface. Photo of trial panels carried out on site are shown in Plate 2.

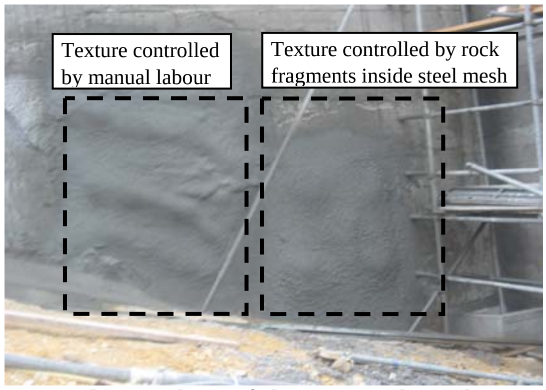


Plate 2: Photo of shotcrete trial panel

The appearance of the DRB before and after applying the visual mitigation measures are presented in Plate 3 and Plate 4 below. The DRB was blended in well with the surrounding.

Plate 3: General layout of DBR before landscaping works

Plate 4: General layout of DBR after landscaping works

5 ENVIRONMENTAL PERMIT APPLICATION

In addition to the Section 16 application, since the sites were partially lied within a 'CA' zone, the construction and operation of the DRBs were classified as a designated project (DP) in accordance with Schedule 2, Part I, Q – Miscellaneous, Q.1 of the Environmental Impact Assessment Ordinance (EIAO). Hence, an Environment Permit (EP) was required.

The majority of DPs would require Environmental Impact Assessment (EIA) studies to be conducted ['EIA route']. However, under some circumstances the applicant could also seek permission from Director of Environmental Protection (DEP) in accordance with Section 5(11) of the EIAO to apply directly for an Environmental Permit (EP) ['DIR route']. Since environmental impact of the DRBs was relatively minor, which should fall within the guidelines and criteria laid down in the Memorandum on Environmental Impact Assessment Process (TM-EIAO), and the effectiveness of the mitigation measures has been demonstrated in practicebefore, it was then decided to try the DIR route. The letter of permission to apply directly for environmental permit from DEP was received and the EP was then also granted.

6 CONCLUSIONS

Debris-resisting barrier is a very common type of mitigation measures to safeguard future development against channelised debris flow type natural terrain hazard. In view of the need to position them along the existing drainage line, they would normally be positioned outside the lot boundary and hence may require Section 16 application or even Environmental Permit application. In determining the position of DRB, prime consideration should be to minimize the environmental and visual impact. Those visual impact mitigation measures described above for the Discovery Bay project can be used as a reference for similar projects. Due to the relatively minor nature of DRB, DIR route should be considered if application for EP is unavoidable. It will have substantial saving in the programme.

ACKNOWLEDGEMENTS

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Field Estimates of Debris Flow Mass Balance and Velocity

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ABSTRACT

Detailed debris trail mapping of the 7 June 2008 landslides in North Lantau has highlighted the many challenges in estimating debris flow mass balance. Given the difficulties in establishing the pre-landslide drainage line morphology, uncertainty arises in estimating the volume of entrainment during the main debris pulse and the quantity of deposited material subsequently eroded by fluvial action. Yet realistic estimates of debris flow mass balance are necessary for fruitful back analyses of debris flows and subsequent design of mitigation works such as debris flow barriers. Unrealistic estimates may also skew the historical data base thereby hindering progress in developing a better understanding of such events.

With particular focus on a large channelised debris flow that affected Yu Tung Road on 7 June 2008, the paper outlines the staged approach adopted to progressively refine estimates of debris mass balance through the use of oblique post-landslide aerial photographs, detailed topographical survey and a series of increasingly focused field mapping exercises. Debris super-elevation data was also recorded to provide estimates of debris flow velocities at various points along the drainage line. The staged mapping process and debris flow modeling was employed to highlight areas where either the mass balance estimates or modeling assumptions required closer scrutiny. This process resulted in greater confidence in the range of parameters considered for design of the debris flow barrier.

1 INTRODUCTION

The intense rainstorm of 7 June 2008 triggered a large number of landslides on the natural hillside above the North Lantau Expressway and Yu Tung Road, Tung Chung (Figure 1). Hazard mitigation measures were proposed for the study area and detailed mapping of several of the landslides was carried out to collect representative data on the debris mobility, likely debris velocity and the mass balance of debris along the trail. The data were then used to refine and calibrate the theoretical debris mobility models for selection of design parameters to be used for design of the mitigation works.

Mapping of debris flow trails to estimate the mass balance and debris flow velocity is a challenging task. Some of the challenges encountered in mapping are related to the changing condition of the debris trail which enters a rapidly evolving state immediately after the first of the debris pulses has passed by. With reference to the largest of 7 June 2008 landslides in this area, which is located in Catchment No. 30 (Figure 1), some of the problems which needed to be tackled included:

- Distinguishing between areas of erosion, deposition and transportation when the debris trail was first mapped due to surfaces being plastered with dried-up debris which can superficially resemble abraded colluvium or freshly deposited debris,
- Misinterpretation of many of the debris covered surfaces as areas of erosion, which may result in an overestimation of entrainment. In later inspections of the drainage line, after much of the fines had been washed away by further rain, many of these surfaces were revealed to be abraded topsoil indicating debris transportation with very little erosion,
- Inherent difficulties in estimating the debris mass balance during the main debris pulse due to remobilization of debris by subsequent pulses and ensuing erosion by fluvial action.

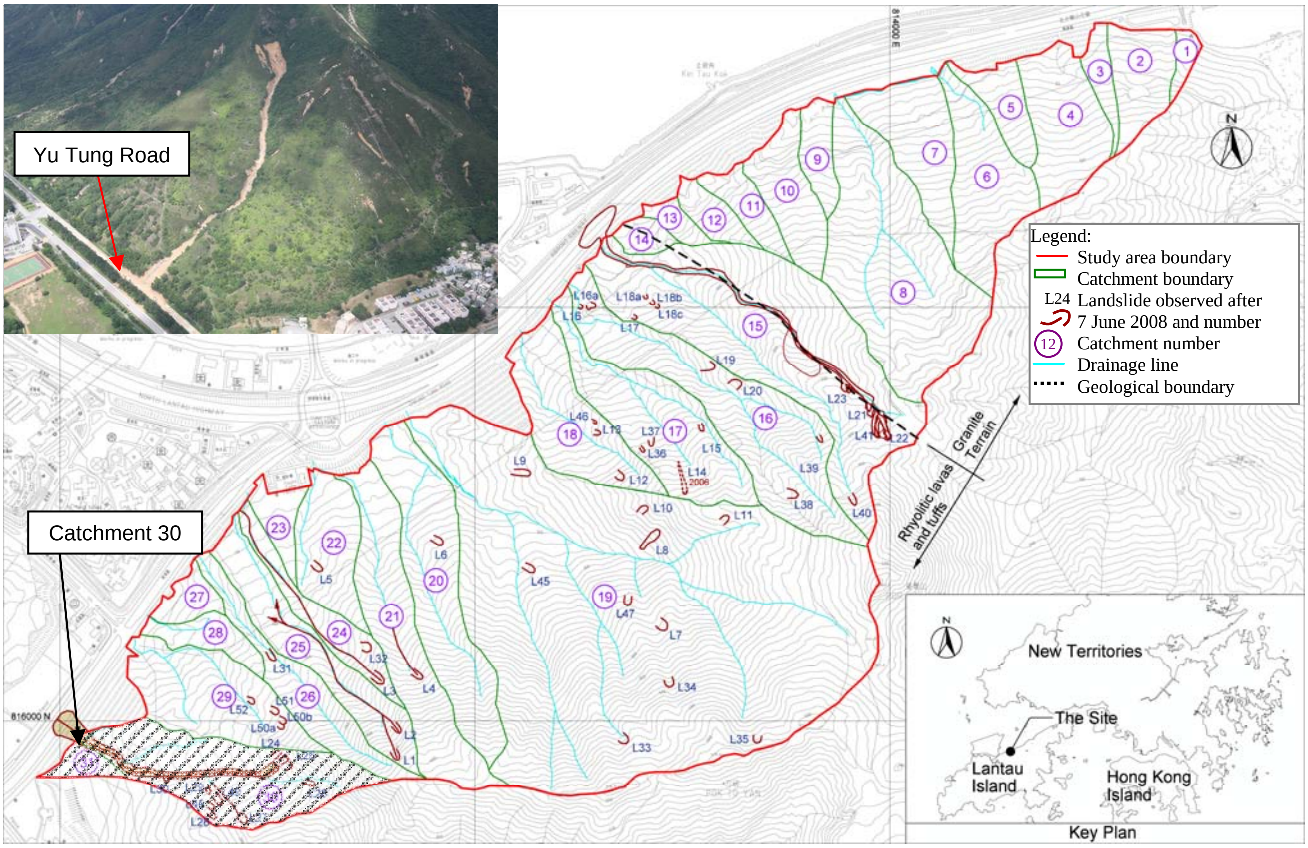


Figure 1: Location of Catchment 30 with the largest of the 7 June 2008 landslides

2 THE SOURCE VOLUME

A good estimate of the landslide source volume is the starting point for determining the overall mass balance and establishing the initial conditions for debris mobility analysis. Based on field mapping of the debris trail in Catchment No. 30, the initial estimate of displaced material at the source was about 2700 m^3 of which about 300 m^3 remained within the source area. This compares favourably with the results of the subsequent laser scan survey (Figure 2), which suggested that about 2400 m^3 of debris may have come from the source area, assuming that the original topography had already been eroded by previous landslides that were identified from aerial photographs interpretation (API).

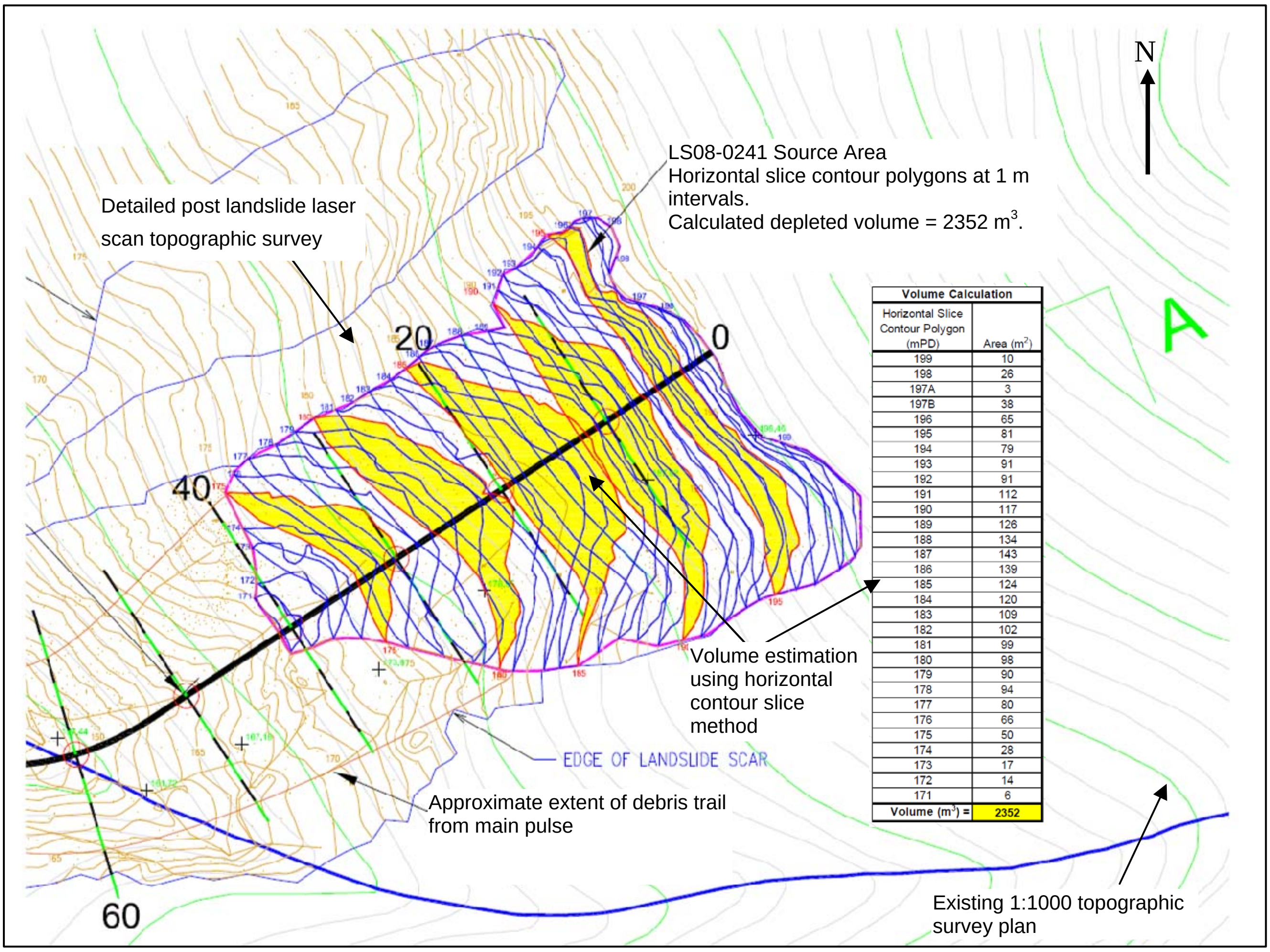
3 MASS BALANCE

Figure 3 outlines the typical processes in the evolution of a debris trail that need to be considered when attempting to estimate the mass balance for the largest debris pulse and the likely volume changes along the drainage line. This involves an overall assessment of the estimates of volume lost from the landslide source areas, estimates of erosion and deposition along the trail and the quantity of material that was deposited at the toe of the trail which may be based on measurement or on reports of the volume that was removed during the emergency works. Such data is essential for the debris flow modelling calculations.

Field mapping was carried out in stages in parallel with the debris mobility modelling studies in order to identify areas where further mapping was required and where changes to the mobility modelling assumptions are necessary. A mass balance table which takes account of the evolution of the debris trail over time was used to provide a framework within which plausible scenarios were tested to ensure that both the estimates of mass balance from both field mapping and analytical models were compatible and realistic. A graphical presentation of the mass balance table is shown in Figure 3.

The debris trail was measured at 543 m in horizontal length from the crown of the landslide source to the crest of the cut slope alongside Yu Tung Road. The estimates of mass balance indicate that about 3100 m^3 of debris may have reached Yu Tung Road (Figure 3). However, this volume, which was not mobilized all at the same time, includes:

- ➢ the volume of the main debris flow pulse,
- ➢ secondary pulses,
- ➢ secondary landslides,
- ➢ material transported by fluvial action prior to the debris removal and subsequent erosion and transportation.



Volume Calculation	
Horizontal Slice Contour Polygon (mPD)	Area (m^2)
199	10
198	26
197A	3
197B	38
196	65
195	81
194	79
193	91
192	91
191	112
190	117
189	126
188	134
187	143
186	139
185	124
184	120
183	109
182	102
181	99
180	98
179	90
178	94
177	80
176	66
175	50
174	28
173	17
172	14
171	6
Volume (m^3) =	2352

Figure 2: Detailed topographic survey of source area by laser scanning

As can be seen from Figure 3 below, the location of apparent maximum ‘active volume’ (the maximum volume which passed any point along the debris trail) tends to migrate downstream with time as more material is eroded and transported by fluvial action.

Estimates of erosion and deposition based on field mapping take account of higher entrainment rates in the upper channel and erosion of deposited debris by subsequent fluvial action. Debris mass balance estimates also took into account about 40 m^3 of bulked debris assumed to be added from secondary landslides between chainages 252 and 339 (Plate 1), about 175 m^3 of fresh debris, deposited between chainages 360 and 410 above a rock step and about 360 m^3 of fresh debris deposited between chainage 486 and chainage 543, above the cut slope along Yu Tung Road, was estimated to have been removed by fluvial erosion. The margin of error in the estimation of the debris mass balance has been reduced by surveying the source and refinement of estimations of entrainment, deposition and erosion.

The mass balance profile of the main debris flow pulse based on field evidence and analytical results of the debris modelling (Tattersall et al. 2009), is shown by ‘line B’ in Figure 3 below. The un-bulked source volume based on the survey data is 2352 m^3 (Figure 2), however, a bulked source volume of 2400 m^3 was assumed to contribute to the main pulse of debris modelled by the dynamic debris mobility modelling programs since about 300 m^3 of debris remained in the source. Debris modelling was first carried out using typical channelised debris flow parameters of apparent friction 11.3° and turbulence coefficient of 500 ms^{-2}.

However, an apparent friction angle of between 5.7° and 8° and a turbulence coefficient of about 350 ms^{-2} was derived from back-analyses in order to match the estimated velocities and measured debris heights of the mapped debris flow (Tattersall et al. 2009), which is comparable to the most mobile landslides back-analyzed in Hong Kong (Ayotte & Hungr 1998).

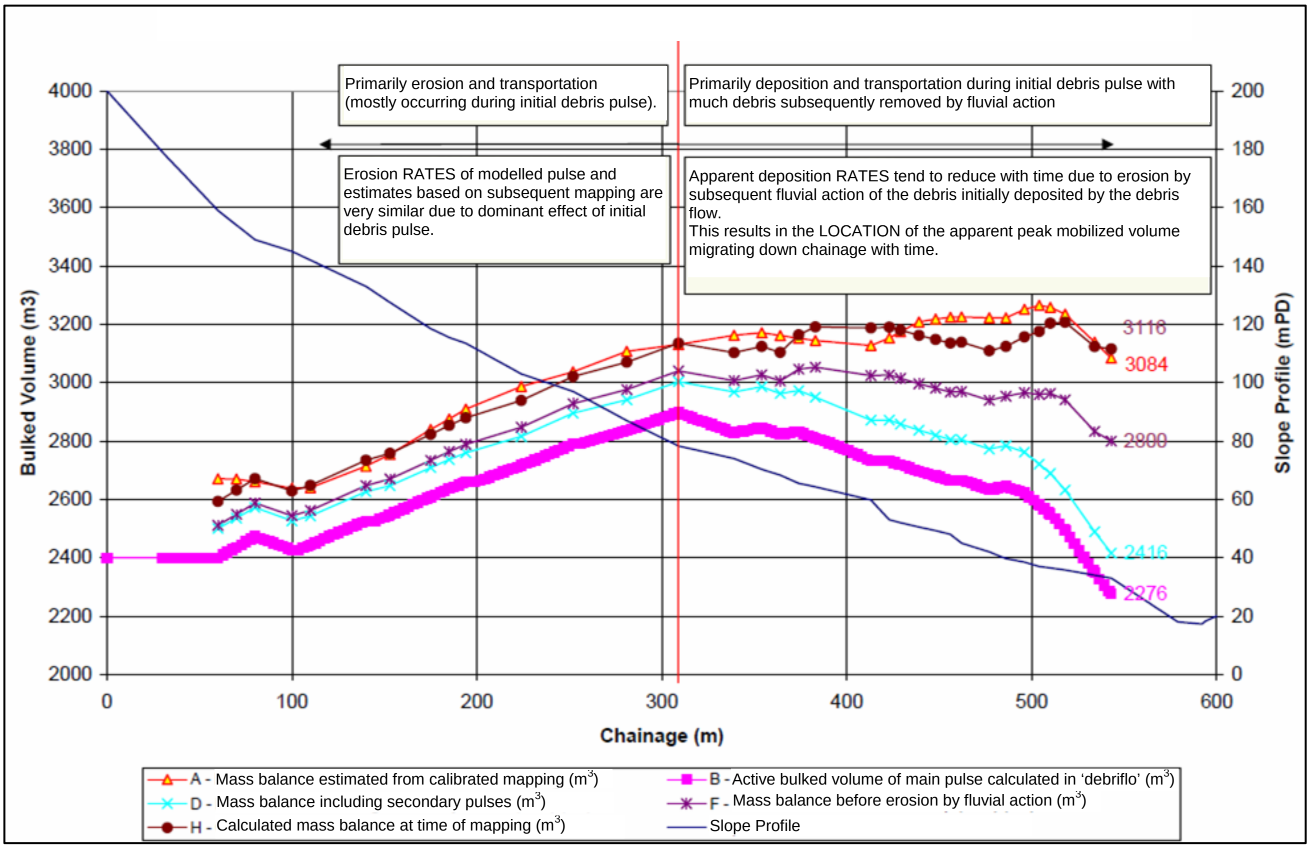


Figure 3: Estimates of mass balance based on mapping and debris flow modelling

4 FIELD MAPPING

Due to the heavily wooded banks the main difficulties in mapping debris flow trails are to locate features and to obtain a good overview of the debris trail in order to understand what had occurred in terms of the main processes of entrainment, transportation and deposition. Although a Global Positioning System (GPS) is now widely available, photocopies of oblique aerial photographs taken by helicopter a few days after the debris flow had occurred (Plate 1) provided the greatest assistance in locating features and providing the necessary overview of the main landslide processes. The use of tape measurement and a 5 m long telescopic ranging rod to establish a chainage reference and dimensions of the trail were indispensable. The use of LiDAR (Light Detection and Ranging) based survey data should in future significantly enhance the determination of pre-landslide drainage line morphology.

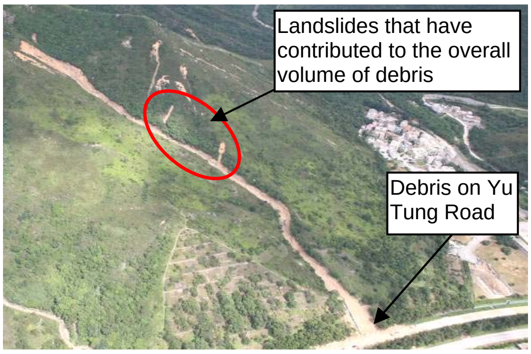


Plate 1: Oblique aerial photograph of the Yu Tung Road debris flow

An inspection of the lower portion of the drainage channel in Catchment No. 30 had previously been carried out in September 2007 as part of the study for the design of the hazard mitigation works. Comparison between the September 2007 photographs and the post June 2008 landslide photographs helped to identify more clearly areas of erosion and deposition (Plate 2).

