Dynamic Analyses of Slopes in Hong Kong

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Abstract

Hong Kong is mountainous and slope stability has been the subject of local research for decades. While the static performance of slopes in Hong Kong is relatively well understood, very little attention has been paid to the effect of dynamic load on soil slope stability. Slopes in Hong Kong subject to dynamic load, including earthquake load and blasting load are often considered using the conventional pseudo static force approach. This paper presents a more rational approach of using dynamic time history analyses to estimate the dynamic load induced displacement in slopes. A realistic earthquake time history input matching the seismic hazard having a 10% probability being exceeded in the next 50 years for Hong Kong has been developed. Also, a blasting time history record measured from a site formation project in Hong Kong has been used. Non-linear soil dynamic numerical analyses have been carried out using the finite difference program FLAC dynamic. The deformation of slopes in terms of displacement time histories and response spectra have been obtained as output from the analysis and compared to the site record.

Keywords: dynamic analysis, slopes, blasting, seismic

1. INTRODUCTION

Hong Kong is mountainous and slope stability has been the subject of local research for decades. While the static performance of slopes in Hong Kong is relatively well understood, little attention has been paid to the effect of dynamic load on soil slope stability. Slopes in Hong Kong subject to dynamic load, including earthquake load and blasting load are often considered using the conventional pseudo static force approach. However, the transient nature of dynamic motion and response may not be fully understood in such an approach.

This paper presents a simple and rational approach using dynamic time history analyses to estimate the dynamic load induced displacement in slopes. Non-linear soil dynamic numerical analyses have been carried out using a finite difference programme FLAC (for Fast Lagrangian Analysis of Continua) dynamic. The deformation of slopes in terms of displacement time histories has been obtained as output from the analyses. The FLAC dynamic program is widely used in international engineering practice.

1.1 Earthquake Loading in Hong Kong

The Hong Kong Special Administrative Region is located in an area of low to moderate seismicity. The Chinese Code for seismic design of buildings GB 50011 – 2001, categorises Hong Kong as being in Zone VII+ having a peak horizontal ground acceleration with a 10% probability of being exceeded in the next 50 years of 15% g. The GEO (Geotechnical Engineering Office of Hong Kong) Report No. 65 (1996) shows a 10% in 50 year value of about 10% g whereas Pappin et al. (2008) show a range of the 10% in 50 year peak horizontal acceleration varying between 7 and 15% g depending on the attenuation models and calculation methods being used.

Whilst seismic loading is considered in the design of bridges and other major infrastructure in Hong Kong, the current code of practice for building design does not require any seismic considerations. A few researchers have studied the seismic effect on slopes in Hong Kong which provided useful background information to this study. Pappin and Bowden (1998) studied a range of typical slopes in Hong Kong and quantified the probability of seismically induced down slope movement based on the pseudo static Newmark ‘sliding block method’ (described later in Section 2.2). The results show that, for a slope to be at a meaningful risk from seismic activity, it would need to have a static factor of safety (FoS) of less than about 1.1 for slopes affecting typical structures and less than about 1.2 for slopes affecting essential facilities. This is based on the conventional practice that for slope movements of less than 20mm, the risk of damage is low whereas a displacement of 100mm is associated with a high risk of failure in most soil slopes. It then follows on to conclude that current design methods in Hong Kong which ensure a FoS of 1.4 under the action of severe rainfall are sufficiently conservative to cater for the effects of seismic loading on slopes. GEO Report No. 98 presents a preliminary quantitative risk assessment (QRA) on the effects of earthquakes on slope stability and the likely consequences with respect to loss of life in the event of slope failures. Based on pseudo static methods the report concludes that the risk of earthquake-induced landslides is much smaller than the risk of rain-induced landslides for pre-1978 man-made slopes that have not been upgraded to current standards.

It follows that the current geotechnical standards appear to be adequate in maintaining the overall risk of earthquake-induced failures to new or upgraded slopes to a relatively low level. However it would be comforting have this conclusion confirmed by a more rational dynamic analysis method.