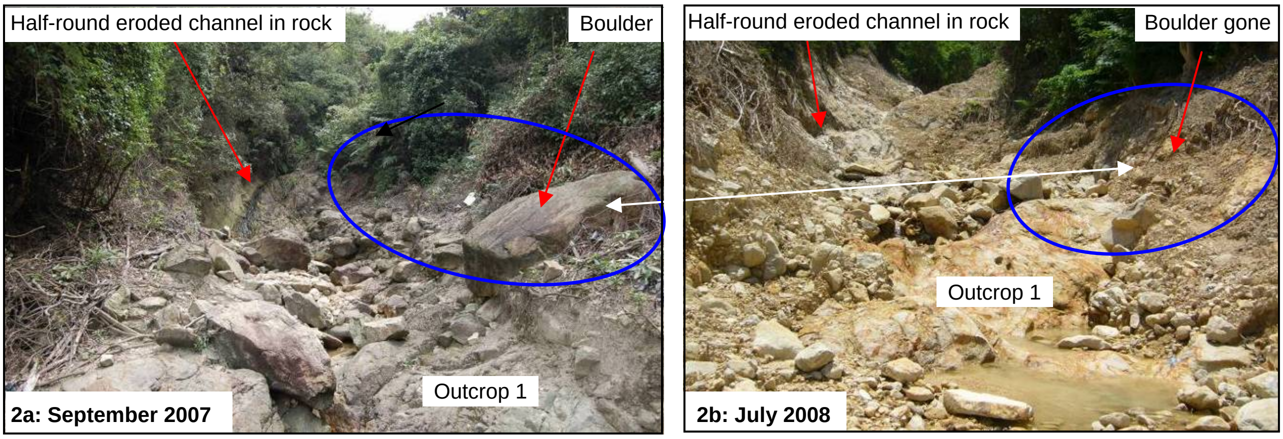


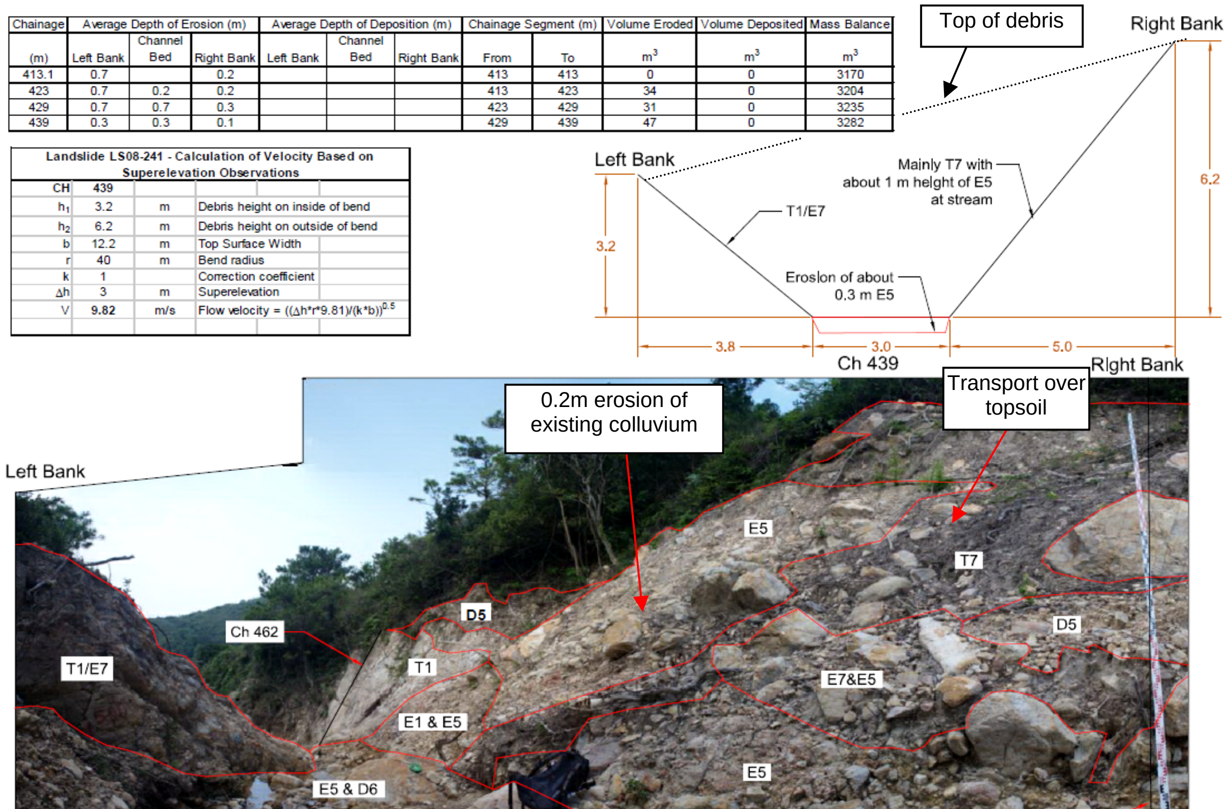
Plate 2: Drainage line before and after the 7 June 2008 debris flow

In this particular case the availability of pre- and post-landslide photographs taken less than one year apart was fortunate (Plate 2). However, only a short section of the drainage line was inspected prior to failure due to the dense vegetation. Nevertheless this gave a greater degree of confidence in the estimates of erosion and deposition in this part of the channel.

Chainage	Average Depth of Erosion (m)			Average Depth of Deposition (m)			Chainage Segment (m)		Volume Eroded	Volume Deposited	Mass Balance
(m)	Left Bank	Channel Bed	Right Bank	Left Bank	Channel Bed	Right Bank	From	To	m^3	m^3	m^3
413.1	0.7		0.2				413	413	0	0	3170
423	0.7	0.2	0.2				413	423	34	0	3204
429	0.7	0.7	0.3				423	429	31	0	3235
439	0.3	0.3	0.1				429	439	47	0	3282

Landslide LS08-241 - Calculation of Velocity Based on Superelevation Observations

CH	439		
h_1	3.2	m	Debris height on inside of bend
h_2	6.2	m	Debris height on outside of bend
b	12.2	m	Top Surface Width
r	40	m	Bend radius
k	1		Correction coefficient
Δh	3	m	Superelevation
V	9.82	m/s	Flow velocity = $((\Delta h*r*9.81)/(k*b))^{0.5}$



E1 = erosion of rock, E5 = erosion of colluvium, E7 = erosion of top soil, T1 = Transport over rock, T7 = Transport over topsoil, D5 = matrix supported colluvium, D6 = alluvially reworked clastic debris

Figure 4: Detailed mapping and record photographs for the 7 June 2008 debris flow

Mapping of the landslide and debris trails was carried out using proformas developed under the natural terrain hazard study for the Tsing Shan Foothill Area (MFJV 2004). The proformas for the Yu Tung Road channelised debris flow were also supplemented by simple line diagrams, annotated photographs and tabulations of deposition and entrainment that focused the subsequent mapping stages on identifying fundamental information for the debris modelling analysis (Figure 4).

Estimates of debris velocity can be made through measurement of the debris super-elevation at bends (Plate 3) in the drainage line using equation (1) below (Hungr et al. 1985).

$$\Delta h = k\frac{bv^2}{rg} \tag{1}$$

where Δh = elevation difference between the two sides of the debris flow, b = surface width of the flow, v = mean flow velocity, r = mean radius of curvature, k = correction coefficient and g = gravitational acceleration (9.81 ms^{-2}).

A worked example is shown in Figure 4 above. The estimates of debris velocity based on super-elevation measurements at five locations were used to provide an independent reference against which the results of the analytical debris modelling work could be compared (Tattersall et al. 2009). Care is needed when measuring super-elevation to ensure that the height of 'splash' marks are not included and that curved bends are chosen without abrupt changes in flow direction and that the line of section is normal to the path of the debris flow.

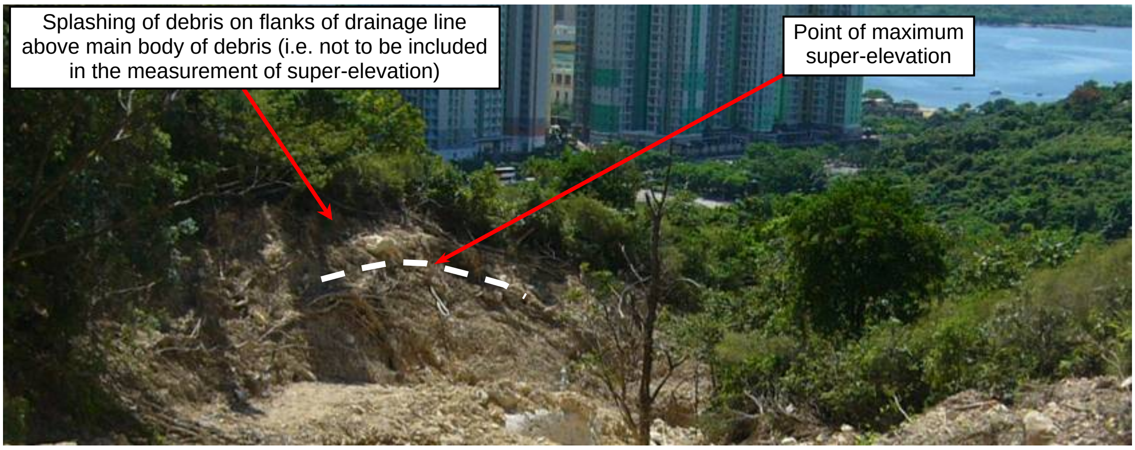


Plate 3: Debris super-elevation recorded around bend at Chainage 420

5 DEBRIS FLOW VIDEO

An advantage in the case of the Yu Tung Road channelised debris flow was that the lower part of the debris flow had been captured on video (www.youtube.com/watch?v=R2uTKyK1c9k). This provided an extremely valuable source of information on the mobility of the debris flow. The video confirmed that most of the debris volume travelled as an initial large pulse followed by several smaller pulses. The information provided a sound basis for the analytical debris mobility modelling work and was used to refine the mass balance estimates.

6 CONCLUSION

Estimating the debris mass balance through detailed field mapping is a challenging task that involves recognizing areas of entrainment, transportation and deposition along the debris flow path and the spatial distributions of these processes. It requires differentiating between the characteristics of the main pulse, subsequent pulses, effects of erosion and re-working of the deposited debris by ensuing fluvial action.

A careful assessment of the super-elevation of debris that occurs around bends of the flow path could give reasonable estimates of debris velocity. The availability of detailed topographic survey, oblique aerial photographs taken soon after the landslide and pre-landslide site inspection photographs could greatly assist in locating features on site and in recognizing significant zones of entrainment and deposition. The characteristics of the debris trail may change within a short time of the landslide and a staged approach to the mapping helps to obtain a better understanding of the event and provides more realistic estimates of the mass balance. However, much still relies on experience, as estimating the mass balance involves uncertainties. Based on our experience so far it is not unreasonable to allow for a significant margin of uncertainty in the maximum volume estimated.

ACKNOWLEDGEMENTS

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Benchmarking Exercise on Landslide Debris Mobility Modelling

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ABSTRACT

A benchmarking exercise on landslide debris mobility modelling was held in December 2007 in Hong Kong as part of the International Forum on Landslide Disaster Management. This international benchmarking exercise was organised for the first time and participated by 13 groups of researchers and practitioners working actively in this new area. The exercise was steered by an International Review Panel, comprising Prof N R Morgenstern of the University of Alberta, Prof O Hungr of the University of British Columbia and Ir H N Wong of the Geotechnical Engineering Office. In this benchmarking exercise, participants completed numerical modelling of selected benchmark cases and provided their reports for review by the Panel. The exercise served the intended purposes of taking stock of progress made in numerical modelling of landslide debris mobility and facilitating interactions among researchers and practitioners. During the Forum, the participants presented key summaries of their findings and took part in discussions on various aspects of dynamic modelling of landslide mobility. It was encouraging to see an overall consistent outcome of the modelling results, despite the different solution approaches adopted. In this exercise, several 3-D models were assessed to have provided consistently similar modelling results for a range of calibration cases. The review has also identified areas requiring attention for further refinement and development.

1 INTRODUCTION

The International Forum on Landslide Disaster Management was held in Hong Kong in December 2007 to mark the 30th anniversary of the implementation of landslide risk management in Hong Kong. This forum was one of the key events of the international geotechnical community and was to address focused subjects of direct relevant to landslide disaster management. One of the highlights of this Forum was a benchmarking exercise on landslide debris mobility modelling, which was steered by an International Review Panel, comprising Prof N R Morgenstern of the University of Alberta, Prof O Hungr of the University of British Columbia and Ir H N Wong of the Geotechnical Engineering Office (GEO) of the Civil Engineering and Development Department. The exercise was to facilitate interaction among research groups and practitioners working on computer-based numerical models for simulation of the dynamics of landslide debris motions. It was also to assess if the different numerical methods adopted by the various parties reached some degree of commonality in landslide debris mobility modelling and to identify areas for further attention and development.

This benchmarking exercise was organised for the first time and participated by 13 groups of researchers and practitioners working actively in this new area. A team of geotechnical engineers, including the authors of this paper, from the GEO provided support to and participated in this exercise. This paper presents a summary report on the benchmarking exercise, including salient details of the findings extracted from the report of the Review Panel (Hungr et al. 2008). Selected figures and data are reproduced from the report of the Review Panel. Other details of this exercise, including the review report by the Review Committee, are published in the Proceedings of the 2007 International Forum on Landslide Disaster Management (Ho & Li 2008).

2 THE BENCHMARKING CASES

Thirteen cases were selected for the benchmarking exercise. A group of verification cases comprises an analytical solution on dam-break scenario and two sets of laboratory tests. The purpose of this group of cases was to verify the numerical codes against analytical solutions or results of physical laboratory model cases conducted under closely controlled conditions, of which the initial landslide dimensions, geometry of the runout path and the rheological parameters of the testing materials was well-defined.

The other cases are actual landslide cases of varied complexities in topography, materials involved and characteristics of the landslide debris. The participants were also invited to undertake a forward prediction for one of the landslide cases.

Data provided for each of the cases included a digital elevation model (DEM) of the landslide area, encompassing the source, the travel path and the deposition area, a DEM representing the vertical thickness map of the landslide source, a brief description of the landslide together with a number of photographs of each site. Other information, including references for detailed site-specific landslide study reports, where available, was given. A map showing the outline and thickness of the final landslide deposits and the outline of any material entrainment areas along the runout path was also provided.

3 PARTICIPANTS AND NUMERICAL MODELS USED

Twenty-one research groups working on the subject were invited in March 2007 to participate in the Exercise and to assemble numerical models for the benchmarking cases. The participating groups were asked to back-analyse the cases using their models, so as to yield the best simulation of observed behaviour, as well as to produce a forward prediction of one potential landslide. By September 2007, a total of 13 teams (from Austria, Canada, France, Hong Kong, Italy, Japan, Norway, Spain and USA) provided submissions using their respective numerical models for the different benchmarking cases (Table 1). During the Forum, the participants also presented summaries of their submission and took part in discussions on various aspects of dynamic modelling of landslide mobility.

The numerical models adopted can be summarised in terms of the basic solution approach, solution dimensions, basal rheology, internal stress assumptions, as well as capability to model entrainment along runout path. The report by the Review Panel provides further detailed information and more elaborated classifications (Hungr et al. 2008).

All, except two (TOCHNOG and PFC), of the models adopt the integrated approach, in which the motion of the debris mass as represented by an assembly of discrete columns above the sliding surface is calculated based on depth-averaged shallow-flow assumptions. The TOCHNOG model is a finite element model, which adopts differential solution with the internal deformation of the debris mass modelled in detail. The PFC model adopts particulate modelling approach where the debris mass is represented by a large number of discrete particles and solution for motion of the particles is obtained using distinct element method.

In terms of solution dimensions, all, except two (Wang and DAN), of the models are 3-D (i.e. analysis the motion of the debris mass in plan and cross-section). The Wang model is a 2-D model (i.e. analysing cross-section of a single pre-defined width). The DAN model is a pseudo-3-D model, which analyses cross-sections of pre-defined widths along runout trail. Many of the 3-D models also have the capability to undertake 2-D analyses.

All models, except FLO-2D, have the option to adopt frictional basal rheology (viz. shear resistance at the surface of rupture to the movement of the landslide debris modelled as linear function of total normal stress). Most of the model adopting integrated approach (e.g. MADFLOW, RAMMS, DAN3D, FLATMODEL, 3dDMM, RASH3D, DAN and Pastor also have Voellmy basal rheology (i.e. basal resistance comprises a frictional term and a term proportional to the square of the flow velocity). FLO-2D and RASH-3D have three-term basal rheology accounting for frictional, viscous and turbulent resistance.

The assumptions on internal stress distribution adopted in various integrated models were different. About half of all integrated models, viz. MADFLOW, DAN3D, 3dDMM, RASH3D, DAN and TITAN2D, adopts the Savage-Hutter relationship, which accounts for equilibrium of lateral earth pressure distribution within the landslide debris and basal resistance. The other half of integrated models, viz. RAMMS, FLO-2D, FLATMODEL, SHALTOP-2D and Pastor, assumes hydrostatic pressure distribution (i.e. with no internal shear strength within the landslide debris). Sassa-Wang assumes constant (at rest) lateral earth pressure coefficient throughout the debris mass. The Wang model assumes Rankine stress state within the debris mass and explicitly incorporates energy changes due to shear distortion of the landslide debris (in additional to change of potential energy) and dissipation of energy through basal friction.

For all the integrated models, seven models, viz. MADFLOW, DAN3D, FLATMODEL, 3dDMM, DAN and Pastor, have facilities to allow for entrainment of material along the runout path. However, it is noted that the approaches to prescribe or to calculate entrainment are different for the seven models. Six integrated models, viz. DAN3D, 3dDMM, RASH3D, DAN, Pastor and Sassa-Wang, also have the capability to allow for variation of basal resistance models (parameters) along the runout path.

Table 1: List of participants, numerical models used and submissions

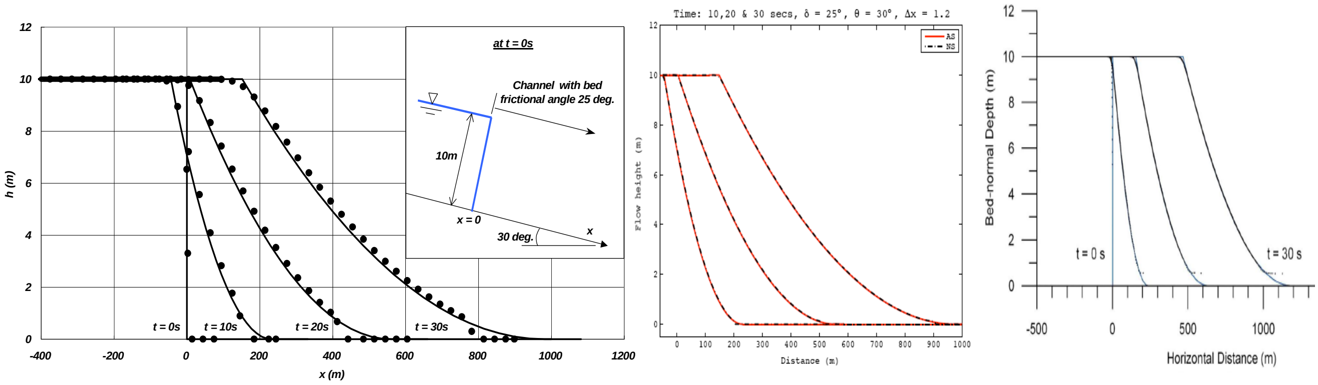
Team	Model	Verification Cases			Landslide Cases									
		Dam Break	Deflected Sand Flow	USGS Flume Test	Shum Wan Landslide	Fei Tsui Road Landslide	1990 Tsing Shan Debris Flow	Sham Tseng San Tsuen Debris Flow	Frank Slide	Thurwieser Rock Avalanche	2000 Tsing Shan Debris Flow	Tate's Cairn Landslide	Tate's Cairn Landslide Forward Prediction	Lo Wai Debris Flood
University of Alberta, Canada	Wang	●	●	●	●	●	●	●			●	●		●
University of Hong Kong	MADFLOW		●		●	●				●				
University of Milan, Italy	TOCHNOG	●			●				●	●				
Norwegian Geotechnical Institute, Norway	RAMMS							●						
	DAN3D(NGI)							●				●	●	
	FLO-2D(NGI)											●		
Technical Uni. of Catalonia, Spain	FLATMODEL	●		●			●				●	●	●	
GEO, Hong Kong	3dDMM	●	●	●	●	●	●	●	●	●	●	●	●	
Universite Paris Diderot, France	SHALTOP-2D	●		●	●	●			●					
	RASH3D(Paris)	●												
Uni. of British Columbia, Canada	DAN	●				●	●		●					
	DAN3D		●		●	●	●	●		●		●	●	
CEDEX, Madrid, Spain	Pastor	●	●	●	●	●	●	●	●	●	●	●	●	●
Vienna Uni. of Tech., Austria	PFC								●	●				
Kyoto University, Japan	Sassa-Wang				●	●			●					
Politecnico Di Torino, Italy	RASH3D		●			●			●			●	●	
University at Buffalo, USA	TITAN2D		●						●					

Note: DAN3D(NGI) and FLO-2D(NGI) denotes DAN and FLO-2D models used by Norwegian Geotechnical Institute; and RASH3D(Paris) denotes RASH3D model used by Universite Paris Diderot.

4 MODELLING RESULTS

4.1 Verification cases

For the dam-break scenario, an analytical solution is available for comparison with the simulation results and validating the debris runout profiles calculated by the numerical models for different times after the 'dam-break'. In the analytical solution, resistance to flow is derived from basal friction only and the debris mass is assumed to have no internal friction. With the same basal rheology and material parameters specified in the exercise, six models, viz. FLATMODEL, 3dDMM, SHALTOP-2D, RASH3D(Paris), DAN and Pastor, give simulation results that match well with the analytical solutions at 10 seconds, 20 seconds and 30 seconds. A landslide mass with internal friction was adopted in TOCHNOG's and Wang's modelling. The modelling results are not comparable to the analytical solution, which assumes that the debris mass has no internal friction. Figure 1 shows the selected simulation results.

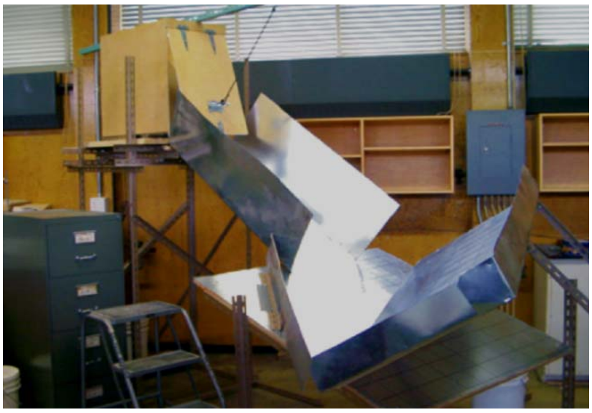


(a) 3dDMM (b) SHALTOP-2D (c) DAN3D

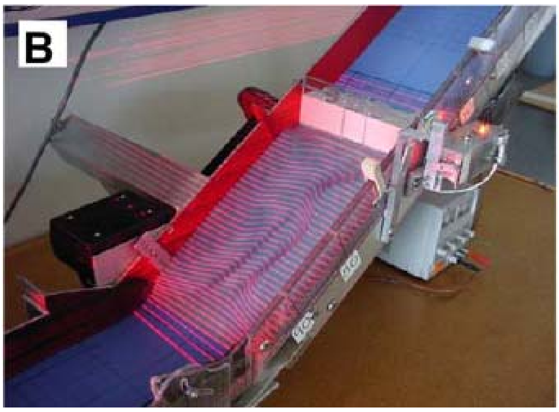
Figure 1: Selected simulation results for the dam break scenario

The deflected sand flow experiment carried out by the Rock Mechanics Laboratory of the Swiss Federal Institute of Technology in Lausanne, involved releasing dry fine sands from a box placed in a flume set up using two inclined planes sloping at different angles (Plate 1). Based on the use of relevant basal rheology and material parameters, all the six 3-D models, viz. MADFLOW, 3dDMM, DAN3D, Pastor, RASH3D and TITAN2D, produced simulations that resemble the test in respect to the overall reach of the sand flow and the broad shape of sand deposition. Although a better match with the test results does not necessary mean better performance of the model, all these models except TITAN2D, produced overall trend results that are consistent with the test results. TITAN2D predicted a greater degree of spreading of the deposit. Nevertheless, there are still notable variations in the spatial extent and profiles of deposition among the submissions. This may be related to the different assumptions for internal stress distribution and the numerical methods adopted.

Results of two dry sand flow experiments using a miniature flume carried out by the US Geological Survey (Plate 1) were available for the benchmarking exercise. Five teams attempted this case, four of which used 3-D models for the simulation. The simulation results of the FLATMODEL, 3dDMM and SHALTOP-2D are very similar, and they generally match well with the experimental results in respect of the debris runout and are able to capture the overall behaviour of the sand flows over the 3-dimensional shaped profile of the flume. Pastor also appears to give simulation results that are similar to the experiment, but the resolution of the result is notably coarser than those given by other models (probably affected by the DEM/boundary conditions adopted).



(a) Laboratory apparatus used for defected sand flow tests (after Pudasaini & Hutter 2007)



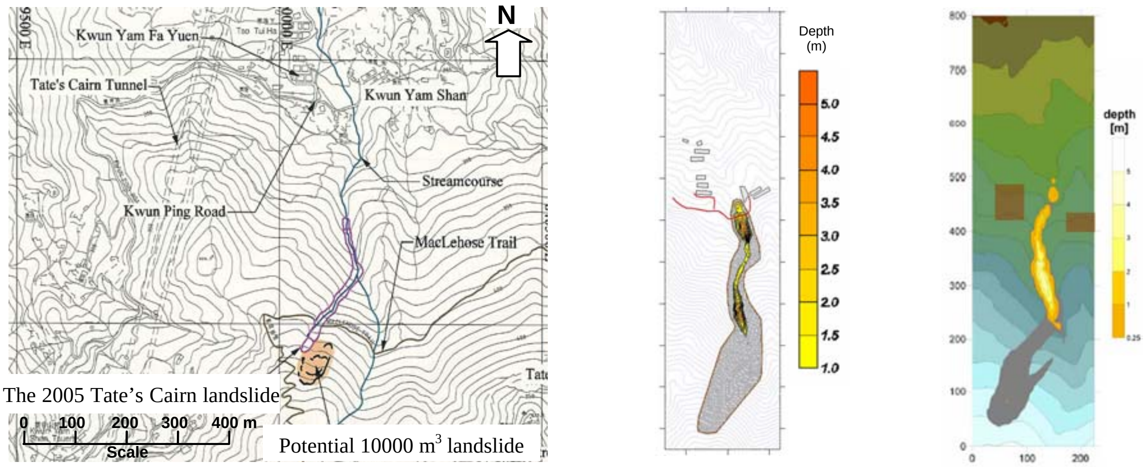
(b) US Geological Survey miniature flume (after Iverson et al. 2004)

Plate 1: Laboratory apparatuses for model calibration experiments

4.2 Landslide cases

The landslide cases available for benchmarking cover a wide range of topography, materials and characteristics of the landslide debris, as well as the mechanisms of the landslide failures and movement the landslide debris.

In back-analyses of the 2005 Tate's Cairn landslide, which involved a mobile debris flow of 1200 m^3 with a travel angle of about 23° putting a number of village houses at risk, four 3-D models, viz. DAN3D, DAN3D(NGI), 3dDMM and Pastor, using the Voellmy rheology matched the overall reach and debris flow path with similar material parameters. Two other 3-D models, viz. RASH3D and FLATMODEL, matched the runout with almost the same set of material parameters. However, the extent of the debris trail simulated by the FLATMODEL appears to be larger than those by others. NGI also back analysed the case using FLO-2D with a different inputs from others together with an inflow hydrograph derived from their DAN3D results. For the forward prediction of the Tate's Cairn landslide, the participants were asked to assess the mobility of a potential 10000 m^3 failure using six sets of Voellmy model parameters and assumptions on entrainment. Three models, viz. 3dDMM, DAN3D and DAN3D(NGI), give similar simulation results, though there are some discrepancies in the debris deposition depth. The predictions using FLATMODEL indicate a longer runout distances than others.



(a) Plan (b) Forward prediction by 3dDMM (c) by DAN3D

Figure 2: Tate's Cairn landslide and forward predictions by 3dDMM and DAN3D

The 1995 Fei Tsui Road landslide involved a planar failure of a cut slope, with 14000 m^3 of debris sliding on a shallow-dipping, weak discontinuity within the weathered rock. Findings of the forensic landslide investigation, including the distribution of the landslide debris were available for benchmarking. In this exercise, all models adopted frictional rheology. 3dDMM and DAN3D produced similar results with similar sets of parameters considering different frictional angles at the surface of rupture and elsewhere. MADFLOW, SHALTOP-2D, Pastor and RASH3D, using similar parameters and average frictional angles, also produced comparable results. A close examination of the results shows similar maximum debris deposition depths predicted by these models although DAN3D overestimated the final deposition area.

The 1990 Tsing Shan debris flow started with an initial hillside failure of about 350 m^3. As the debris travelled downslope along a drainage line, significant entrainment of the loose bouldery colluvium occurred. The final volume of the event is about 20000 m^3 with a travel angle of about 26° to the distal edge of the deposition zone. FLATMODEL, 3dDMM, DAN3D and Pastor produced consistent back-analyses of the event using Voellmy model with similar parameters. It was noted however that the approaches adopted by the different teams in simulating the entrainment are different. This was found to have made notable differences in the simulation results (e.g. the distribution of the landslide deposit).

The 2000 Tsing Shan debris flow also involved significant debris entrainment over a complex topography, which escalated the initial failure volume of about 150 m^3 at the source to a final volume of 1600 m^3. Apart from modelling of entrainment, the most challenging aspect in back-analyses of this event is the bifurcation of the debris at a local bend at about mid-way down the drainage line. FLATMODEL, 3dDMM and Pastor successfully simulated the event, including the bifurcated debris trails, using Vollemy rheology. 3dDMM and Pastor gave reasonable prediction of overall runout of the debris while the FLATMODEL produced a larger deposition area compared with other models.

The 2004 Thurwieser rock avalanche occurred in the Central Italian Alps. The event involved 2.2 M m^3 of rock fragments and the landslide travelled over 2.9 km from its source. The landslide scar/runout path was covered first by glacial ice, then glacial deposits as well as steep and hummocky rocky terrain. Back-analyses of this case demand capability to simulate different types of ground materials over a complex terrain. Based on their respective capability in 3-D modelling, the models of this case adopted different sets of basal rheology and parameters making a direct comparison of the model results difficult. Nevertheless, 3dDMM, DAN3D and Pastor produced consistent predictions on overall runout and debris flow paths. In particular, these three models matched the actual landslide well in terms of branching at the source and at the toe. In this respect, MADFLOW did not predict debris branches and possibly reflected a limitation of its formulations.

Readers may refer to the Review Report by Hungr et al. (2008) for assessments of other models, viz. TOCHNOG, PFC, Sassa-Wang and Wang, and on other landslides cases, viz. the 1995 Shum Wan landslide, the 1903 Frank Slide in Canada, the 1999 Sham Tseng San Tsuen debris flow and the 2005 Lo Wan debris flood.

5 SUMMARY FINDINGS

In this benchmarking exercise, four 3-D integrated models that allow for frictional and Voellmy basal rheologies, viz. DAN3D, 3dDMM, Pastor and RASH3d, consistently provided similar modelling results for a range of cases. In a number of cases where frictional rheology was adopted, SHALTOP-2D and MADFLOW also produced results that are consistent with the four models. It is promising in view of the different solution approaches adopted in these 3-D models. Although these models achieved consistent match on debris runout path, runout distance, time and overall shape of deposition profile, there are notable discrepancies in the final spatial extent and profile of the simulated debris deposition zones, probably related to the different assumptions made in respect of internal stress in the different integrated models.

TOCHNOG, RAMMS and TITAN2D provided results on a relatively small number of cases, in which the results are generally consistent with that of other models. The simulation results of FLATMODEL and Sassa-Wang are also broadly similar to that of other models although some discrepancies were noted. Comparison of the modelling results provided by PFC, FLO-2D and Wang are difficult as they adopt very different approaches/assumptions with other models.

3-D models have demonstrated notable advantages in simulation of landslide mobility in irregular hilly terrains. Some models allow for separation and merging of debris along the runout path, which is useful for modelling complicated cases. The approaches adopted for simulation of entrainment along the runout path are different for different models, which gave rise to inconsistence in modelling results and requiring further attention.

The overall consistent outcome of modelling results, despite that different solution approaches were adopted in the models developed/used, was found to be encouraging. The exercise served the intended purposes of taking stock of the progress made in numerical modelling of landslide debris mobility and the degree of commonality of the methods developed, and facilitating interaction among researchers and practitioners.

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Estimation of 'Design Event' Landslide Sources for the North Lantau Expressway and Yu Tung Road Natural Terrain Hazard Mitigation Works Study

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ABSTRACT

The June 2008 landslides in North Lantau occurred during a low frequency high intensity rainstorm which gave an ideal opportunity to review the 'design event' landslides within the study area. This paper describes the use of graduated time interval magnitude cumulative frequency (MCF) relationships that were derived from estimates of landslide volume based on aerial photograph interpretation of relict and historical landslides and direct field mapping of fresh landslides. The MCF relationship for the entire study area was 'anchored' by a landslide source of approximately 2,400 m^3. The 'anchor' landslide occurred during the June 2008 rainstorm that was estimated to have a return period of about 1 in 600 years for this location and resulted in a debris flow that partially blocked Yu Tung Road. Relationships were also derived for each catchment within the study area by scaling in proportion to the plan area of susceptible ground and the relative intensity of landslides within each catchment. These relationships were then tested against the data available for each catchment and good correlations were obtained in most instances. The approach outlined in this paper helped to place the potential landslide magnitude/frequency for each catchment within the broader context of a larger number of catchments with similar geomorphological characteristics. Given the inherent uncertainties in deriving suitable 'design event' source volumes, consideration of the characteristics of the potential debris path when selecting source locations is also an essential part of the design process that helps to increase the robustness of the overall mitigation strategy.

1 INTRODUCTION

The study area for the North Lantau Expressway (NLE) and Yu Tung Road natural terrain hazard mitigation works project is about 2.3 km^2 and is subdivided into 31 catchments (Figure 1). On 7 June 2008, a low frequency high intensity rainstorm occurred which triggered over 50 landslides within the study area. Based on analysis of local rain gauge data the rainstorm was estimated to have a return period of about 1 in 600 years for the Tung Chung area. The detached source volumes of the landslides ranged from about 30 m^3 to 2,400 m^3. The largest is located in Catchment 30 and gave rise to a channelized debris flow which travelled over 500 m and blocked the west bound carriageway of Yu Tung Road with about 3,000 m^3 of debris. This resulted in closure of the carriageway for about two months. Other landslides with source volumes up to about 350 m^3 gave rise to debris flows and debris floods in Catchments 15, 16 and 17 which travelled up to 800 m and resulted in flooding of a 200 m long section of NLE rendering this major link to Hong Kong International Airport impassable for about 8 hours.

A feature of some of these landslides was the unusually high mobility of the debris. This prompted a detailed review of the 'design events' (Ng et al. 2002) adopted as a basis for determining the mitigation works. This required:

- re-assesessment of the design landslide source volume for each catchment (Section 2), and
- assessment of the critical locations for the 'design' sources which could potentially result in the most onerous consequences (Section 3).

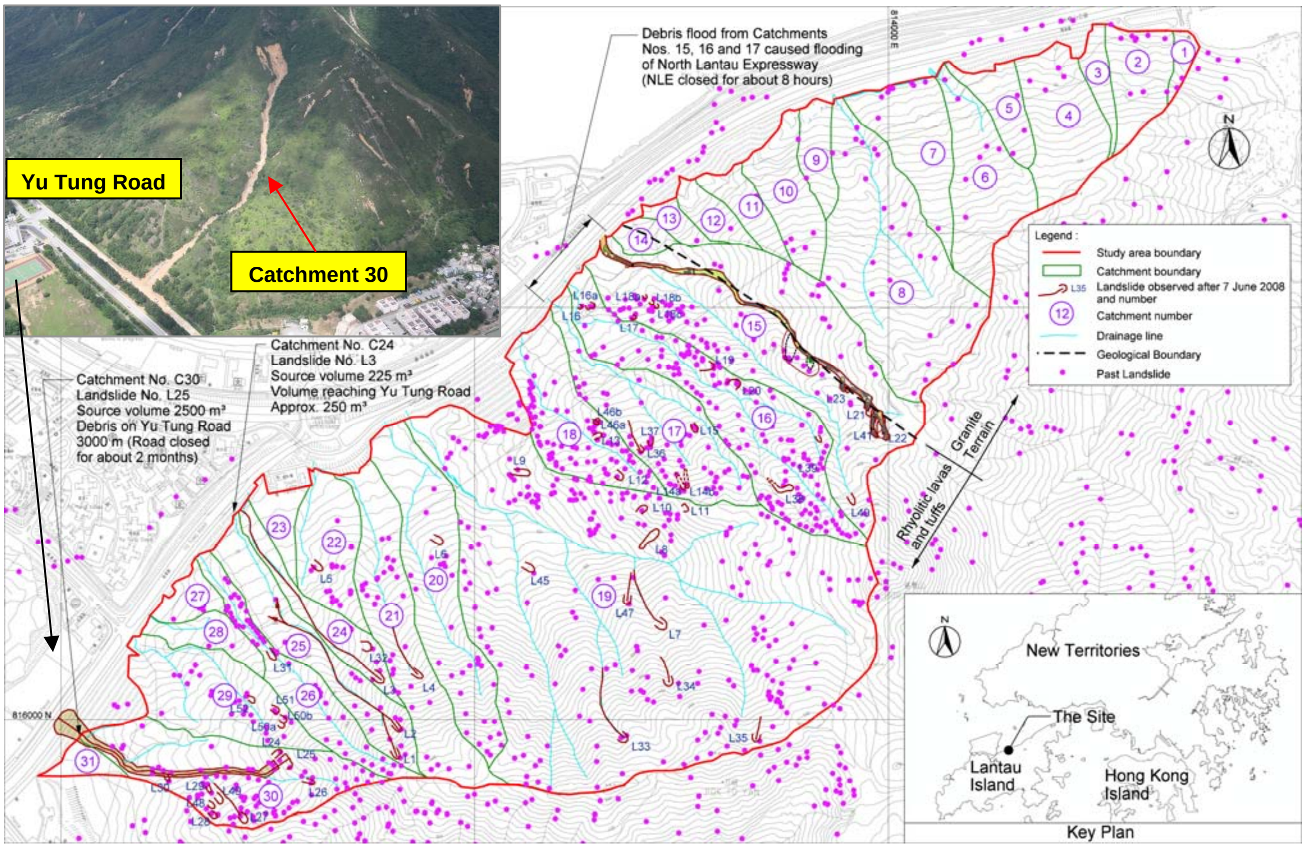


Figure 1: The Study Area

In view of the strategic importance of NLE and Yu Tung Road a 'worst credible' design event basis with a return period of 1 in 1000 years was adopted.

As can be seen from Figure 1, Catchments 15 to 30 which are underlain by predominantly lavas and tuffs support a far higher landslide density than Catchments 1 to 14 which are underlain by predominantly plutonic rocks. In addition, no significant landslide was recorded in Catchments 1 to 14 as a result of the 7 June 2008 rainstorm. Therefore, the focus of this paper is on Catchments 15 to 30.

2 DESIGN EVENT SOURCE VOLUME

2.1 Landslide inventory

According to the Enhanced Natural Terrain Landslide Inventory (ENTLI) database there are 574 recent and relict known or inferred landslides within Catchments 15 to 30. Other landslides have also been inferred during previous studies (Arup 2005), from further aerial photograph interpretation (API) during the course of the detailed design assessment and following the June 2008 rainstorm. After discounting obvious duplicates, a total of 851 recent and relict landslides have been identified within Catchments 15 to 30. Although there may be differences of opinion with regard to whether some of the features actually represent landslides, a cautious approach has been adopted for this study whereby all 851 landslides were included in the inventory on an equal basis.

Apart from some landslides that were mapped directly in the previous study (Arup 2005), the volume of each landslide was assessed from API by considering the calibrated volume/plan area relationships of previous landslides where detailed mapping and estimation of the source area volumes are known to have been carried out to a relatively high standard. These include the data set for the Tsing Shan Foothills Natural Terrain Landslide Study (MFJV 2003a) and investigations of notable recent landslides commissioned by GEO (Figure 2).

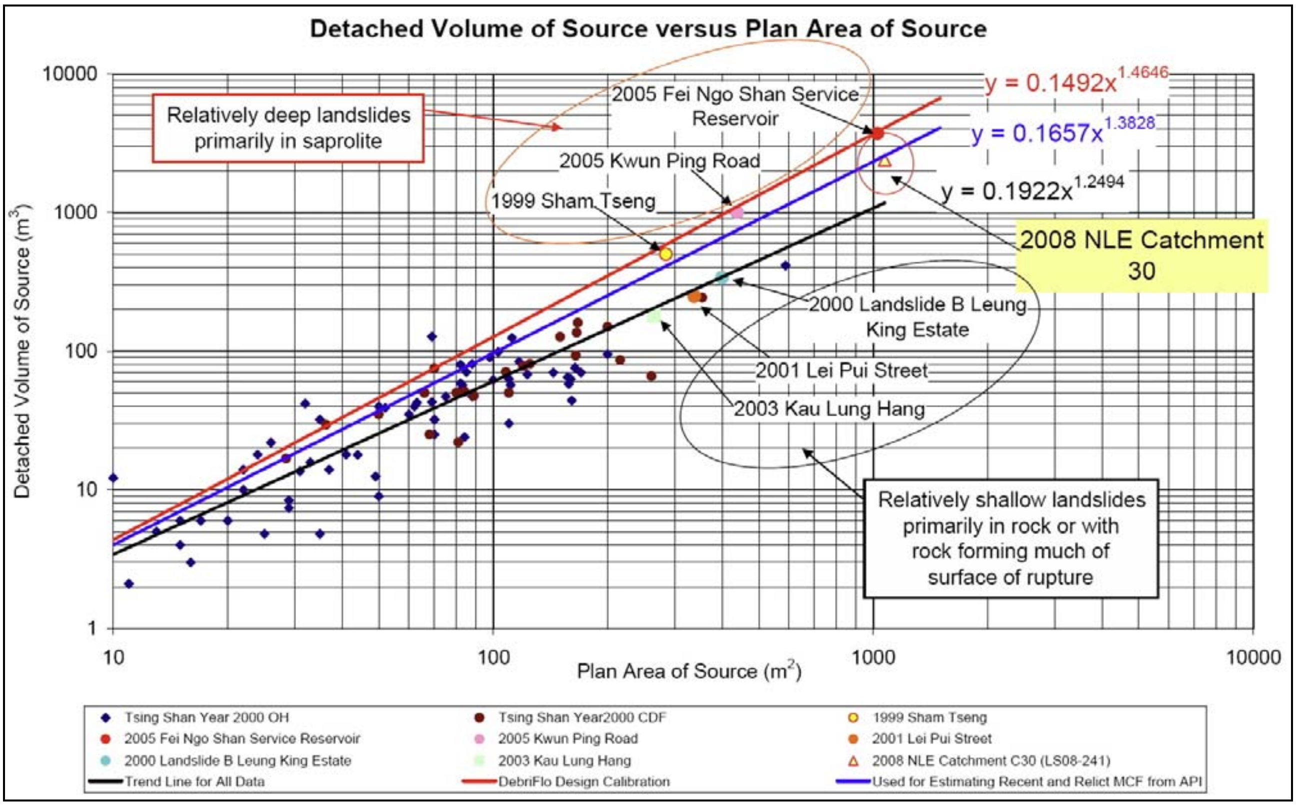


Figure 2: Detached volume of source versus plan area of source

The upper-bound line shown in Figure 2 was originally calibrated for estimating source area dimensions and initial landslide depths for use in dynamic debris modelling using Maunsell's 'Debriflo' program (Sun et al. 2005). This calibration line appears to be most appropriate for source volumes in excess of 500 m^3 which mainly involve relatively deep failures in saprolite. The trend line shown in Figure 2 for all the data is heavily biased towards relatively more frequent and therefore more numerous, shallow landslides with relatively small volumes. As it is not practicable to estimate the depths of relatively degraded relict landslides with a reasonable degree of reliability from API, the mid-range calibration line shown in Figure 2 was used to estimate the volumes of all the landslides in the inventory based on the area of the sources as estimated from API. The largest landslide that occurred in the June 2008 rainstorm plots close to the calibration line adopted.

2.2 Terrain types and general methodology adopted to assess landslide magnitude cumulative frequency

Review of the findings of the previous NTHS study (Arup 2005) and examination of the landslide distribution shown in Figure 1 indicated that the main terrain types where most landslides occur are:

- Ground generally steeper than 30º in the middle and upper reaches of the hillside which contains many small to large scale landslide scars in predominantly saprolite or saprolite blanketed by relatively thin colluvium, and
- Over-steepened (>37º) banks of incised stream courses which support many small scale landslides generally less than 100 m^3 in source volume.

Attempts were made to sub-divide these terrain units further on the basis of geomorphological characteristics and landslide distribution but no obvious sub-types of great significance could be distinguished, primarily due to the overwhelming extent and relative uniformity of the steep, saprolitic terrain that dominates the study area in Catchments 15 to 30. This is in contrast to many other areas previously studied in Hong Kong where the landscape is more geomorphologically diverse and where differences in landslide susceptibility and type can be seen more easily.

Given that Figure 1 indicates that there are differences in landslide intensity between catchments, it was considered that a suitable approach would be to:

- Determine a single MCF plot for the entire area of concern along the lines suggested by Hungr (2002) – see Section 2.3.
- Derive separate plots for each catchment by scaling on the basis of the proportion of susceptible land area in each catchment that gives rise to landslides (Section 2.4).
- In order to at least partially account for any underlying differences in landslide susceptibility between each catchment that may have been overlooked in the geomorphological review, consider the relative landslide density within each catchment (Section 2.5).
- Focus the 'design event' study on landslides with a reasonably large magnitude by scaling on the basis of landslide density after eliminating from consideration all landslides with a source volume less than 100 m^3 (Section 2.5).
- As a final check to verify that a reasonable MCF relationship has been established for each catchment based on the overall data set and scaling procedures, compare this relationship with the actual landslide dataset from each catchment (Section 2.5).

2.3 Initial graduated time interval MCF plot

The difficulties in establishing a representative time interval for a large landslide inventory are outlined in Hungr (2002) and in Section 6.2 of GEO (2007). In addition, the results of landslide age dating in Hong Kong suggest that some landslide scars are as much as 50,000 years old and that many of the relict scars that form a significant part of most landslide inventories may be of the order of 5,000 years in age (Sewell & Campbell 2005). To a certain extent, these difficulties can be compensated for by assigning different sampling intervals to each magnitude class. This approach was applied to determine a composite MCF plot for the Tsing Shan Foothills Study (MFJV 2003b) whereby different sampling intervals were applied by a process of trial and error to each magnitude range in order to derive a reasonably straight composite line on a log-log plot.

A similar approach has been applied to the inventory for Catchments 15 to 30. In addition, the MCF relationship was 'anchored' by the largest landslide that occurred within the area during the 1 in 600 year rainstorm of 7 June 2008 (Figure 3). This assumes that the landslide in Catchment 30 with a source volume of 2,400 m^3 can be taken as being reasonably representative of the entire susceptible area of Catchments 15 to 30 for a 1 in 600 year design event. Given the severe effect that this rainstorm had on other parts of north-west Lantau, this is considered to be a reasonable assumption. Figure 3 indicates that a landslide magnitude of about 3,200 m^3 would be representative of a 1 in 1,000 year event for the entire susceptible area of Catchments 15 to 30.

2.4 Scaling on the basis of susceptible land area for each catchment

As suggested by Hungr (2002), larger areas will exhibit higher landslide frequencies and vice versa which can be allowed for by adjusting the value of 'A' in the equation below in proportion to the relative size of the susceptible area in each catchment:

$$F_c = A.M^b \qquad (1)$$

where F_c = cumulative frequency, A = scale factor, M = landslide magnitude, and b defines the negative slope of the MCF line which Hungr (2002) suggests is fairly constant for a range of similar sites.

The area of susceptible land in each catchment was determined using GIS to plot different ranges of slope angle derived from the digital elevation model (DEM) and by comparing these ranges with the distribution of landslides. It was found that all landslides occur in areas where the DEM indicates that more than 50% of the ground surface is steeper than 30º when assessed on the basis of small units of the order of about 100 m^2 in plan. This kind of approach is necessary to compensate for the resolution of the DEM which commonly depicts a patchwork of different slope angle increments in the GIS near gradual breaks in slope and where the terrain is uneven.

The resulting MCF lines for each catchment are shown in Figure 3 which indicates initial 1 in 1000 year design events which range from about 300 m^3 to 2,100 m^3 depending on the area of susceptible ground in each catchment.