1.2 Blasting in Hong Kong

Blasting for the excavation of rock is frequently employed in construction in Hong Kong, especially in tunnelling and site formation works. The blasting works often have to be carried out in close proximity to existing slopes. GEO Report No. 15 presents the formulation of an analytical method for routine assessment of the stability of soil slopes subjected to blasting vibration using the pseudo-static approach. The method incorporates the dynamic response of soil slopes by a one dimensional multi-degree-freedom model which takes into consideration higher vibration modes of the slope. The dynamic response assumes a simple harmonic blast vibration input at the bedrock and the magnification factor of the response acceleration is calculated. The critical peak particle velocity is derived from the pseudo static acceleration required to give rise to a FoS of the slope of one, by assuming 30 Hz as the natural blasting frequency. However, this simple method does not allow the variation of the ground motion across the slope as it models the entire slope moving in unison. Given the very short duration of the blasting loading this method is expected to be overly conservative as stated in the report in that it ensures that there is no possibility of even very small permanent deformation arising from the blasting. Also, the dynamic deformation of the slope cannot be estimated by this simple method.

2 DYNAMIC SLOPE STABILITY ANALYSIS

The development of dynamic slope stability analysis and a quick overview on the relative merit of the various methodologies is presented in this section.

2.1 Earthquake Loading

The simplest method of dynamic slope stability analysis is to compute the FoS using a pseudo-static approach. Pseudo-static analysis simply extends the static analysis and determines the FoS of slopes subjected to a static horizontal acceleration. The main advantages of this approach are that it makes use of the well-established limit equilibrium method and it is easy to use. However, the transient nature of earthquake motion may not be understood in this approach. When the FoS is less than unity, the available strength along the failure surface is less than the driving forces in the slope and some movement will occur. This does not necessarily mean the slope will fail, as the movement may occur for a very short period of time and the total displacement can be small. If the cumulative displacement is large enough, however, the slope will fail.

2.2 Newmark's Sliding Block Model

Newmark (1965) developed the 'sliding block method' to determine earthquake induced displacement. The method considers the equilibrium of a rigid block on an inclined plane subjected to horizontal accelerations. Initially the critical acceleration (A_c), defined as the pseudo static acceleration required to reduce the FoS of the slope to one. When the earthquake acceleration imposed on the soil mass exceeds A_c, the net disturbing force on the soil mass will be larger than the net resisting force and permanent slope displacement will result. Newmark (1965) presented results showing that insignificant displacement occurs unless the A_c/A_m ratio is less than about 0.5, where A_m is the peak earthquake ground acceleration. Sarma (1975) extended the work by incorporating the dominant period of the earthquake motion (T) and Ambraseys & Menu (1988) extended it further by conducting statistical assessments of the displacement of slopes based on 48 near-field earthquake records.

2.3 Numerical Modelling of Earthquake-induced Permanent Displacement

One of the major limitations of 'sliding block method' is the requirement to pre-define a sliding plane which generally involves a search for a critical failure surface using limit equilibrium analyses. However, with the use of numerical finite element analysis, the displacement can be computed based on the distortion in each element, according to a non-linear stress-strain relationship. In this study, the finite difference package FLAC dynamic has been adopted.

2.4 Hong Kong Blasting Assessment Methodology (GEO Report No. 15)

In this document GEO specifies an analytical method for routine assessment of the stability of soil slopes subjected to blasting vibration using the pseudo-static approach. The proposed method takes the dynamic responses of soil slopes by a one dimensional multi-degree-freedom model which takes into consideration the higher vibration modes of the slope. By determining the critical acceleration from the slope stability analysis and estimating the corresponding magnification factor, the Critical Peak Particle

Velocity (PPV_c) can be calculated. The PPV_c calculated is used as the allowable limit of blasting induced vibration that can be imposed on soil slopes in Hong Kong.

According to GEO Report 15, the PPV_c of soil slope subjected to blasting vibrations with horizontal bedrock can be estimated by the following equation:

$$PPV_c = |K_c\, g / \omega\, K_a|$$

where K_c is the pseudo static critical acceleration of soil derived from static stability analysis;
g is gravitational acceleration;
ω is circular frequency of the ground motion (2π * frequency in Hz);
K_a is magnification factor, see Fig. 1;
S is shear wave velocity of soil, usually assumed to be 300m/s.