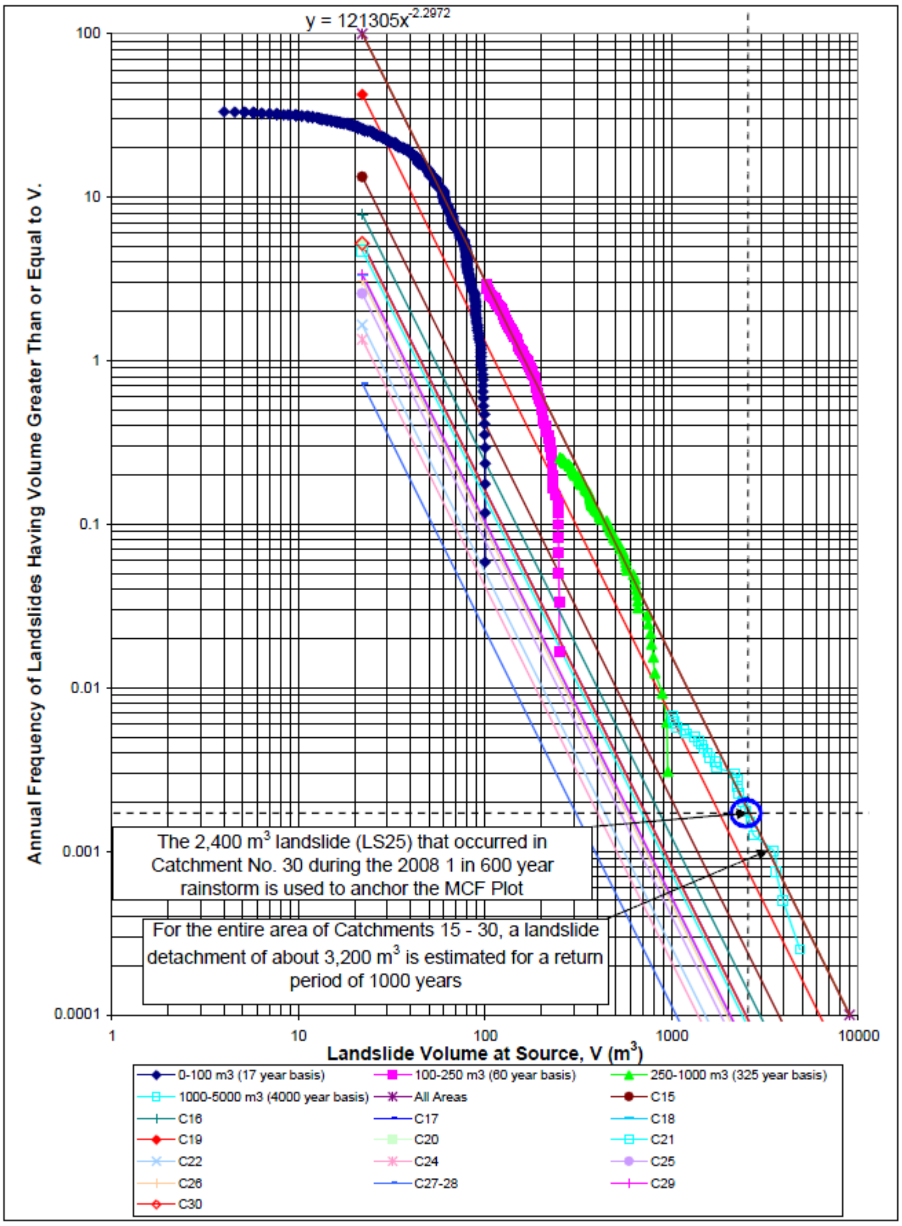


Figure 3: Graduated time interval MCF plots for combined data from Catchments Nos. 15 to 30

2.5 Scaling on the basis of landslide density for each catchment

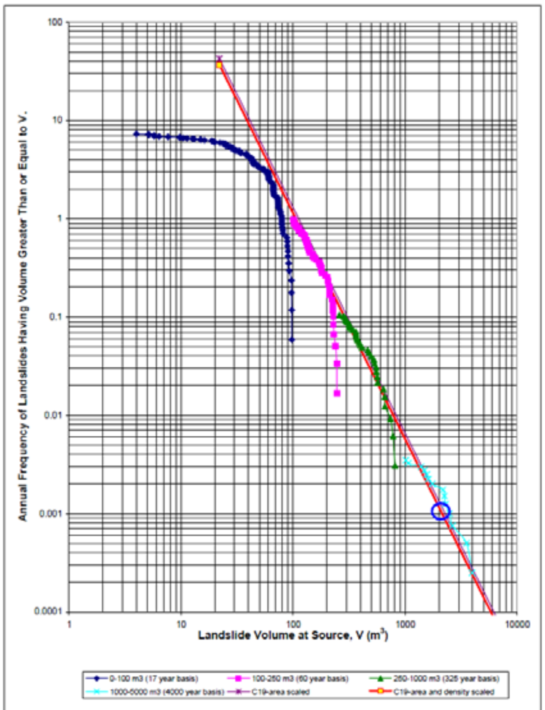
As indicated in Section 2.2 above, further scaling was carried out on the basis of the relative density of significant landslides with a magnitude greater than 100 m^3 to account for underlying differences in landslide susceptibility and landslide type in each catchment. Basic landslide statistics for Catchments 15 to 30 are shown in Table 1 and the final plots for Catchments 19 and 30 are shown in Figure 4 as examples which demonstrate the generally good fit between the MCF line obtained from scaling from the entire data set and the plots of the inventory from within the actual catchment.

3 LOCATION OF THE DESIGN EVENT

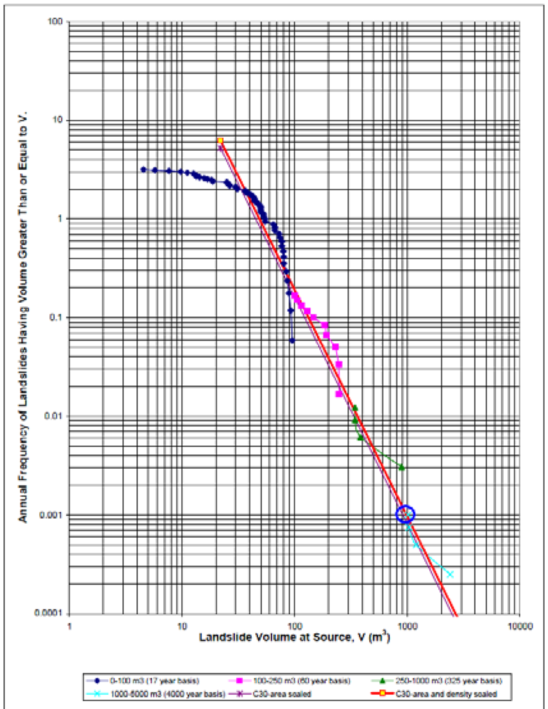
Methodologies were also developed to facilitate determination of the more critical locations of the source of the ‘design event’ in each catchment. Considerations included local engineering geomorphological characteristics, locations of previous landslide sources, gradient of potential debris path and entrainability factors such as occurrence of materials susceptible to entrainment, channelization ratio and length of the potentially entrainable path. All these factors influence landslide mobility and the volume and velocity of material that may reach the development of interest. Given the inherent uncertainties in deriving suitable ‘design event’ source volumes, consideration of the characteristics of the potential debris path when selecting source locations is an essential part of the design process that helps to increase the robustness of the overall mitigation strategy. This process also helps to focus attention on the more critical scenarios that need to be assessed in detail using dynamic debris mobility modelling.

Table 1: Basic statistics for Catchments 15 to 30

Catchment No.	Gross Plan Area (m^2)	Percentage of Total Plan Area	Total Plan Area of Susceptible Ground (m^2)	Percentage of Total Area of Susceptible Ground in Each Catchment	Number of Landslides	Landslide Density on Susceptible Ground (Landslides/km^2)	Number of Landslides >$100m^3$	Density of Landslides >100m3 on Susceptible Ground (Landslides/km^2)	Percentage Difference from Average
15	217,591	12.48%	166,879	13.29%	94	563	28	168	72.65%
16	120,997	6.94%	98,247	7.82%	153	1557	41	417	180.68%
17	89,611	5.14%	66,083	5.26%	54	817	23	348	150.69%
18	67,227	3.85%	41,712	3.32%	72	1726	16	384	166.08%
19	646,735	37.09%	532,493	42.41%	231	434	107	201	87.00%
20	100,713	5.78%	64,773	5.16%	28	432	16	247	106.95%
21	90,617	5.20%	59,393	4.73%	31	522	11	185	80.19%
22	45,822	2.63%	20,797	1.66%	17	817	6	289	124.91%
23	15,535	0.89%	0	0.00%	0	-	0	-	-
24	31,947	1.83%	16,949	1.35%	7	413	3	177	76.64%
25	61,790	3.54%	32,338	2.58%	19	588	9	278	120.50%
26	49,702	2.85%	38,763	3.09%	45	1161	5	129	55.85%
27	12,561	0.72%	3,249	0.26%	4	1231	0	0	-
28	19,163	1.10%	5,912	0.47%	1	169	0	0	-
29	73,723	4.23%	42,434	3.38%	23	542	7	165	71.42%
30	100,191	5.75%	65,572	5.22%	72	1098	18	275	118.85%
Total Area	1,743,925	100.00%	1,255,594	100.00%	851	678	290	231	-



Catchment No. 19



Catchment No. 30

Figure 4: Comparison of MCF lines derived by scaling from the entire data set with plots of the inventory for each catchment

4 CONCLUSIONS

The approach outlined in this paper helps place the potential landslide magnitude/frequency of an individual catchment within the broader context of a larger number of catchments with similar geomorphological characteristics. However, when using the ‘design event’ approach, considerable relevant experience needs to be applied when locating the ‘design’ source within a catchment to ensure that the location is credible and that the potential conditions likely to be experienced at the site boundary or location of mitigation works as a result of the ‘design event’ occurring are amongst the most onerous that can be realistically predicted.

With further testing and refinement by application to other NTHS Area Studies, the methodology presented in this paper could provide a basis for the assessment of latent landslide potential for catchments with few or no ‘historical’ landslides based on comparisons with a much larger number of broadly similar catchments which contain more frequent ‘historical’ landslides.

ACKNOWLEDGEMENTS

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A Case History of a Natural Terrain Landslide Investigation above Shek Lei Estate, Hong Kong

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ABSTRACT

The rainstorm of 1st September 2001 resulted in a large landslide on the hillside above Lei Pui Street, Shek Lei Estate, Kwai Chung, Kowloon. The landslide involved a total displaced volume of about 780 m³, with about 250 m³ of material detaching from the source. The debris entered a drainage line and developed into a channelised debris flow and travelled about 350 m, entraining substantial amounts of substrate material and destroying two squatter structures before coming to rest within a former quarry floor just above Lei Pui Street at the toe. This paper outlines the detailed landslide study carried out, which included detailed engineering geological mapping, ground investigation, and theoretical analyses to establish the probable causes of the landslide. The geomorphological setting of the affected hillside, including steep saprolitic upper hillside, extensive steep rock outcrop with well developed sheeting joints in the middle hillside, and dissected colluvial deposits in the lower hillside combined to generate a significant channelised debris flow event that was fortunate to avoid loss of life.

1 INTRODUCTION

In the late evening of 1 September 2001, during a Black Rainstorm, a landslide occurred on the natural hillside above Lei Pui Street, Shek Lei Estate, Kwai Chung (Figure 1 and Plate 1). As part of a continuing landslide investigation programme of the Government of the Hong Kong Special Administrative Region, a detailed study was carried out by Maunsell Geotechnical Services Limited (Maunsell, 2002) as consultants to the Geotechnical Engineering Office (GEO). The landslide involved a total detachment of about 250 m³ from the source area, located in an area of granite rock outcrop in the middle to upper hillside area. The detached material cascaded over a 25 m high steep rock slope and entered an ephemeral drainage line where it developed into a channelised debris flow. The landslide debris travelled a total distance of about 350 m down the drainage line entraining a further 500 m³ of material. The debris flow demolished two squatter structures (Plate 1) although fortunately the occupants had vacated their dwellings. Most of the debris came to rest within an active construction site (a former quarry area) above Lei Pui Street, and about 50 m³ of outwash material entered Lei Pui Street and adjacent open areas in Shek Lei Estate (Plate 1 and Figure 2).

2 THE SITE

The September 2001 landslide occurred on a NW facing hillside approximately 350 m from Golden Hill, the highest point (370 mPD) of a NE-SW trending ridge line separating Kwai Chung in the north from Kowloon Reservoir in the south (Figure 1). The landslide source was located about 135 m above Lei Pui Street at an elevation of about 220 mPD on the steep (approximately 41°) hillside, comprising rock outcrop with intermittent surficial colluvium deposits. The rock outcrop is extensive across the middle hillside area of the catchment and forms a series of steep planar slabs and steps which have been developed along major toppling and sheeting joints that strike across the hillside (Plate 1). Below this area, the natural hillside comprises less steep terrain with rounded spurs and valleys, where several ephemeral drainage lines converge into a streamcourse at the location of former squatter settlements. The streamcourse continues through an active construction site (former quarry) just above Lei Pui Street. The lower hillside, below the rock outcrop, slopes more gently towards Lei Pui Street at an average angle of about 30°. On this part of the hillside, there is much evidence of anthropogenic disturbance, mainly from former squatter structures (Plate 1).

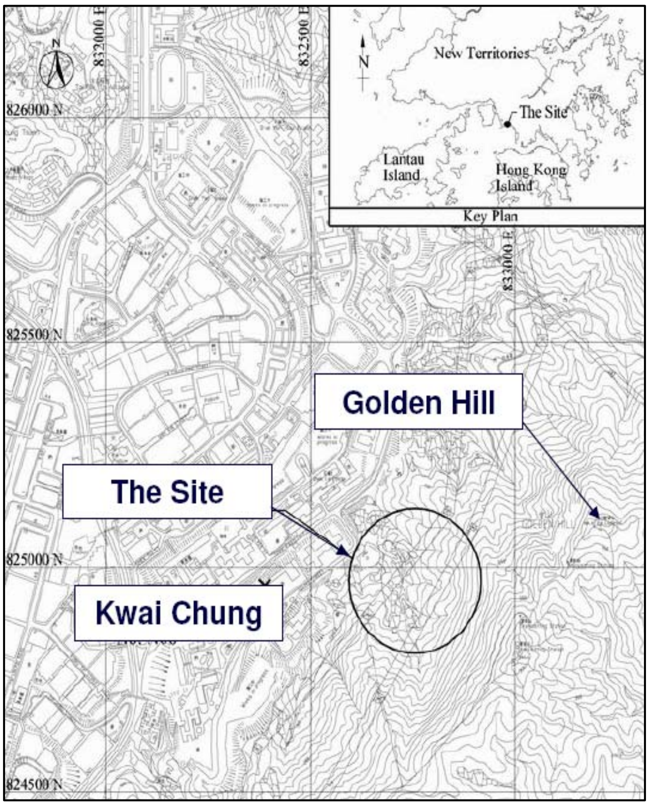

Figure 1: Landslide location plan

Plate 1: Oblique aerial view of the September 2001 landslide

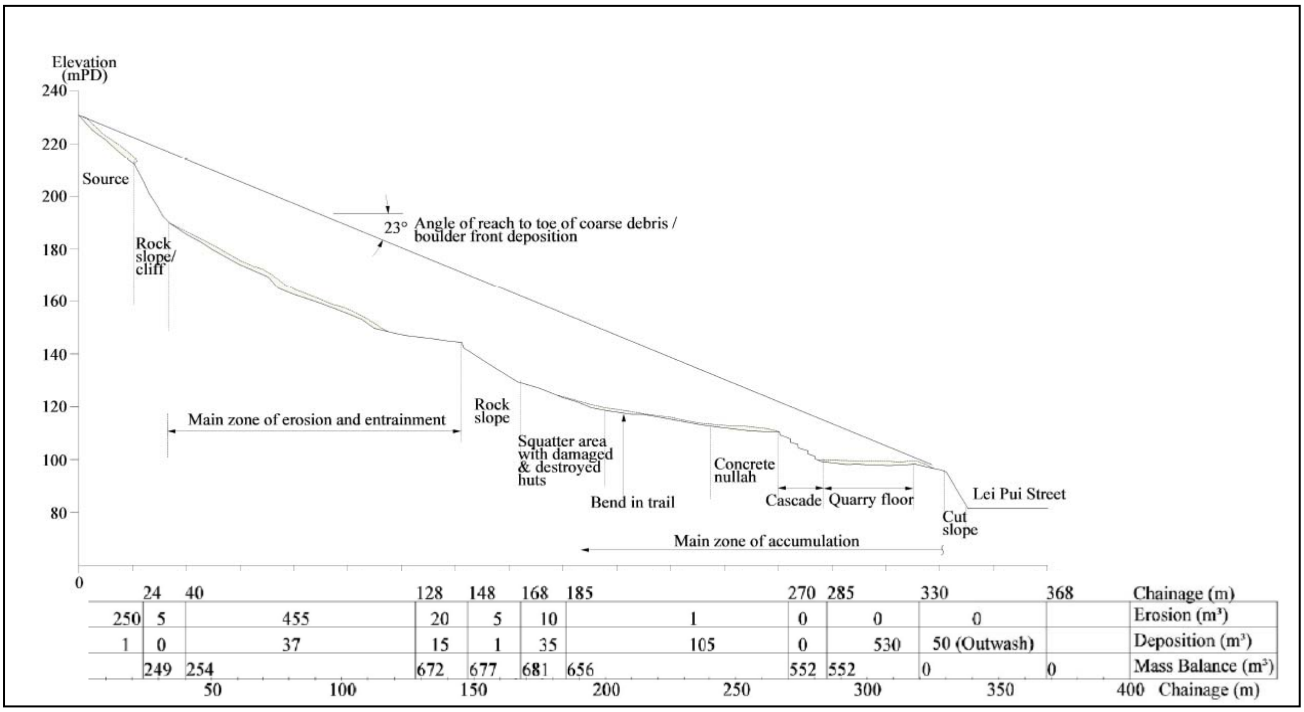

Figure 2: Cross section and mass balance of the September 2001 landslide

3 SITE HISTORY AND PAST INSTABILITIES

Desk study records and available aerial photographs of the site from 1949 onwards were reviewed to determine the site development history and evidence of previous instability. There is much evidence of squatter development on the lower hillside from 1949 onwards, peaking in the 1970's and early 1980's, before being mostly cleared during the late 1980's and 1990's (Plate 3). There is little change in the middle and upper hillside area where the September 2001 landslide source area is located. A review of the aerial photographs show that the landslide catchment contains several recent and relict landslides including seven landslides recorded at that time from the Natural Terrain Landslide Inventory (King, 1997), all of which were less than 20 m wide and below the area of rock outcrop where the September 2001 landslide source is located. From the aerial photographic interpretation (API) carried out, a further 18 relict landslides and 8 recent landslides were identified within the catchment of the 1 September 2001 landslide although most were located in the upper catchment above the granite outcrop.

4 THE SEPTEMBER 2001 LANDSLIDE AND POST FAILURE OBSERVATIONS

4.1 Landslide source area

The landslide debris trail is approximately 300 m long and some key features have been located against a chainage line as shown in Figure 1. The crown of the source area was located at approximately 233 mPD, on a natural hillside inclined at approximately 41°, within an area of granite outcrop with intermittent colluvial cover (Figure 2 and Plate 3). The landslide source was up to 26 m long (along the north flank) and 15 m wide at the base and the maximum depth of the landslide scar (measured normal to the ground surface) was about 1.5 m adjacent to the north flank. The volume of detached material was estimated to be approximately 250 m^3. The detached material mostly comprised a granite rock slab and the surface of rupture was formed by undulating sheeting joints that dip between about 31° and 45°. Along the north flank of the main scarp, the exposed profile comprises approximately 0.5 m of colluvium, overlying 1.0 m of moderately to slightly decomposed granite. Below this granite slab the undulating nature of the sheeting joint failure surface is exposed which was dilated up to about 100 mm and locally infilled with sandy silt sediment containing soil pipes up to 50 mm diameter (Plate 3). Below the source area there is a sharp convex break in slope where the gradient of the granite outcrop increase to up to about 60°.

4.2 Upper debris trail

After detachment, the granite rock slab and overlying colluvium cascaded over the 25 m high granite cliff. In the upper debris trail the slope gradient decreases sharply to an average of about 33° and extensive colluvial deposits have accumulated up to the base of the cliff. The colluvium at this location is up to 1 m thick and overlies about 0.3 m thickness of completely decomposed granite (CDG). The impact of the source debris eroded most of the colluvium and some of the underlying CDG exposing the moderately decomposed granite (MDG) bedrock. Further down the upper debris trail the gradient decreases to about 30°, and the colluvium thickness increases to about 2.5 m (Plate 2). Along this section of the trail the erosion channel bifurcates into two distinct channels. The upper debris trail was the main zone of entrainment with approximately 455 m^3 of material entrained and only about 37 m^3 deposited.

4.3 Lower debris trail

The lower debris trail is approximately 200 m long with an average angle of about 10°. The trail includes a 30 m section of relatively steep (about 40°) rock outcrop near the top, and a former quarry floor near the toe. Most debris was deposited within the lower debris trail with about 140 m^3 deposited in the drainage line and 530 m3 deposited on the former quarry floor. Approximately 50 m^3 of fine outwash material entered Lei Pui Street. Only 35 m^3 of material was entrained. The debris flow destroyed two squatter structures which were located on a small platform near the base of the rock outcrop (Plate 1). At about chainage 190 m, a confluence of the debris trail drainage line and the main catchment drainage line occurs where there is a sudden change in drainage line direction and decrease in gradient. At this area coarse debris was deposited on the outer bend.

5 GEOLOGICAL AND GEOMORPHOLOGICAL SETTING

5.1 Ground investigation

As part of the investigation, GI works were carried out in two phases and comprised 12 trial pits and six drillholes. The trial pits were located to investigate the nature and extent of the colluvial deposits along the toe of the rock outcrop and the substrate materials along the drainage line. The drillholes were all carried out within the source area to assess sub-surface structure and joint characteristics. This required the construction of scaffolding on the rock outcrop and a platform over the source area. Due to the difficult and remote location, the drillholes were carried out using a light weight portable drilling rig and using foam flush and a maximum depth of 15 m. Undulation, roughness or 'waviness' of surface of rupture was measured and its influence assessed on shear strength at the time of failure.

5.2 Geology

The underlying geology of the area comprises porphyritic fine-grained granite and equigranular medium-grained granite of the Needle Hill granite pluton (Sewell et al, 2000). Significant deposits of mixed debris flow and talus deposits are noted on the lower slope areas where the landslide debris trail descended. No geological faults are noted in the immediate vicinity to the landslide source, although north-south trending photogeological lineaments are shown within and adjacent to the study area catchment. The mapping and drilling works in the source area generally encountered fine-grained granite and indicated that sheeting joints with a similar orientation to the surface of rupture are found within 3 m of surface, with apertures of up to 200 mm. Beyond 4 m the joint aperture decreases rapidly and some silt/sand infilling was identified within joint apertures but no clay infilling was identified. The colluvium (talus) comprised approximately 75% angular to sub-angular cobbles and boulders of slightly to moderately decomposed granite and 25% silt, sand and gravel. The lack of decomposition of the cobble and boulder clasts and their 'loose' nature (being easily excavated by a hand pick), indicates that the colluvium is relatively recent (Plate 2). The fines content of the recent colluvium increased with distance away from the base of the rock outcrop.

5.3 Geomorphological setting

The main geomorphological findings of the detailed API and field mapping are shown in Figure 3. The catchment was divided into five terrain units, each with distinct geomorphological characteristics. Terrain Units 1 to 3 are of most significance and comprise the upper hillside, middle hillside rock outcrops, and lower hillside respectively. Terrain Unit 1 is the uppermost unit within the catchment and comprises rounded, west-facing slopes dipping at between 30° and 40° and containing several relict landslides and one recent landslide. Rock outcrops are sparse, and the nature of the topography suggests that the soil profile is relatively deep, comprising thin colluvium and saprolite. This terrain unit is primarily a zone of relict depletion which may be the source of much of the finer colluvium component observed in Terrain Unit 3. Terrain Unit 2 lies immediately below Terrain Unit 1 and consists primarily of fine-grained granite rock outcrops inclined at between about 40° and 50°, which have developed along prominent sheeting joints that are intermittently broken by a series of sub-linear cliffs and ledges running almost directly across the strike of the slabs. The geomorphology and stability of the hillside in Terrain Unit 2 is heavily influenced by the underlying rock structure with sheeting joints adversely orientated for stability with regard to planar sliding and sub-vertical joints adversely orientated for stability with regard to toppling failure and forming release surfaces and potential tension cracks, which allow plane sliding. Terrain Unit 3 lies below the lower edge of the rock outcrop of Terrain Unit 2. A substantial proportion of this terrain unit is covered by colluvium, which is greater than 1 m thick (Figure 15) and forms dissected lobes where drainage lines descend from Terrain Unit 2. The colluvium near the foot of the granite slabs below the landslide source is generally thin, and contains slightly to moderately decomposed clasts, with little saprolite development between the colluvium and bedrock. This indicates that the colluvium is relatively recent and that a process of frequent erosion by mass movement events has probably occurred near the foot of the cliff on a geomorphological timescale. Terrain Unit 3 is primarily a zone of deposition of debris from landslides originating in Terrain Units 1 and 2 above. Transportation of material occurs along drainage lines and streamcourses in the form of debris flows and hyper-concentrated stream flow.

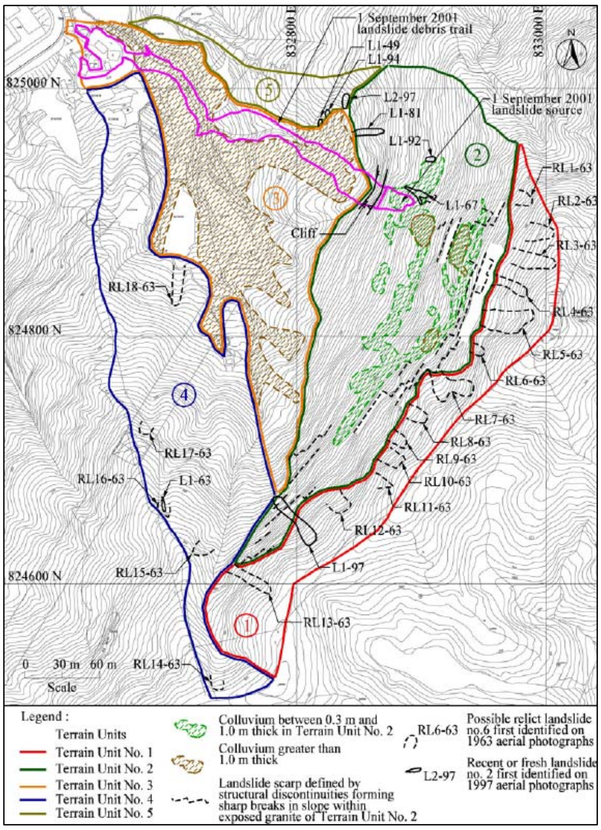

Figure 3: Geomorphological setting

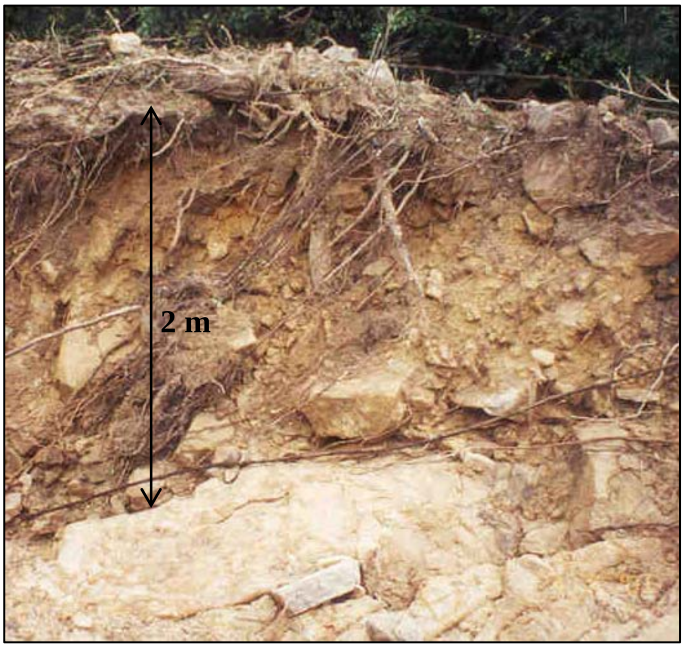

Plate 2: Bouldery colluvium located at upper part of terrain Unit 3

6 THEORETICAL STABILITY ANALYSIS

A stability analysis was carried out for a range of effective friction angles (ϕ') and apparent cohesion c'_a. The shear strength of the persistent, wavy sheeting joints was considered to be essentially frictional with an effective friction angle comprising a basic angle of friction (ϕ_b) and an additional joint roughness angle component (i) provided by small and large scale asperities. The work required to dilate joints over asperities is reflected in increased shear strength. Cleft water pressure and scale effects and joint-wall rock were also considered in the analysis (Barton, 1990). Derivation of the range of 'i' values from field measurements using circular plates (Richards & Cowlands, 1982) was carried out. The basic friction angle (ϕ_b), derived from shear box tests on saw-cut samples of granite from the post-landslide ground investigation, was found to vary from 32° to 42°. The results of the analyses indicate that for a high groundwater condition, failure could occur with either a high angle of friction (ϕ_b + i) or a high apparent cohesion (c'_a). Alternatively, failure could occur with a combination of relatively modest friction and apparent cohesion values under high groundwater conditions.

7 DIAGNOSIS OF THE PROBALE CAUSES OF THE LANDSLIDE

7.1 Initiation

The landslide initiated as a structurally controlled failure of a granite rock slab defined by two adversely orientated sets of rock joints, namely inclined sheeting joints and sub-vertical joints, which are sub-parallel to the strike of the sheeting joints and form overhanging release surfaces for sliding. It is likely that infiltration, particularly during the period of heavy rain preceding the failure, led to the build up of cleft water pressures in the sub-vertical rock joints and groundwater pressure along the basal joint planes causing the affected blocks to become unstable. The theoretical slope stability analyses have shown that sliding failure of the rock mass could occur with an unfavourable combination of factors, including friction angle, undulations and waviness of the sheeting joints and properties of the infill (where present), steepness of the sliding surface and build up of water pressure within the joints. Deterioration of the joint condition due to small incremental downhill movements over time, possibly due to heavy rainstorms in the past, and leading to dilation and partial infilling with sediment could have led to progressive loosening and gradual reduction in shearing resistance. Therefore apparent equilibrium may be maintained for many years until the joints weaken or key blocks move beyond a certain threshold, which then allows failure during a comparatively unexceptional rainstorm.

7.2 Entrainment and mobility

After initiation, the landslide debris comprising much rock material descended over a steep rock cliff immediately in front of the source area, thereby gaining further energy to continue down the hillside. Upon impacting the colluvium at the toe of the cliff, fluidisation of the material possibly occurred due to undrained loading effects and the generation of excess pore water pressures of the probably wet material. The entrained material flowed along an ephemeral drainage line, mixing with more surface water from intersecting ephemeral drainage lines and further increasing the debris mobility. In this upper section of the landslide debris trail about 455 m^3 of material was entrained from the predominantly colluvium substrate in this manner. The confluence of the debris flow with a streamcourse draining the main catchment probably increased the mobility of the debris by the injection of additional floodwater following the breaching of a temporary debris dam at this location where the debris mass had to change direction suddenly with a local decrease in gradient. In the lower debris trail mostly deposition took place with much of the debris depositing within the flat floor of the former quarry where the debris was able to spread and lose energy.

7.3 Geomorphological setting

The geomorphological setting of the September 2001 landslide was a key influence on the location and scale of the September 2001 landslide. Areas with similar geomorphological characteristics were divided into Terrain Units as outlined in Section 5.3, and some of these terrain units in combination form a hazard model that facilitates the development of a large channelised debris flow from a relatively modest initial detachment.

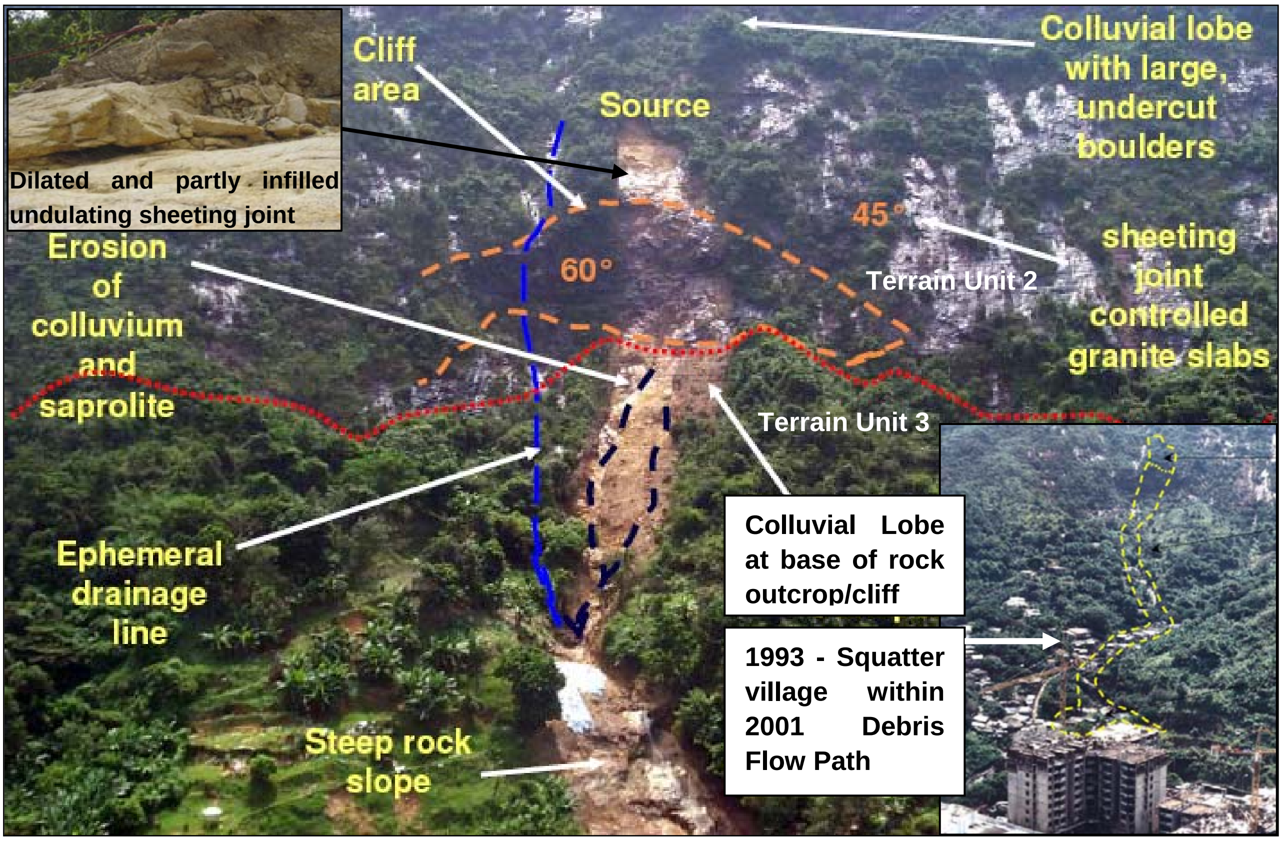

Plate 3: Setting of the source area and upper debris trail

Terrain unit 2 comprises the steep rock outcrop in the middle to upper hillside area and is a potential source area for rock slides, and is the probable source of much of the coarse colluvial material/talus. The local increase in rock outcrop gradient below the source area provided a daylighting surface for the sheeting joints which had significant evidence of progressive deterioration. Terrain unit 3 forms a predominantly depositional area on the less steeply inclined lower slope area and is where significant colluvial accumulation was identified, including the base of the steep rock slope under the September 2001 landslide. Terrain unit 1 at the upper catchment area was identified to be a source of many relict landslides probably within shallow colluvium/saprolite and is the probable source of much of the finer colluvial deposits. Ephemeral drainage lines in the upper debris trail began to channelised the mobilized debris and confluences with perennial drainage lines in the lower debris trail probably added floodwater.

8 CONCLUSIONS

The 1 September 2001 debris flow was probably triggered by infiltration through shallow colluvium and open joints in the rock outcrop surface and the subsequent development of cleft water pressures within joints in the underlying granite rock mass during severe rainfall preceding the failure. This resulted in a structurally controlled rock slide which was vulnerable to rain-induced failures owing to the unfavourable orientation of the sheeting joints steeply dipping joint sets that strike across the hillside to form potential dilation and release surfaces. Evidence of progressive deterioration and incremental movement was observed including undulating, dilated and partly infilled sheeting joints. Shearing resistance necessary to maintain equilibrium during previous periods of heavy rain was probably provided by the roughness and lack of large-scale persistence of the sheeting joints in addition to interlocking of blocks of rock. However, progressive deterioration probably reached the critical point, where the reduction in effective shear strength was sufficient to trigger the failure of rock slabs in the source area of the landslide during the heavy rainstorm on 1 September 2001. The rainstorm preceding the landslide was not exceptionally heavy which suggests that progressive deterioration of the marginally stable hillside at this location probably played a significant role in the landslide initiation.

ACKNOWLEDGEMENTS

This paper is published with the permission of the Head of the Geotechnical Engineering Office and the Director of Civil Engineering and Development of the Government of the Hong Kong Special Administrative Region.

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Natural Terrain Hazard Mitigation Works

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ABSTRACT

In Hong Kong, Natural Terrain Hazard (NTH) is one of the main potential hazards to both infrastructure and public safety. This concern has been continually risen because of the frequent occurrences of landslide at natural slopes during rainy season in recent years. The government has spent much resources to mitigate the hazards from natural terrains to an as low as reasonably practical (ALARP) level. Nevertheless, NTH mitigation works had been taken under in many recent civil projects in which extensive knowledge and experiences had been acquired from the actual construction of those works. We, Gammon Construction Limited as a main contractor, had successfully dealt with many challenges in NTH works contract in Hong Kong including design and build projects. From those projects, we have acquired precious experiences that we would like to share with the industry.

1 INTRODUCTION

NTH mitigation works contain different type of works such as boulder & debris flow fence installation, check dam construction, in-situ boulder stabilization works by means of buttress & dentition and other associated geotechnical works, which depends on the extent and nature of impact and adversity of the risk

As most of the NTH mitigation works are located at uphill where large plants and equipment could not be easily established, logistic and delivery method in association with the construction become paramount important to contractor's planning and management. Along with, safety, environmental, sustainability and preservation issues have also been the concerns for a responsible contractor although those issues pose great difficulties to contractor's resources planning and programming.

2 TENDER STAGE

A NTH assessment report includes the geomorphological and engineering maps covers the concerned area will be carried out by the geologist and the engineer at tender stage. Nominated contractors will obtain some aerial photos from the government department on their own necessity. As the contractors will base on the report and photos to interpret and estimate the resources required and the program, the accuracy and update of these information become very important to overall project risk and/or contractor risk.

More often, contractor will arrange a site visit to specified work locations so as to clearly identify the difficulties and risk of individual work. As some of the works will be done by sub-contractors for more cost-effective and more efficient, main contractor should also look carefully into the prices quote by the subcontractors to avoid wrong pricing which may lead to contractor's financial risk.

2.1 Engineer's design

As usual, contractor will prepare the cost estimation, program and planning based on the tender drawings and technical specification of the NTH mitigation works if the design is so-called "Conforming Design" or "Engineer's Design" by the engineering consultant. Generally speaking, the contractor has specified scope of risk as the extent of the works is clearly bound by the drawings and specification. However, for the design and build project, this may not be the case.

2.2 Design and build

For the design and build project, as the contractor should also take up the design responsibilities, we shall very carefully study the scope of work in accordance with the contract and then carry out the NTH assessment covers the concerned area. If our interpretation of the scope of works was different to what engineer and relevant authorities' expectation after the contract was awarded, disputes might involve and thus pose risk to contractor. One advantage for the design and build project is the contractor can coordinate with our designer directly and closely during early preliminary design stage and suggest them the most feasible and cost-effective construction methods in line with the designed type of works.

3 TYPE OF WORKS

NTH mitigation works contain different type of works such as boulder & debris flow fence installation, check dam construction, in-situ boulder stabilization works by means of buttress & dentition and other associated geotechnical works, which depends on the extent and nature of impact / adversity of the risk. Below are some photos:

3.1 Dentition

Dentition was proposed to stabilize individual unstable boulder locally. As most of them were located on the slope where assess road was not available. The delivery of readily mixed concrete become a difficulty to the construction. As a result, "hand-mix" concrete was adopted which was made at site near to the location of works. Workers mixed up the cement, aggregate, sand and water on site in accordance with the approved mix proportion and procedures. The mixed concrete then was transported manually to the location of the boulders. Temporary working platforms were required for those boulders located at steep slope where working space was not sufficient.

Photo 1: Dentition (Castle Peak Road project)

3.2 Buttress

As the buttress was located at uphill where large plants and equipment could not be easily established, helicopter was used to transport the construction materials such as steel reinforcement, plywood, timber, concrete and etc to the location of the works. The one done in Nam Wan Project used approx. 150 m3 of concrete and 6 tons. of re-bar. Cost was almost HK$5M in total. Since the serving of helicopter has many restrictions, for example, no buildings or obstructions shall be within 50 m diameter (on plan) of the landing of the helicopter, this method might not be adopted at somewhere.

Photo 2: Buttress (Nam Wan project)

3.3 Boulder fence

The materials of boulder fence were supplied from Europe due to inadequate reference to other source product, long lead-time of procurement required. Accurate site survey, coordination with supplier and designer are key factors to ensure the correct installation of the boulder fence. The erecting works need to be done by experienced workers and supervised by specialist. Mainly two types had been used in Hong Kong:

Photo 3: Boulder fence (Castle Peak Road project)

1) When the rock block hit the nets, the nets would then move through the post and activating the "brake device". The brakes absorb most of the energy and the stress in anchors would be reduced. As a result, a "braking distance" together with deformation of net is required in front of the net.

2) During the impact of rock onto nets, the forces are transferred to the system and finally into the upslope anchor ropes. Because the post base is designed as a fixed base, the anchor take up most of loads directly.

3.4 Check dam

More or less like a retaining wall with minipiles and tieback as foundation. Some antique and unidentified graves were found at the hillside of the check dam. Time-consuming to coordinate with relevant authorities, villagers and get approval and consent from them respectively.

Photo 4: Check dam (Deep Bay Link project)

3.5 Gabion wall

Challenging is to ensure the consistency of the pattern / size of the stone and keep the basket square and diaphragm straight to reach a pleasing appearance. Assemble the basket one by one, final gabion alignment must be checked before filling begins. Fill material must be as specified by the Engineer. It must have suitable compressive and durability to resist the loading, as well as the effects of water and weathering. Clean and hard stone was specified. The stones were placed evenly to minimize voids and ensure a pleasing appearance along the exposed faces. The baskets should be kept square and diaphragms straight during the filling process. Close liaison with the Engineer and close supervision of the workers were the key of success to the works.

Photo 5: Gabion wall (Deep Bay Link project)

3.6 Removal of boulders

Mechanical plants could not be delivered to break up the boulders, which located at uphill. Hand-held tools with assistance of Non-explosive chemical agent were used. Each boulder shall be identified in accordance with the boulder survey and/or mitigation report. 25mm diameter and 100mm deep holes were drilled on top surface of rock boulder by hand held pneumatic breaker. Wooden wedges were placed to prevent the rolling of rock fissure during splitting the boulder. Non-explosive blasting agent would be placed into each drilled holes to facilitate the splitting effect of boulder. Steel wedge was also placed into each drilled hole. After the "chemical reaction" over several hours, hit the end of steel wedges by hand held pneumatic breaker to split the large rock boulder into smaller pieces. The fragments of the breaking up boulders were then transported manually to downhill.

Photo 6: removal of boulders (Deep Bay Link project)

4 LOGISTICS AND DELIVERY

As many of the NTH mitigation works are located at uphill where large plants and equipment could not be established, logistic and delivery method in association with the construction become paramount important to contractor's planning and management.

Most often, manual lifting and transportation are adopted because of the inaccessibility of the lifting plant and lorry. For some locations, we could use the helicopter to deliver the materials if the surroundings are permissive. For the concreting works, we will adopt so-called "hand-mix" for easy transportation of materials.

5 SAFETY AND ENVIRONMENTAL ISSUES

Safety and environmental protection became hot issues of the industry in recent years and they are always the top priorities of our company. What we shall enhance and do more is so-called "Safety by Design" promoted by our company. For those design and build projects, we had closely coordinated with our designer to have a design not only consider its necessity, but also consider the feasibility and could also minimize the risk and hazard to our worker during construction. We shall make sure the construction wastes are completely cleaned up and no adverse impact to the natural environment and creatures.

6 SUSTAINABILITY AND PRESERVATION

Tree preservation is another issue that both consultant and contractor shall tackle in order to complete the project on time. As some of the NTH mitigation works are located at natural slopes, inevitably they may be obstructed by the existing trees. Pruning, transplant and other tree preservation work are thus may be required. Since the application for those works usually take not less than 3 months for relevant authorities to approve, early identification of the encroaching trees are very important to the overall program of the works. As some of the works were located adjacent to antique buildings or graves, relevant authorities' approvals were required and the procedures were often time-consuming.

7 CONCLUSIONS

We, Gammon Construction Limited as a main contractor had successfully dealt with many challenges in NTH works contract in Hong Kong including few design and build projects. From those projects, we have acquired precious knowledge and experiences that we hope the whole industry could be benefited.

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Modelling of Debris Flows for the North Lantau Expressway and Yu Tung Road Study Area

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ABSTRACT

Back analysis using dynamic debris mobility modelling was carried out to gain insight into factors affecting the mobility, deposition and entrainment characteristics of the debris flows that occurred during the 7 June 2008 rainstorm in North Lantau. This insight was then applied to the modelling of debris flow 'design events' for each catchment within the study area in order to formulate appropriate mitigation strategies. The largest debris flow had a source volume of approximately 2,400 m^3 which resulted in significant entrainment and deposition along the debris path and partial blockage of Yu Tung Road. An iterative process of calibration mapping and debris flow modelling was employed to refine estimates of mass balance and modelling assumptions which helped to increase confidence in the overall mitigation strategies for the study area (Devonald *et al*. 2009). Detailed mapping of debris widths and heights along the debris path, mapping of debris super-elevation at significant bends and a video recording of much of the debris flow event provided good data for calibration of the back analyses with particular regard to debris heights, widths and variations in velocity. This paper discusses the results of the models and recommends that allowance for uncertainty should be applied during subsequent design of mitigation measures. Much of this uncertainty can be attributed to the difficulty in obtaining a representative model of the morphology of the debris path, the complexity of actual debris flows and the practical limitations of any computer program that is used to provide approximations of debris flow motion. The use of more than one type of mobility modelling program gives additional insight which can be applied to increase the robustness of mitigation designs.

1 INTRODUCTION

On 7 June 2008, a rainstorm with an estimated return period of 1 in 600 years triggered over 50 landslides within the North Lantau Expressway (NLE) and Yu Tung Road natural terrain hazard mitigation works study area. The largest landslide, GEO Reference No. LS08-0241 (which was temporarily referenced as NLE No. L25), gave rise to a channelized debris flow that travelled over 500 m and blocked the west bound carriageway of Yu Tung Road with about 3,000 m^3 of debris (Plate 1). This resulted in closure of the carriageway for about two months. Other landslides with source volumes up to about 350 m^3 gave rise to debris flows and debris floods which travelled up to 800 m and resulted in flooding of a 200 m long section of NLE rendering this major link to Hong Kong International Airport impassable for about 8 hours.

A feature of some of these landslides was the unusually high mobility of the debris. This prompted a detailed review of the 'design events' (Tattersall *et al*., 2009), detailed mapping of the debris trails (Devonald *et al*., 2009) and detailed back analyses to gain better insight for the design of the mitigation works.

This paper gives an outline of the modelling of LS08-0241 with focus on the actual mobility of the debris flow and some features of the debris path that may have contributed to its relatively high mobility. A detailed technical discussion of the numerical models used for the back analyses is outside the scope of this paper.

Plate 1: Landslide LS08-0241

2 DEBRIS MODELLING

2.1 The models

Maunsell's Debriflo program (Sun *et al*. 2005) and GEO's 2d-DMM program (Kwan & Sun 2006) were used for the back analyses of LS08-0241 and subsequent dynamic modelling of the 'design events' for each catchment in the study area. The program DAN3D developed by McDougall & Hungr (2004) was also used to back analyse LS08-0241 in three dimensions, primarily to provide additional calibration of the mass balance estimates derived from surveying of the source of the landslide and mapping of the debris trail and also to provide an additional check on the findings of the much simpler models.

2.2 Morphology and mass balance

The morphology of the source and debris trail was defined by laser surveying and mapping of the source area, surveying of the longitudinal profile of the channel bed and a series of cross-sections based on detailed mapping of the debris trail. Devonald *et al*. (2009) describe the mapping of the landslide and estimations of mass balance which enabled the debris modelling to be calibrated against the site observations.

The surveyed longitudinal profile and cross sections from the field mapping were used to define the landslide and debris trial within the Debriflo model. A 'smoothed' version of the same morphology was used as input for the 2d-DMM model with the generally steep and irregular side slopes of the channel being simplified to a uniform 45º. However, the morphology defined in the DAN3D model was based on a digital elevation model (DEM) derived from the survey of the landslide source area and a 2 m grid DEM obtained from the Survey and Mapping Office of the Lands Department, Hong Kong (Figure 1).

All three models assumed a bulked volume of the source of the main debris flow pulse of about 2,400 m^3. This was estimated from the volume loss in the source area based on survey data after allowance for the volume of probable secondary failures based on site observations and evidence from the 'You Tube' video which shows several smaller debris pulses (www.youtube.com/watch?v=R2uTKyK1c9k) following the main pulse after several seconds had elapsed.

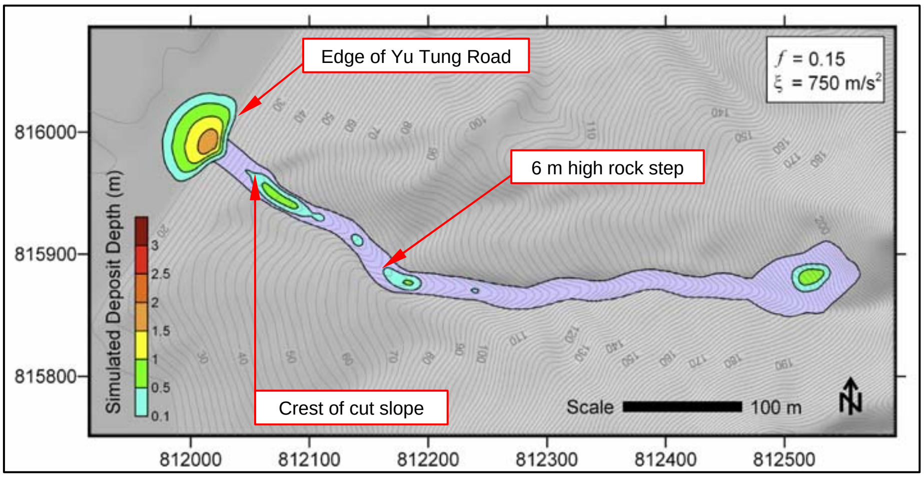

Figure 1: Example of Output from the DAN3D Model Showing Surface Morphology and Deposition Zones

The volume of material in the main debris pulse that reached the cut slope above Yu Tung Road was estimated to be about 2,300 m^3 based on assessments of mass balance from field mapping and knowledge of the approximate volume of material removed from Yu Tung Road in the main clean-up operation. Initially, the Debriflo model was iterated with the entrainment and deposition rates varied to match the field mapping estimates of erosion and deposition along the debris trail. Where anomalies were suspected, confirmatory site inspections were carried out and either the mapping data were revised or the model changed as appropriate.

The 'maximum active volume' (the volume of the main debris pulse that passes any point along the debris trail) was estimated to be about 2,900 m^3, with net entrainment predominating in the upper portion of the trail and net deposition being more common in the lower portion of the trail, particularly in segments with gentler gradients such as immediately above the 6 m high rock step and within 60 m of the crest of the cut slope (Figure 1). Further details of the mass balance profile are given in Devonald *et al.* (2009).

As the volume of debris reaching the cut slope in the main pulse was assessed to be similar to the bulked volume of debris from the main landslide that contributed to the main pulse, no entrainment or deposition was assumed in the 2d-DMM model to simplify the analysis. The DAN3D model assumed erosion rates to match approximately the mass balance profile from the Debriflo model and the field mapping. However, the amount of deposition and the volume of material reaching Yu Tung Road were directly calculated by the model.

2.3 Rheology and summary of results

The Voellmy rheological model (Hungr 1998; Kwan & Sun 2006) was adopted in all cases to simulate shear resistance of the debris in the channel below the source. The frictional resistance parameter is expressed in terms of the total stress bulk friction angle (ϕ_b) or by its tangent (f). The Voellmy turbulence coefficient (ξ) is a velocity dependent term that simulates the effect of irregularities in the channel profile which create turbulence that acts to 'damp' the velocity of the flow. A higher value simulates lower turbulence and vice versa, while very high values simulate negligible turbulence approximating to a friction-only model.

Based on previously calibrated case histories in Hong Kong, the Debriflo and 2d-DMM models adopted the Voellmy rheological model to simulate shear resistance within the landslide source area with $\phi_b = 23°$ and $\xi = 845$ m/s^2. In the DAN3D simulations a friction-only model was adopted for the source with $\phi_b = 28°$ primarily to simulate the deposition of about 300 m^3 of debris which was observed within the source. The results of the modelling are summarised in Table 1 below.

Table 1: Summary of results

Case	Model	Channel Parameters	Results
1	Debriflo	ϕ_b= 11.3°, ξ= 500 m/s^2	Debris flow stops close to crest of cut slope
	2d-DMM		
2	Debriflo	ϕ_b= 5.7°, ξ= 500 m/s^2	Velocity = 8.6 m/s at crest of cut slope
	2d-DMM		Velocity = 8.9 m/s at crest of cut slope
A	DAN3D		Velocity = 8.4 m/s at crest of cut slope, 2,500 m^3 debris on road
B	DAN3D	ϕ_b= 8.5°, ξ= 750 m/s^2	Velocity = 8.0 m/s at crest of cut slope, 2,300 m^3 debris on road

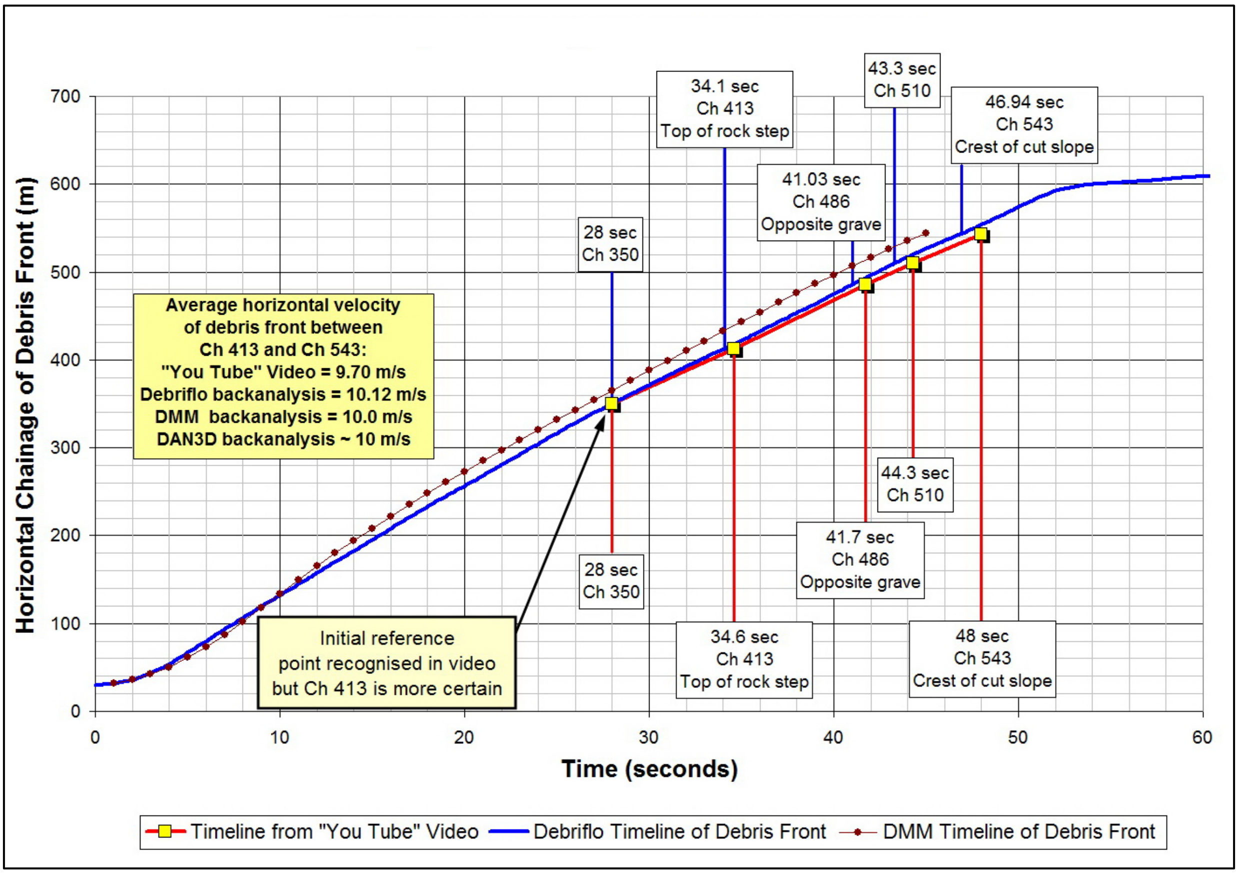

Figure 2: Comparison of timelines with the 'You Tube' video

2.4 Comparison of timelines with the 'You Tube' video data

The 'You Tube' video shows the progress of the main debris pulse in the lower half of the debris trail. This progress was timed in relation to landmarks that are recognizable on site and which can be related to the horizontal chainage established for field mapping and subsequent debris modelling. Figure 2 above shows a plot of the timelines for the Debriflo and 2d-DMM analyses of 'Case 2' and the timeline established from the video recording. From assessment of the velocity profiles in Figure 3 below it can be seen that the timelines for the DAN3D 'Cases A and B' also indicate an average velocity of about 10 m/s over the same distance. The video recording provides valuable evidence that the three models have produced reasonable simulations of the average debris velocity in the lower part of the debris trail.

2.5 Velocity profiles

Figure 3 below shows the velocity and average flow depth profiles from all three debris models for 'Case 2' and 'Cases A and B'. Also plotted are five velocity values estimated from field measurements of debris superelevation at significant bends in the debris trail (Devonald *et al*., 2009). Owing to the inherent difficulties in mapping and interpreting the field evidence in addition to the complexity of debris flows when compared to the relatively simple models used to simulate their behaviour, it would not be unusual to find much greater inconsistencies than are shown in Figure 3 between the field data collected and the results of the simulations and also between the simulations themselves. The generally good agreement between the velocity profiles and the field evidence over the greater part of the debris trail gives further assurance that the models have provided reasonable simulations of the forward motion of the main debris pulse.

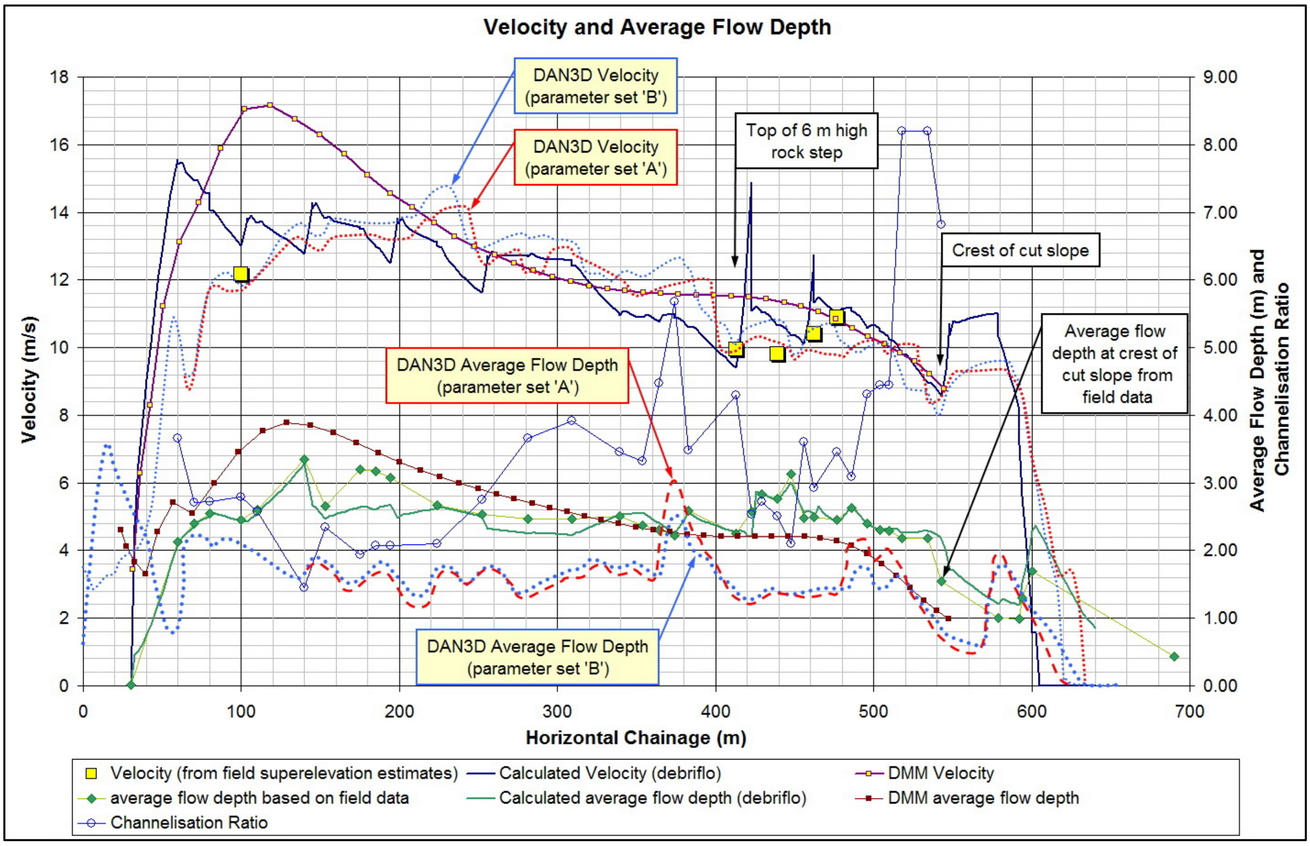

Figure 3: Velocity and average flow depth profiles

2.5 Average flow depths

Although there are similarities between the flow depth profiles of two of the models over about 400 m of the debris trail, the plots in Figure 3 highlight two key issues that are relevant to the design of natural terrain mitigation works.

The first issue is the fact that none of the models are in agreement with the estimated average flow depth of about 1.6 m at the crest of the cut slope where the debris front is decelerating on the flatter-lying ground behind the slope crest. The estimated depth was based on measurements of the maximum height of debris marks carried out on site using a tape measure, ranging rod and compass-clinometer (Plate 2). Although the estimate can only be regarded as approximate and will be subject to some degree of error, an underestimation in debris height of a certain percentage at the location of a barrier could lead to debris impact forces being underestimated by a corresponding amount.

Plate 2: Measurement of height of debris marks at crest of cut slope

Plate 3: Ravine in upper section of the debris trail

The second issue highlighted in Figure 3 is the relatively low flow depth profiles obtained from the DAN3D models. However, the formerly overgrown stream course which is deeply incised in places (Plates 1 and 3), could not be expected to be accurately depicted by the conventional DEM that is currently available. The main value of the DAN3D modelling was to provide an independent check on the work carried out here in Hong Kong. Although severely constrained by the available DEM, the model was able to confirm the relatively low operative rheological parameters, overall velocity profile, and distribution of the debris deposits from the June 2008 event. Of equal importance was the additional insight that the model brought to the assessment of other scenarios in different catchments for the design of the mitigation measures.

3 SUBSEQUENT MODELLING

A key consideration for the modelling of the design events in other catchments was the possible connection between unusually low frictional properties of the debris and its potential degree of confinement along the debris path. This might lead to increased dynamic undrained loading in irregular channels with a consequent drop in ϕ_b. Previous back analyses of some debris flows with significant segments having low channelization ratios (channel width/channel depth) – e.g. 1993 Tung Chung Landslide 5A2 (Ayotte & Hungr 1998) located in the next catchment to LS08-0241 and the 1999 Sham Tseng San Tsuen debris flow (MGSL 2000; FMSWJV 2005) – found that low rheological parameters identical to those adopted for the back analyses of 'Case 2' and 'Case A' in Table 1 above were appropriate. However, there is probably only a very low chance of being able to identify the true morphology of steep, overgrown stream courses from a study of topographic contours alone. Even though significant sections of LS08-0241 have channelization ratios less than 3.0 (Figure 3 and Plate 3), their deeply incised nature is not truly reflected in the DEM.

Inspections of major drainage lines in the other catchments were carried out (where safe access allowed), and the dynamic debris modelling work applied 'Case 2' parameters in addition to the more typical 'Case 1' parameters where the channelization ratio of the potential debris path as judged from topographic contours or site inspections was consistently close to or below a value of 4.0.

All cases where mitigation works were considered likely to be required were modeled using two different programs and the most onerous condition was adopted to increase the robustness of the mitigation design.

4 CONCLUSIONS

The findings of the back analyses indicate that relatively low rheological parameters were operative during the LS08-0241 debris flow in June 2008. The mobility of the debris flow and velocity profiles demonstrated by the three modelling programs are corroborated by field estimates of velocity from superelevation measurements and also by the video recording of the debris flow event in the lower portion of the stream course. The use of more than one type of mobility modelling program gives additional insight which can be applied to increase the robustness of mitigation designs. However, a good ground model with reliable input data based on careful site observations is necessary to make full use of the models.

The possibility that there may be a connection between high mobility and low channelization ratio in steep sided, incised channels may warrant further consideration. The wider application of LiDAR technology which has some capability to 'see through' thick vegetation would be of assistance in defining the morphology of potential debris flow paths more realistically.

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Case History of a Natural Terrain Landslide Investigation at Kwun Yam Shan, Hong Kong

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ABSTRACT

The intense rainstorm of 19th – 20th August 2005 resulted in a large natural terrain landslide on the Kwun Yam Shan hillside, below Tate's Ridge, Kowloon. The landslide involved a total displaced volume of about 2,350 m^3, with about 1,000 m^3 of material detaching from the source, severing a 10 m section of the MacLehose Trail directly below and entering a drainage line. The landslide developed into a channelized debris flow, with debris travelling about 330 m down the drainage line. Significant signs of distress were subsequently identified on the hillside above the source of the landslide, which included an extensive network of tension cracks, some with local maximum down throws of more than 1 m, bounding an area of distressed hillside within which smaller scale tension cracks, collapse features and erosion gullies were present. Failure of the distressed hillside could result in a sizeable landslide with a volume exceeding 10,000 m^3, with the risk that the debris would travel along the drainage line potentially causing catastrophic damage to the residential area about 500 m down the drainage line. This paper outlines the landslide study carried out, which included detailed engineering geological mapping, ground investigation, laboratory testing, and theoretical analyses to establish the probable causes of the landslide and the observed hillside distress and considers the recommendations made for risk mitigation.

1 INTRODUCTION

In the early morning of 22 August 2005, following heavy rainfall on 19 and 20 August 2005, which corresponded to a return period of about 50 years, a landslide was reported to have occurred on a natural hillside at Kwun Yam Shan, below Tate's Ridge (Figure 1a and Plates 1 and 2). As part of a continuing landslide investigation programme of the Government of the Hong Kong Special Administrative Region, a detailed study was carried out by Maunsell Geotechnical Services Limited as consultants to the Geotechnical Engineering Office (GEO).

The landslide involved a total displaced volume (terminology after Cruden & Varnes (1996)) of about 2,350 m^3 from the source area (about 1,000 m^3 of material detached from the source and about 1,350 m^3 of material remained in the source area below a series of tension cracks, see Figure 1b). The detached material entered an ephemeral drainage line, where it developed into a channelised debris flow. The landslide debris travelled a total distance of about 330 m down the drainage line. An approximately 10 m long section of the MacLehose Trail was severed about 30 m to the north of and below the toe of the August 2005 landslide. Significant signs of distress, in the form of an extensive system of tension cracks with local maximum down throws of more than 1 m, were subsequently identified on the hillside about 50 m uphill of the source area of the August 2005 landslide. These tension cracks defined an area of distressed hillside where smaller scale tension cracks, collapse features and erosion gullies were located.

2 THE SITE

The August 2005 landslide and adjacent area of distressed hillside are located on a north-northeast facing densely vegetated hillside inclined at between 30° and 35°. The site is within, and adjacent to, a densely vegetated, linear topographical depression (about 60 m wide by 100 m in length) between two rounded spurlines and at the head of an ephemeral drainage line. The 2005 landslide is above a convex break-in-slope.

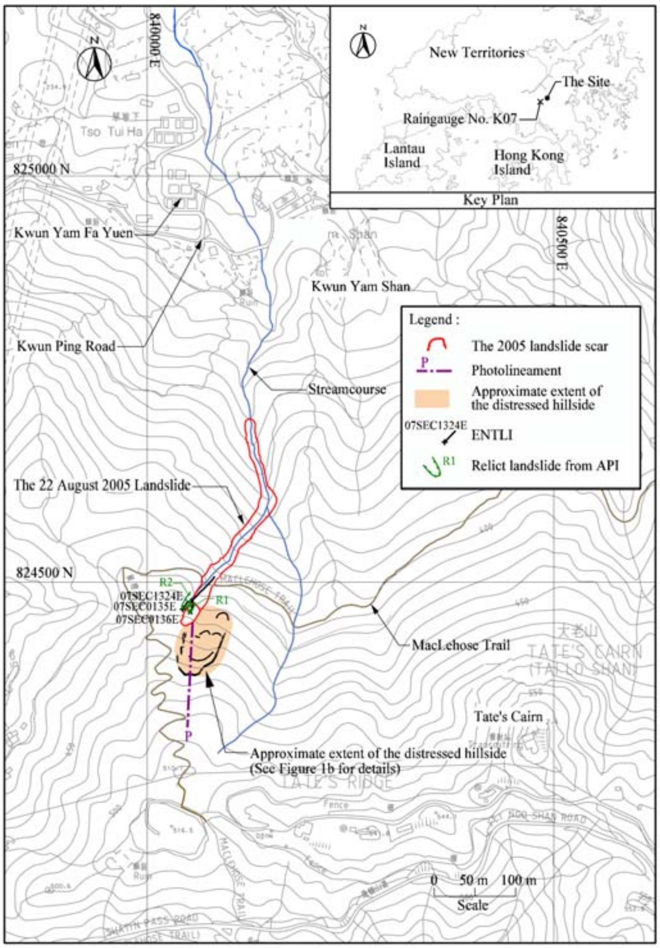

Figure 1a: Landslide location plan

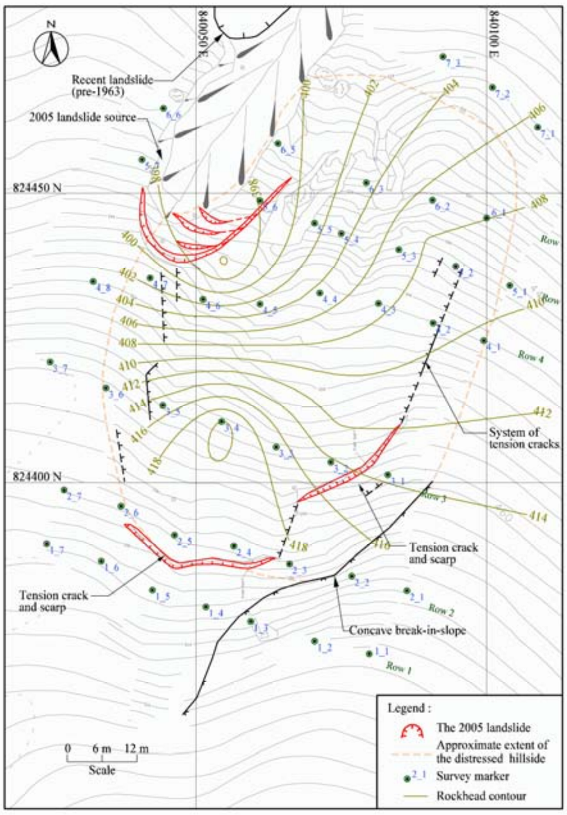

Figure 1b: Detailed field mapping

Plate 1: Oblique aerial view of the August 2005 landslide

Plate 2: Central part of the eastern flank of the main scarp within the distressed hillside

The distressed hillside is inclined at between 20° and 30° and is situated within the above mentioned topographical depression to the southeast of the August 2005 landslide. There is much anthropogenic disturbance particularly evidence of past military activities on the hillside, which includes abandoned military structures along Tate's Ridge and an extensive network of military tracks, trenches, pits and tunnels.

The nearest facility below the study area is Kwun Ping Road, which is about 500 m to the north of the August 2005 landslide. A few registered squatter structures are located in the vicinity of the streamcourse below Kwun Ping Road.

3 SITE HISTORY AND PAST INSTABILITIES

Available aerial photographs of the site from 1945 onwards were reviewed to determine the site setting, history and evidence of previous instability. The landslide site comprises an area of natural hillside located within a linear topographical depression between rounded spurlines. There is much evidence of anthropogenic disturbance across the hillside including military structures along the ridgeline and a number of chevron shaped wartime trenches and several footpaths that cross the site. Apart from a landslide that occurred between 1956 and 1963 below the toe of the 2005 landslide, there are no observable changes to the subject hillside between 1963 and present day, except for a general increase in the density of the vegetation.

According to the Natural Terrain Landslide Inventory (NTLI) (King 1999) and the Enhanced Natural Terrain Landslide Inventory (ENTLI) three past landslides (one recent and two relict) are indicated on the hillside affected by the August 2005 landslide (Figure 1). At the time of the failure, the August 2005 landslide and the distressed hillside were not located within a Historical Landslide Catchment (Wong *et al*, 2006) and there are no landslide incidents recorded in GEO's landslide database for the study area and immediate vicinity. A photolineament is observed to trend north–south at the western side of the distressed hillside (Figure 1a).

4 THE AUGUST 2005 LANDSLIDE AND POST-FAILURE OBSERVATIONS

4.1 Landslide source area

The total area occupied by the source of the August 2005 landslide measured about 36 m long by 22 m wide. The displaced material that remained below the crown and along the eastern flank of the source area measured about 26 m long and varied in widths from 8 m to 16 m. The series of stepped tension cracks within this material were about 0.5 m to 1 m high. The scar left by the material that detached from the source measured about 29 m long by 15 m wide with a maximum depth of about 5.5 m. The main scarp of the detached material was about 3 m high and inclined at an angle of about 60°. The landslide source is located immediately above a convex break-in-slope, where the terrain gradient changes from about 25° to 28° above to about 30° to 35° below. The break-in-slope probably developed as the result of the pre 1963 landslide, as identified in the aerial photographs, the scar of which is situated in front of the toe of the August 2005 landslide.

The detached material primarily comprised boulder-rich colluvium of about 3 m thick, described as young colluvium consisting of reddish brown slightly sandy silty clay with many cobbles and boulders of coarse ash crystal tuff. The material below the young colluvium was described as 'old colluvium' and comprised sandy clayey silt with some to many cobbles and boulders of coarse ash crystal tuff and abundant kaolin, in patches and veins. Local soil piping was observed in the lower portion of the old colluvium. Completely decomposed tuff (CDT) was observed in the floor of the landslide scar, which comprised light yellowish brown to reddish brown sandy clayey silt. The surface of rupture appears to have been primarily along the colluvium/saprolite interface with the deepest part of the landslide passing through the upper CDT that contains several thin kaolin veins.

The difference in elevations between the landslide source and the end of the debris trail was approximately 138 m, and the debris flow travelled a plan distance of 330 m. The travel angle (Wong & Ho 1996) of the landslide debris was about 24°.

4.2 Distressed hillside adjacent to the August 2005 landslide

The extensive system of tension cracks within the distressed hillside had a cumulative length of about 100 m and was observed to be continuous for about 70 m along the eastern and southern flanks of the distressed hillside, sub-parallel to and bounded by ridge lines on both sides. The western side of the distressed hillside is weakly delineated by a series of en-echelon discontinuous tension cracks and two trees at the western boundary of the distressed hillside were split along the tension cracks (Plate 3). The system of tension cracks included two prominent features (with throw up to about 1.1 m) of about 25 m to 30 m in length with a maximum horizontal separation of up to 300 mm that are open to a maximum depth of approximately 1.7 m. The tension cracks appeared fresh with an absence of any algal growth; though a portion of the southernmost tension crack (i.e. the uppermost scarp) appeared to have a weathered patina which is a sign of having been exposed for some time.

The distressed ground probably underwent internal distortion, fragmentation and dislocation forming minor terraces/benches akin to local breaks-in-slope. The distressed hillside contained several collapse features as well as gaps up to 100 mm across observed between boulders and the surrounding soil at several locations indicative of ground movement below the boulders. Two prominent gully features, with persistent seepage, are located immediately adjacent to the August 2005 landslide source area and were probably initiated from ground collapse into large subsurface voids and developed further following removal of fines from the boulder-rich colluvium and due to side-wall collapses.

Numerous underground cavities and soil pipes up to 1 m across are present below the subject hillside, some of which were only identified as a result of the cavities causing ground surface collapse. Most of the cavities appear to be present in the top 2 m below ground surface within the young colluvium. Soil pipes were also encountered during the GI in the old colluvium and incipient soil pipe development was observed throughout the CDT.

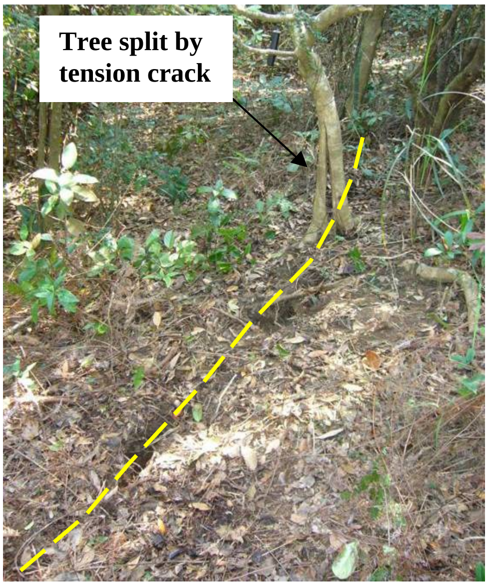

Plate 3: One of a Series of Stepped Tension Cracks on the Western Flank of the Distressed Hillside

5 EMERGENCY REMEDIAL WORKS ON THE AUGUST 2005 LANDSLIDE AND THE DISTRESSED HILLSIDE

Emergency remedial works were carried out on the August 2005 landslide source area and the hillside above, following the identification of the extensive distress in the hillside. The remedial works include installation of soil nails tied by reinforced concrete beams, provision of prescriptive raking drains and surface drainage channels. As a contingency measure, two flexible debris-resisting wire mesh barriers were also provided on the lower part of the hillside above Kwun Ping Road and a debris deflector wall was constructed along the streamcourse near Kwun Yam Fa Yuen (Figure 1a) to protect the residential area.

6 GROUND INVESTIGATION, GEOLOGY AND GEOMORPHOLOGY

6.1 Ground investigation

As part of the study, GI works which comprised five vertical drillholes, one inclined drillhole, nine trial pits and 41 GCO probes were carried out. Continuous Mazier samples were taken in all drillholes and were split and logged in the Public Works Central Laboratory (PWCL). Falling head permeability tests were carried out in the drillholes, and three double ring constant-head infiltration tests were carried out on the ground surface of the distressed hillside. Standpipes and piezometers were installed in drillholes and water levels were monitored using automatic vibrating wire piezometers. In-place inclinometers, equipped with real-time monitoring devices, were installed.

6.2 Geology

The regional geology of the site comprises a crystal and lithic tuff, coarse ash tuff, tuff-breccia, tuffite with local feldsparphyric rhyolite (Addison 1986; GCO 1986; Sewell et al. 2000) of the Cretaceous Mount Davis Formation. Based on the GI and field mapping the superficial deposits encountered comprised a young and an old colluvium with the underlying solid geology of the site comprising a coarse ash crystal tuff.

The young colluvium was encountered in all exploratory holes and was exposed in the tension cracks bounding the distressed hillside. The young colluvium is up to 2.9 m thick and generally comprises a soft to firm, reddish brown, slightly sandy clayey silt with some sub-angular to sub-rounded cobbles and occasional boulders of coarse ash crystal tuff and appears to have been derived entirely from landslide debris.

The old colluvium, which was between 2 m and 3 m thick, appears to be disturbed with discontinuous structure and apparent localised sedimentary layering, and typically comprises firm to stiff, reddish brown, light yellowish brown and greenish grey, sandy clayey silt with occasional quartz gravel and some to many cobbles of CDT. The origin of the old colluvium is uncertain; however, it is postulated to be remnant landslide debris from possible ancient landslide(s).

The solid geology comprises completely decomposed to moderately decomposed coarse ash crystal tuff. The depth to the base of the CDT ranged between 31 m to 48.3 m. Corestones of MDT were encountered within the CDT at various levels in the drillholes. Furthermore, it appears that the transition from CDT to MDT occurs with little or possibly no HDT. During the splitting of the Mazier samples, it was observed that even at the deepest levels (i.e. deeper than 25 m), small samples of the decomposed tuff would slake quite readily suggesting a completely decomposed grade.

6.3 Geomorphology

The landslide is situated at the head of a drainage line and immediately above a break-in-slope associated with an older landslide scar. The distressed hillside where extensive tension cracks are present is situated within the linear topographical depression through which poorly defined ephemeral drainage lines run. The mouth of the depression is marked by a series of convex and concave breaks-in-slope with the terrain inclination observed to steepen from between 20° and 25° above the breaks-in-slope to between 30° and 35° below.

The hummocky ground surface morphology of the distressed hillside is indicative of a colluvial terrain with a history of instability. However, the footpaths and military activities on the hillside might also have contributed to the hummocky appearance of the hillside. The volcanic terrain is deeply weathered with up to 30 m of saprolite above rock. The presence of photolineaments and evidence of a possible structural control to landslides in the vicinity and possible parallel breaks-in-slope along the eastern flank of the depression suggest that there may be an underlying structural influence on slope instability within the study area.

7 THEORETICAL STABILITY ANALYSES

The results of theoretical stability on the August 2005 landslide, using shear strength parameters of $c' = 3$ kPa and $\phi' = 30°$ for both young and old colluvium that were derived from laboratory tests, suggest that a rise of about 2.5 m above the colluvium/CDT interface, would have been sufficient to cause failure.

Theoretical stability analyses indicate that a shallow failure (i.e. along the interface between old colluvium and CDT) would be plausible with a perched groundwater of about 1.3 m below ground surface. A perched water level of 1.4 m below ground was recorded in the upper piezometer at drillhole No. BH2 after the September 2006 rainstorm, which suggests the perched water level required for the shallow failure to initiate, is viable.

8 DIAGNOSIS OF THE PROBABLE CAUSES OF THE LANDSLIDE AND THE DISTRESS ON THE HILLSIDE

8.1 The August 2005 landslide

The close correlation between the August 2005 landslide and the preceding heavy rainstorm suggests that the failure was triggered by rainfall. The likely mechanism of failure involved a wetting up of the ground mass due to direct infiltration and subsurface seepage from the uphill catchment leading to the transient development of perched groundwater pressure above the old colluvium/CDT interface.

The geomorphological setting of the August 2005 landslide is likely to have played a key role in the failure. The landslide occurred above a smaller pre-1963 landslide, the crown of which formed a prominent break-in-slope, which possibly defined the pre-landslide upper limit of a zone of instability. The landslide is located at the head of an ephemeral drainage line, where convergent surface runoff and subsurface seepage would likely occur during periods of heavy rainfall. The previous instabilities and progressive degradation of the hillside appears to be typical of a hillside retreat land forming process.

8.2 The distress on the hillside

The majority of the tension cracks appeared fresh and may have occurred subsequent to the August 2005 landslide. Evidence on the distressed hillside, *viz.* hummocky morphology, leaning trees, possible weathered patina on the southernmost tension crack, however, suggests that the initial movement of the distressed hillside most likely pre-dates the August 2005 landslide and that initial development of at least part of the tension cracks may have been a result of earlier rainstorms. The morphological settings are indicative of a large ground mass undergoing a complex movement in terms of its kinematics. The presence of numerous underground cavities and soil pipes, some with layered deposits and others blocked, together with ground surface collapse features, also indicate that the distressed hillside has a complicated and highly variable hydrogeology.

The presence of a basal slip surface within the distressed hillside could not be positively established by the available GI information. The series of en echelon tension cracks along the western flank of the distressed hillside are possibly related to a poorly developed or discontinuous slip surface, possibly below separate blocks of groundmass with internal distortions. The theoretical stability analyses suggest that shallow instability of the distressed hillside appears to be a likely mechanism. While this appears to be a credible and plausible failure mechanism, it does not satisfactorily explain some of the observations, e.g. the very persistent tension cracks and other displacement features in a distressed hillside that is inclined at an average angle of only 20°. The mode of displacement of the groundmass within the distressed hillside is likely to be highly complex. Based on the area of the distressed hillside bounded by the tension cracks and a failure surface of at least 3 m below ground, i.e. similar to that of the 2005 landslide, catastrophic failure from the distressed hillside could result in a landslide with a potential volume greater than 10 000 m^3. With the emergency works in place, the risk of such a massive failure was significantly reduced.

9 CONCLUSIONS

The August 2005 landslide was triggered by intense rainfall and involved the failure of up to 5 m of colluvium, primarily along the old colluvium/CDT interface in an uncontrolled manner giving rise to fast-moving debris that travelled a total distance of 330 m down the drainage line below. The landslide location immediately above a break-in-slope on terrain comprising deeply weathered rock, is typical of an erosion front advancing uphill through the natural hillside as part of the progressive degradation of the hillside in the form of a hillside retreat process. Both surface runoff and subsurface flow would be directed towards the landslide site at the head of a drainage line and the presence of an interconnected subsurface drainage network would have promoted rapid groundwater flow to the landslide site.

The development of the extensive tension cracks with intermittent surface opening and other displacement structures within the distressed hillside are indicative of a large ground mass undergoing a complex mode of instability.

Hillsides with major distress pose a significant safety concern where the landslide debris in the event of a failure could reach occupied or frequently used facilities, as the ground condition is liable to deteriorate under successive rainstorms which may result in complete detachment of the unstable ground mass. The identification of vulnerable hillsides with major distress, which could have a major bearing on risk assessment, is a challenge to the practitioners.

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The Application of Remote Sensing Techniques for Movement Monitoring of Natural Terrain

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ABSTRACT

One of the difficulties associated with the monitoring of movement of natural terrain has always been the necessity for instrumentation and surveying personnel to be physically present at the monitoring location, which may often be difficult or dangerous to access. Remote sensing techniques that allow natural terrain movement to be monitored from a distance have become technically feasible in recent years. This paper highlights a number of the commercially available remote sensing techniques considered to have potential for application in Hong Kong, including terrestrial and satellite-based radar interferometry (InSAR), as well as airborne techniques such as LiDAR. The principles of operation of the techniques are described, and the paper illustrates their use with several case studies describing relevant applications on overseas projects.

1 INTRODUCTION

The hilly terrain and high seasonal rainfall mean that landslides are one of the most common natural hazards in Hong Kong, and the high population density has placed a premium on available space for construction, which has led to a large proportion of buildings and infrastructure being constructed adjacent to slopes.

Since the 1970s, the government of Hong Kong has had in place a large Landslip Prevention and Mitigation (LPM) programme to control the design of new man-made slopes, and to maintain, inspect and upgrade existing man-made slopes. It is widely acknowledged that this programme has been very successful, and the risk of major failures of man-made slopes has been dramatically reduced.

However, the risk of failure in natural terrain remains a significant problem. Chan (2008) notes that a review of historical aerial photos identified some 16,000 landslides that had occurred over the last 50 years on natural terrain, and a further, more detailed, investigation showed about 2,700 natural slopes that were presenting hazards to existing buildings and transport corridors. In late 2007, the government launched a new Landslip Prevention and Mitigation Programme (LPMitP) that will, amongst other activities, systematically work on prioritising and reducing the natural terrain landslide risks.

The increased emphasis on risk management of natural terrain slopes will undoubtedly bring with it an increased requirement for movement monitoring of natural terrain. One of the difficulties associated with the monitoring of movement of natural terrain has always been the necessity for instrumentation and surveying personnel to be physically present at the monitoring location, which may often be difficult or dangerous to access.

Remote sensing techniques that allow natural terrain movement to be monitored from a distance, whether from space, from an aircraft or simply from a distant ground-based location eliminate the requirement for regular physical access to the area monitored, thus making it practical to monitor locations that were previously too dangerous, impractical or expensive to monitor using more direct methods.

In recent years, various techniques for remote sensing have been developed and are now moving out of the field of academic research and are finding their way into the commercial marketplace. In this paper, we review several techniques considered practical and applicable for natural terrain monitoring in Hong Kong.

The remote sensing techniques addressed in this paper have been grouped into three categories – space, airborne and ground-based. Emphasis has been placed on techniques which are commercially available, rather than techniques that still under development as university research topics.

2 MOVEMENT MONITORING FROM SPACE

Interferometric analysis of Satellite Aperture Radar images (InSAR) allows ground displacements to be measured from space with millimetre-level accuracy over areas covering thousands of square kilometres. In 1991, the European Space Agency (ESA) launched ERS-1, the first earth observation satellite equipped with Synthetic Aperture Radar (SAR) equipment, and this has since been followed by several others (Table 1).

The SAR sensors onboard earth observation satellites transmit electromagnetic radiation signals at microwave frequencies and record the intensity (amplitude) and time delay (phase) of the signal reflected from the Earth's surface and the features upon it.

The phase component contains information about the height of the ground at the time the image is acquired. As the satellite orbits the earth and subsequent SAR images are acquired over the same location it is possible, through interferometric processing, to map very small changes in topography (deformation) over time (Figure 1).

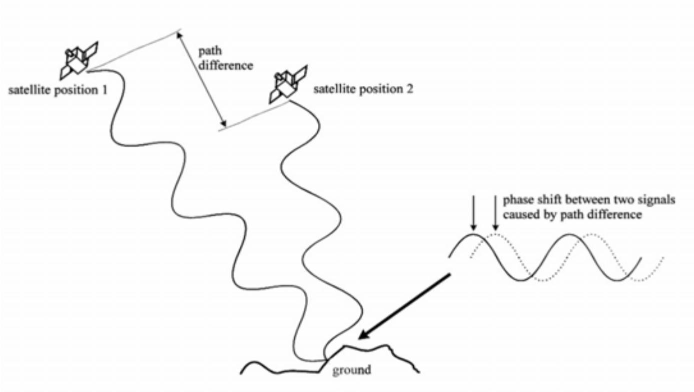

Figure 1: SAR satellite imaging geometry during successive orbit cycles.

Satellite InSAR techniques are capable of mapping deformation ranging from millimetres to metres over periods spanning months, years and even decades. The techniques are routinely used in a diverse range of applications in a wide variety of market sectors, including civil engineering, oil and gas, mining, geohazards and utilities. As satellite InSAR involves the processing of existing SAR images, the technique offers the unique ability of being able to look backwards in time to analyse historical motion.

Different satellites have different capabilities in terms of radar wavelength, image size, spatial resolutions and revisit times. In general, however, spatial resolution (pixel size) is of the order of metres to tens of metres, and revisit times are of the order of several weeks. The radar wavelength determines the maximum vertical resolution that may be achieved. Table 1 summarises the key characteristics of the main SAR satellites.

The three main satellite InSAR techniques available are conventional differential interferometry (DifSAR), persistent scatterer interferometry (PSI), and artificial reflector interferometry (CRInSAR and CATInSAR). These techniques are detailed further in the following paragraphs.

Table 1: Characteristics of the main SAR satellites

SAR Satellite	*Radar wavelength*	*Lifespan*	*Incidence angle range (°)*	*Spatial resolution range (m)*	*Revisit time (days)*	*Image size range (km)*
ERS-1	5.6 cm	1991 - 2000	20 - 26	30	35	100 x 100
ERS-2	5.6 cm	1995 -	20 - 26	30	35	100 x 100
Envisat ASAR	5.6 cm	2002 -	15 - 45	30 - 150	35	100 x 100 to 400 x 400
Radarsat-1	5.6 cm	1995 -	20 - 59	8 - 100	24	50 x 50 to 500 x 500
Radarsat-2	5.6 cm	2007 -	20 - 59	10 - 15	24	20 x 20 to 500 x 500
ALOS PALSAR	23.6 cm	2006 -	8 - 60	10 - 100	46	30 x 30 to 350 x 350
TERRASAR-X	3.1 cm	2007 -	20 - 55	1 - 16	11	10 x 5 to 100 x 1500
COSMO-SkyMed	3.1 cm	2007-	TBC	1 - 100	1	10 x 10 to 200 x 2000

2.1 Conventional Differential Interferometry (DifSAR)

DifSAR maps relative surface deformation occurring in the temporal period spanned by two SAR images. DifSAR is capable of mapping centimetres to metres of motion. Figure 2 illustrates typical results from this type of analysis. The left-hand image shows DifSAR analysis of a pair of SAR images from the ALOS PALSAR satellite before and after the Sichuan earthquake of 12 May 2008. Each interference fringe on the image corresponds to a motion contour of 11.8cm. The right-hand image shows DifSAR analysis of a pair of SAR images from the Envisat ASAR satellite from 2004 and 2007 and shows settlement basins associated with water abstraction for oil and gas production activities in Long Beach, California. Each interference fringe corresponds to a motion contour of 2.8cm. The key limitations associated with this technique are atmospheric artefacts and the ability to maintain coherence between successive images.

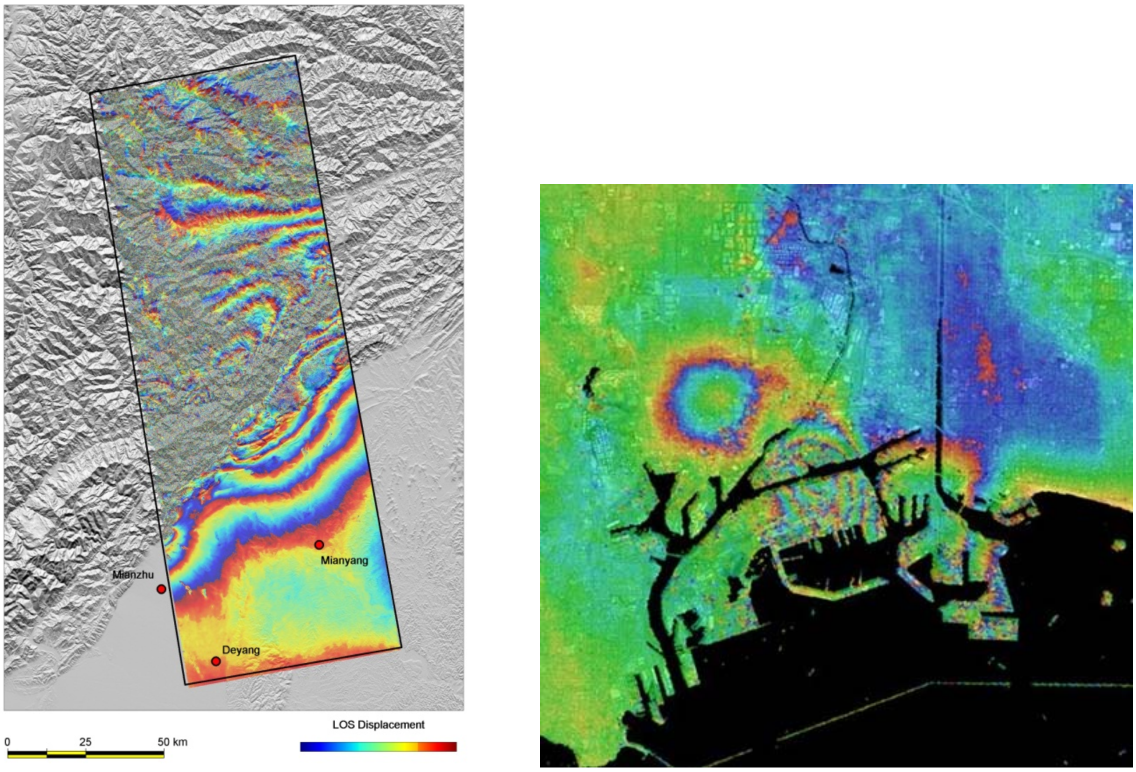

Figure 2: DifSAR analysis of ALOS PALSAR data showing effects of 2008 Sichuan earthquake (left-hand image) and DifSAR analysis of Envisat ASAR data showing settlement related to water abstraction associated with oil and gas production activities in Long Beach, California, between 2004 and 2007 (both images courtesy Fugro NPA Ltd)

2.2 Persistent Scatterer Interferometry (PSI)

PSI is an advanced differential interferometric technique, involving the processing of a collection of SAR images (typically 20 or more) acquired at different times, in order to identify networks of persistently reflecting ground features (such as buildings, bridges, infrastructure and rocky outcrops) that are present in every image. These can be used to improve coherence, and thus allow more precise measurements of motion to be made - PSI is capable of mapping millimetre-level motion trends. PSI relies on the presence of naturally occurring radar reflectors, meaning it is usually best suited to urban and semi-urban areas, and it also requires a suitable archive of SAR data to be available. In typical urban areas, there can be hundreds of measurement points (Persistent Scatterers) per square kilometre, as shown in Figure 3. PSI has been used in the UK for monitoring of settlement due to tunnelling associated with the Jubilee Line Extension project in London (Millis et al, 2008), and it is understood that the upcoming Crossrail project in London also includes PSI studies as a complimentary tool for the mapping of ground and structure motions above the tunneled sections.

Figure 3: Location of persistent scatterers on Hong Kong Island (image courtesy Fugro NPA Ltd)

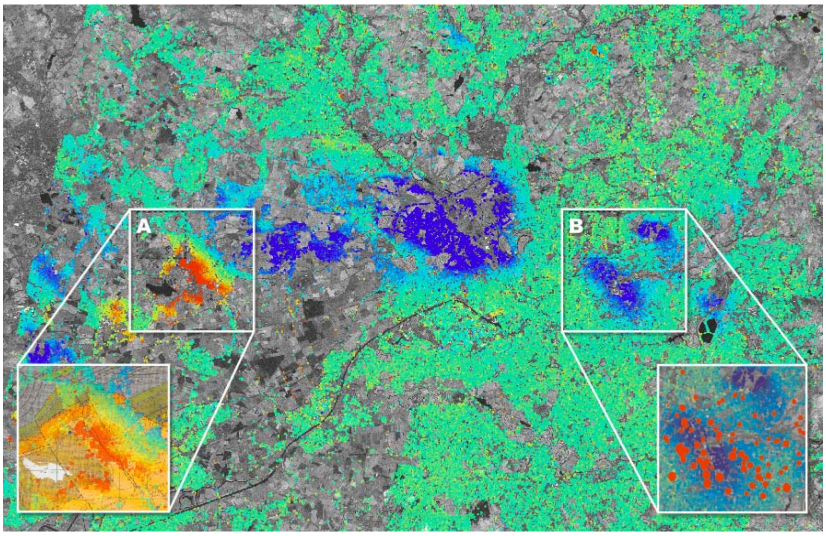

Figure 4: PSI motion data for Manchester, UK, for the period 1992 to 2002 (image courtesy Fugro NPA Ltd)

Figure 4 shows PSI motion data for Manchester, UK, for the period 1992 to 2002, styled according to average annual velocity (mm/yr). The image reveals localised zones of surface subsidence and uplift. Correlation with geological information shows that the subsidence is correlated with fault structures (inset A on the image) and the uplift with seismically active areas (inset B on the image).

2.3 Artificial reflector interferometry (CRInSAR and CATInSAR)

Although PSI allows smaller movements to be detected than conventional DifSAR, it is not usually possible to find suitable scatterers at the exact locations to be monitored, particularly in areas of low natural radar response, such as rural settings. This problem can be overcome by installing small networks of artificial reflectors at specific points of interest such as dams, embankments, underground water reservoirs and landslides. The artificial reflectors generate a strong response in the SAR image and enable any movement at the points of interest to be monitored. The most common type of artificial reflectors are passive corner reflectors (CRs), although compact active transponders (CATs) are now gradually coming into use. CRs are aluminium structures of the order of 0.5m to 1m square (depending on the radar wavelength) that strongly reflect the radar signal transmitted by the satellite, in a similar manner to the natural scatterers. CATs are active units that detect the incoming radar signal and transmit an amplified version of the signal back up to the satellite. CATs have the advantages that they are much smaller than the CRs (about the size of a shoebox), and that they can return a signal on both the ascending and descending passes of the satellite, potentially doubling the number of images that can be processed. Figure 5 shows a typical CR and CAT, and the strong responses created in a SAR image.

Figure 5: (clockwise from bottom left): Corner reflector (CR); Compact active transponder (CAT); CAT responses in SAR image (all images courtesy Fugro NPA Ltd)

When considering whether Satellite InSAR techniques are appropriate for a particular situation, it is necessary to consider several issues on a case-by-case basis, including:

- *Motion direction:* Satellite InSAR detects motion in the ‘line of sight’ of the satellite (typically near vertical and looking perpendicular to the near polar orbit of the satellite).
- *Topography:* Significant variations in topography can cause distortions in SAR imagery because the satellite does not look straight down. Steep slopes may be poorly imaged or obscured completely.
- *Ground cover and land use:* Satellite InSAR only works under ‘coherent' conditions (where radar reflections are correlated between SAR images). Land cover change (e.g. vegetation) is the main cause of temporal decorrelation.
- *Motion magnitude:* In order to successfully sample motion, two co-dependent factors require consideration, the radar wavelength and revisit time of the satellite. Different radar wavelengths are sensitive to different magnitudes of motion; larger radar wavelengths will better sample large motions (metres), and smaller wavelengths will better sample subtle motions (centimetres). Appropriate temporal sampling (satellite revisit time) must be chosen to successfully sample the signal of interest.
- *Spatial extent of motion:* If the spatial extent of the motion is smaller than the spatial resolution of the SAR images then it cannot be measured.

Within Hong Kong, Satellite InSAR presents a unique remote sensing tool that can contribute to motion hazard assessments in natural terrain. Some DifSAR analyses of areas prone to settlement have been carried out in the past in Hong Kong by various academic institutions (Liu et al. 2001) with reasonable success. The GEO has also previously investigated the use of DifSAR specifically for detection of slope movement (Wong 2007) and although some constraints were noted related to the low spatial resolution and temporal decorrelation of the images used, it was considered that the quality and availability of SAR images would improve in the future as more satellites are launched.

Wong (2007) mentions that the GEO have installed a total of nine CRs at three trial sites in Hong Kong. Millis et al. (2008) discusses the feasibility of the use of PSI for monitoring the deformations associated with tunnelling work, in particular the Hong Kong West Drainage Tunnel currently under construction. A PSI study was included in the specification of this project (Drainage Services Department 2007).

Although the topography of Hong Kong presents challenges, a reasonable archive of historical SAR images is available from ESA satellites and the expanding family of SAR satellites. Satellite availability and processing techniques continue to improve, and when the availability of newer and more advanced techniques such as PSI and artificial reflectors (especially CATs) is taken into account, it is considered that InSAR presents many opportunities for monitoring of both natural and artificial terrain in Hong Kong into the future.

3 MOVEMENT MONITORING FROM THE AIR

Airborne Light Detection and Ranging (LiDAR) is a remote sensing tool for mapping surface topography conducted from both fixed-wing aircraft and helicopters. A LiDAR system measures the distance between the aircraft and the ground by measuring the time that it takes for a laser pulse sent from the aircraft travel to the ground and its reflection to be detected by a sensor on the aircraft. By sending out and detecting multiple laser pulses, a three-dimensional "point cloud" corresponding to a model of the elevation of the ground surface beneath the aircraft's path may be built up. Analysing the difference between elevation models of the same area taken at different times allows an estimate of movement to be made.

A LiDAR system comprises a scanning laser, a navigation system and an inertial measurement unit (IMU). The laser system transmits light pulses at a rate of up to 250,000 Hz. The laser pulses are typically transmitted in a 20 to 30 degree swath perpendicular to the direction of flight. Depending on flying height and speed, point densities range from 1 to 100 points per square meter.

As the speed of light and the time taken for a pulse to make a round trip from the laser back to the aircraft are known, the distance between the ground and the aircraft may be calculated. Knowing the position of the aircraft allows the absolute coordinates of the laser reflections on the ground surface to be established. The position and flight path of the aircraft are determined using differential GPS in combination with the IMU system, which measures the accelerations and attitude of the aircraft.

As well as measuring the time for the initial pulse to be reflected, most LiDAR systems allow multiple returns to be measured, as the laser pulse is reflected from vegetation and building structures before it reaches the ground. By analysing the last returns to be received from each pulse, the true ground surface can be mapped, rather than the surface of the vegetation canopy (Figure 6). This technique is of great interest for natural terrain monitoring in Hong Kong, as it allows clear definition of ground features that would otherwise be hidden by a thick vegetation cover, and not visible using conventional aerial photographic techniques.

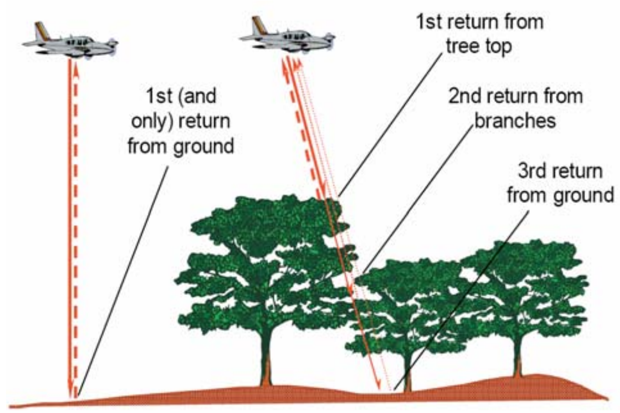

Figure 6: Multiple returns allow "virtual deforestation"

The accuracy of the surface coordinates measured with a LiDAR system consists of two parts: relative and absolute accuracy.

The relative accuracy is mainly influenced by imperfections in the hardware components of the laser scanner and navigation system. These imperfections, such as angular jitter and timing errors are of a random nature and do not cause a bias in the LiDAR point cloud, but result in "noise" in the measured results. These random imperfections are usually at cm level, but because so many points are measured per second, the noise may easily be filtered out, particularly if one is detecting planes or linear features in the terrain.

The absolute accuracy is mainly affected by the GPS processing techniques used for the positioning of the LiDAR system. By adopting various improvement techniques such as using twin GPS systems and keeping the distance between the airborne GPS systems and the base station for differential GPS correction within 10-15 km, the position of each coordinate in the point cloud can be determined with an accuracy of several centimetres. As with terrestrial differential GPS measurements, the vertical (z) coordinate is generally less accurate than the plan (x, y) coordinates because of the 20,000km altitude of the GPS satellites, and fluctuations in vertical accuracy can be observed of the order of 5cm.

The vertical accuracy may be improved by the use of known control points on the ground to model these fluctuations. Control points should normally be placed every 2km to 4km along the flight path, and should have a known elevation, based on a higher accuracy terrestrial survey.

Typical control points used are LiDAR marker boards (LMBs) coated in a reflective material, such as road marking paint, so as to guarantee maximum reflection of the laser pulses (Figure 7). LMBs are typically sized from 60cm to 100cm, depending on the anticipated laser point density.

As well as improving the vertical accuracy achievable, control points such as LMBs may be used to register common observation points in different LiDAR surveys to permit accurate assessment of any changes in elevation that may have occurred between the two surveys.

Figure 7: LiDAR marker board

Fugro's in-house LiDAR system, FLI-MAP, is a modular helicopter-mounted system that attaches to the skids of a suitable helicopter without any airframe modifications being necessary (Figure 8). Twin GPS antennas are mounted on booms to give accurate positioning of the aircraft. Typical overall absolute survey accuracies achievable with the FLI-MAP system are 5cm, improving to 2cm (1σ) with the use of LMBs as control points.

Figure 8: FLI-MAP LiDAR system mounted below a helicopter (photo courtesy of Fugro Aerial Mapping BV)

Typical applications for monitoring deformation or changes in volume using airborne LiDAR include open pit mines and active waste disposal sites (Figure 9, left hand image). Authorities responsible for water management in the Netherlands have used Fugro's FLI-MAP system to annually survey dike systems. Certain dikes show displacements of more than 50 cm over large stretches. These deformations would be unnoticeable when manually inspecting the dikes in the field. LiDAR may also be used to monitor cm-level deformations of structures such as bridges and viaducts, jetties (Figure 9, right hand image).

Because of the density of the LiDAR data taken from helicopter mounted systems, the point noise can be filtered out and very high relative accuracies of the order of 5mm can be achieved. This is very useful in cases where relative accuracy is of greater importance than absolute accuracy, such as when measuring small deformations in applications such as railway track alignment and highway rutting.

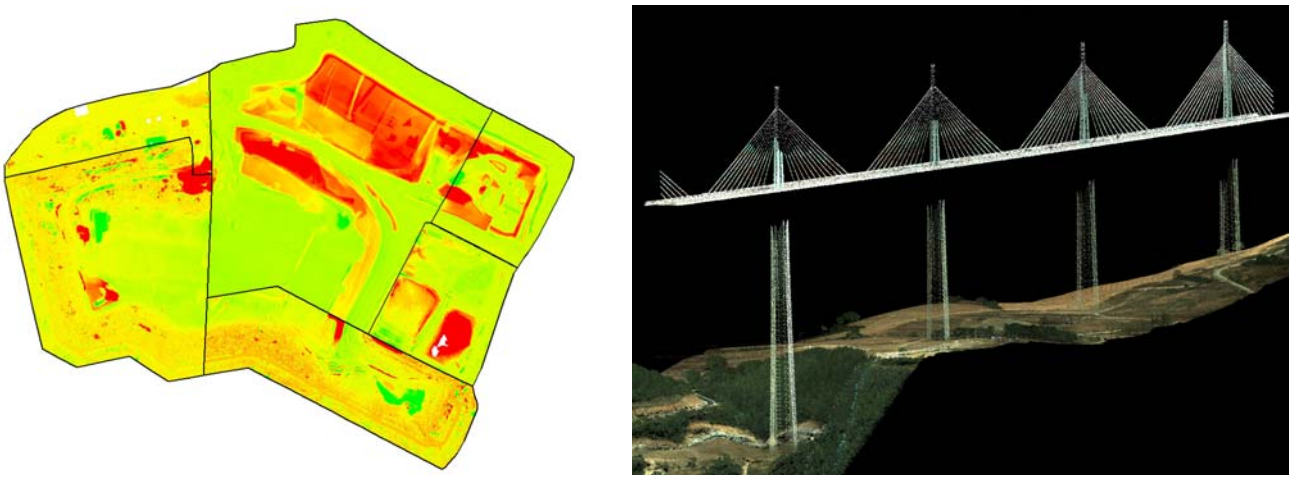

Figure 9: Map showing deformations on a waste disposal site (left hand image) ; LiDAR point cloud showing the Millau bridge in the South of France (both images courtesy Fugro Aerial Mapping BV)

In December 2006, the GEO conducted a pilot airborne LiDAR survey of Hong Kong Island (Ng & Chiu, 2008) for the purposes of evaluating the "virtual deforestation" capability on heavily vegetated hillsides for assessment of natural terrain hazards. The survey was carried out using a fixed-wing aircraft, and had technical requirements of a sampling interval of 1.3m and accuracies of 0.3m (horizontal) and 0.13m (vertical). The survey took eight hours of flying time to cover an area of approximately 80km², and approximately 768 million data points were collected. The survey confirmed the potential of the use of the technique in Hong Kong for identifying ground features such as relict landslides and terrain morphology that are disguised under vegetation cover, and it is understood that a further survey is currently being planned.

4 MOVEMENT MONITORING FROM A REMOTE GROUND LOCATION

4.1 Total station systems

Total stations are the most common instrument used to monitor deformation of unstable areas or constructions, and they are a familiar sight in Hong Kong. A total station is an instrument that measures the distance, vertical angle and horizontal angle to a target. These observations are used to calculate the position of the target relative to the total station. If reference targets are installed at known locations, or the location of the total station is accurately known, the absolute position of the target may be ascertained. Targets used are normally prisms, although reflectorless total stations also exist. Typical accuracies for total station measurements range from ±1mm to ±1cm over ranges up to 2,500m. Accuracy depends on the distance to the target. For typical ranges of around 25m, accuracies of ±2mm are usually achievable.

Where repeated measurements of multiple targets are required, automated or robotic total stations are an efficient choice, and allow high quality data to be obtained without any human interaction. A robotic total station may be established in a stable area that overlooks the area of interest and will scan the targets at regular intervals, with readings being either stored locally or via a data link to a central processing centre. A single robotic total station may acquire readings from up to hundreds of targets, noting that it can take up to a minute to locate each target and take the reading, and that all targets must be within the line of sight of the total station.

Robotic total stations do not work well in poor weather conditions such as rain or fog, and as a total station is a rotating device with many moving parts, the provision of regular maintenance is also an issue that has to be taken into account. Total stations are also relatively expensive items, and unattended robotic total stations are vulnerable to theft and vandalism. For use in natural terrain applications, keeping the targets clear of vegetation is also an issue.

Robotic total station systems have commonly been used in the construction industry in Hong Kong for several years now, typically for railway-related projects related to tunnel and track deformation (Chan et al. 2003), and adoption for natural terrain monitoring is readily achievable.

4.2 *Terrestrial laser scanning*

Terrestrial laser scanning is a remote sensing technology that can rapidly acquire accurate three-dimensional spatial data. The primary engineering applications include architectural surveying, terrain surveys for volume analysis and measuring complex mechanical systems for modelling.

The principle of operation is essentially the same as that of the airborne LiDAR systems described above. The result of a terrestrial laser scanning measurement is an accurate 3D point cloud describing the surveyed object. The range of the applied laser scanners is currently 2-1000m, with typical accuracy of ±6mm achievable at a distance of 50m.

Laser scanning is rarely used in automatic monitoring systems, but primarily for periodic high accuracy surveys of a larger area within a relatively short time. A laser scanner can collect up to 500,000 points per second. No artificial targets or preparation of the monitored area is required, although as is the case with the airborne LiDAR, the inclusion of known control points in the scan can improve accuracy. Several scans from different station setups can also be combined to produce a single model.

Disadvantages with this technology are that laser scanning does not work in rain or fog and although the measurement distance is up to 1km the accuracy decreases with distance. Price for the system is also quite high. Wong (2007) notes that terrestrial laser scanners have been used for surveying natural terrain areas where physical access is difficult or dangerous, such as landslide sites, and potentially have many other novel geotechnical applications.

4.3 *Ground-based radar system*

Ground based interferometric radar measurements are becoming a useful tool for measuring movements with very good accuracy and a high sampling rate at large distances. The principle of operation is similar to the InSAR systems described earlier, with phase information in the radar signal reflected from a target being used to infer small changes in distance. The difference is that in this case, the radar transmitter is ground-based, rather than mounted on a satellite orbiting the earth.

Ground-based radar systems offer some significant advantages compared with optical systems such as total stations and terrestrial laser scanning:

- Long range remote monitoring	Capable of monitoring displacements at a measuring distance of up to several kilometres without any access needed to the area being monitored, although the installation of reflectors can improve accuracy.
- Simultaneous wide area monitoring	Areas of up to several square km can be monitored at the same time
- Sub-millimetre measurement accuracy	±0.1 mm in normal situations; up to ±.01mm is achievable in certain conditions
- Any time, all-weather operation	Capable of operating in all weather conditions: day, night, fog and rain.
- Automatic operation	Can operate on a long term basis without human intervention.
- High measurement frequency - dynamic measurement	Radar not only allows continuous monitoring of slow displacements and deformations but can also measure structure vibrations up to 100 Hz.

In Norway, Fugro have been involved in a major natural terrain monitoring project of Norway's largest potential landslide (Krangnes 2008). This project involves several of the land-based techniques described in this paper, but a key part of the instrumentation used is a ground-based radar system.

A major potential rockslide was discovered 10-15 years ago in one of Norway's most scenic and popular tourist areas. The area, which is located on Norway's west coast in a UNESCO World Heritage area, comprises a narrow fjord surrounded by steep mountains. The unstable zone covers an area of almost 0.8 km^2 with the main rift 900m above sea level. The unstable zone holds a possible volume of 30-100 million m^3 of rock, moving with a velocity of 3-10 cm per year. Should the unstable zone suddenly fail and plunge into the fjord, a tsunami wave up to 40m in height will be generated, which will wipe out the communities living by the side of the fjord.

Following extensive risk analysis work, a major monitoring and early-warning system was implemented to track the rockslide movement in real time and thus to allow early evacuation in the event of a failure. The monitoring system had to overcome many challenges due to the rugged and remote terrain and the severe weather conditions.

The radar system is located approximately 2.3 to 3.3 km from the landslide area (Figure 10), and measures sub-millimetre movement of six reflectors located within the potential slide area. The system also monitors a further three control reflectors located in a stable zone outside the slide area (Figure 11).

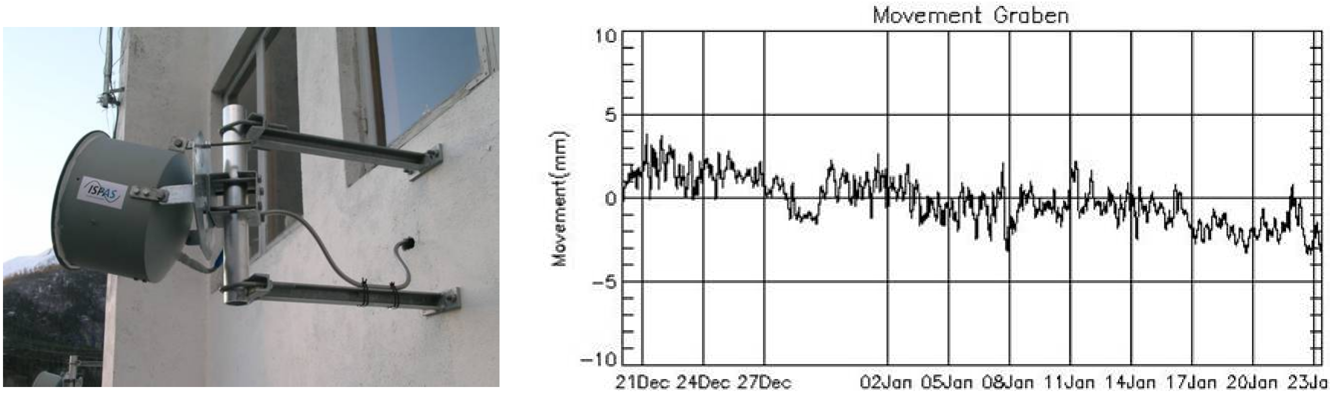

Figure 10: Radar system and plot of one month’s data from one of the reflectors

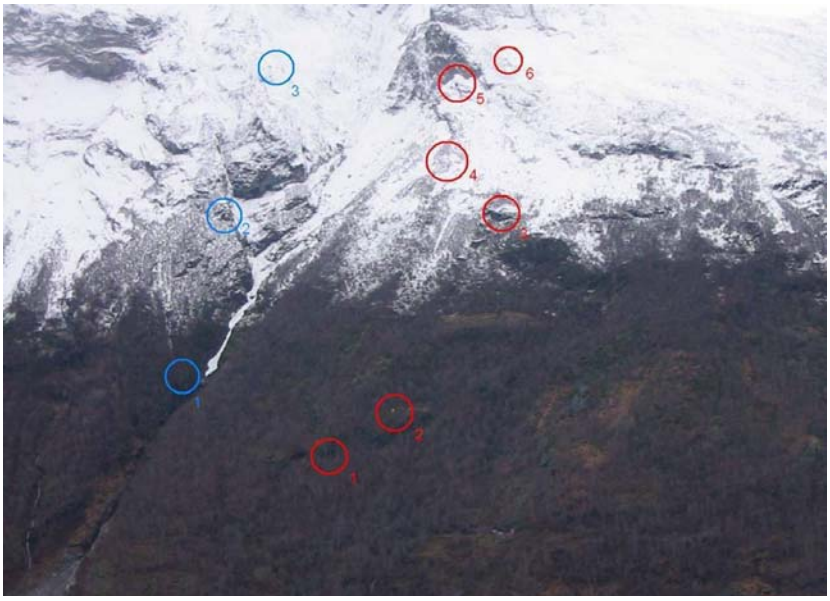

Figure 11: Location of reflectors on monitored area. Control reflectors are on the left of the image.

5 CONCLUSIONS

Remote sensing techniques for monitoring movement of natural terrain, whether from space, air or on the ground are commercially available, and have potential for use (or further use) in Hong Kong. The geotechnical community should continue to maintain awareness of such techniques, and it is to be hoped that we shall see them used more frequently in the future.

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