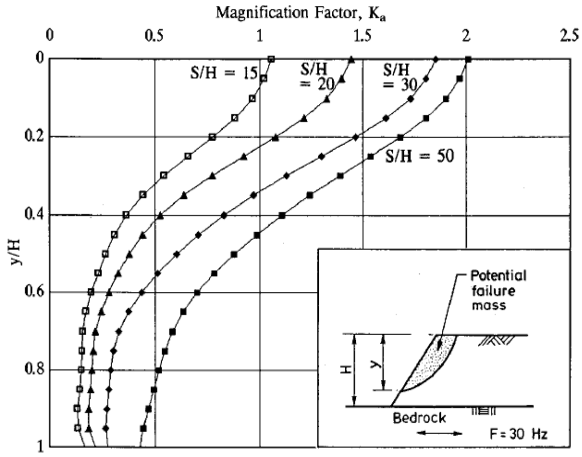

Fig. 1 K_a vs y/H for slopes with horizontal bedrock

3 ANALYSIS METHODOLOGY OF THIS STUDY

FLAC uses the explicit finite difference method to solve the full equation of motion using lumped grid point masses derived from the surrounding finite elements. It uses an updated Lagrangian procedure for coping with large deformations. FLAC has been extensively used to model dams and slopes in dynamic analyses, e.g. Kong (2003), Chugh & Stark (2005), Cetin & Isik (2005) and Marcuson et al. (2007). The application of dynamic FLAC for seismic analysis had been validated with other well accepted one dimensional non-linear soil response analysis programs in the United States, such as program SHAKE (Itasca Consulting Group, 1993). It is a simple and direct method to consider the non-linearity of soil and earthquake time history analysis. For this study, two dimensional analyses have been carried out using FLAC version 6.0 with the dynamic option.

3.1 Slope Geometries and Material Properties

A typical cut slope which is 10m high and has a slope angle of 45° has been considered. The slope is formed in a soil produced from granite which has been subjected to intense tropical weathering. This soil is known as completely decomposed granite (CDG) and in the model is underlain by moderately decomposed granite (MDG) which is assumed to be rockhead (Fig. 2). The slope is assumed to be dry. The material properties are determined from the range of static and dynamic parameters recommended in GEO Geoguide 1 (1993) which are listed in Tables 1 and 2 and are considered to be typical parameters in Hong Kong.

Arup, have previously carried out laboratory cyclic tri-axial tests on CDG in association with the Hong Kong University of Science and Technology. The small strain shear modulus (G_0) of the materials was determined from bender element tests, while the shear modulus degradation (G/G_0) and damping were assessed. The details of the tests can be found in Leung et al. (2010) and the results have been used to model the shear modulus degradation of the CDG.

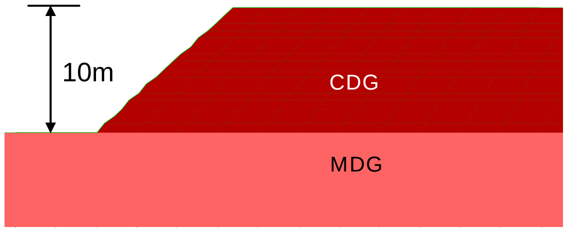

Fig. 2 The 45^{o} cut slope with horizontal bedrock base model

Table 1. Material properties of CDG

Soil Model	Mohr-Coulomb
Density ρ	1900 kg/m^3
Poisson's Ratio ν	0.3
Shear Wave Velocity Vs	300 m/s
Initial Shear Modulus $G_0 = \rho\ Vs^2$	171 MPa

Table 2. Material properties of MDG

Rock Model	Elastic
Density ρ	2400 kg/m^3
Poisson's Ratio ν	0.25
Shear Wave Velocity Vs	1000 m/s
Shear Modulus $G = \rho\ Vs^2$	2400 MPa

3.2 Static and Pseudo-static Analysis

The static and the pseudo static FoS has been determined using the limit equilibrium method. Two slope models with the same geometry but different Mohr-Coulomb strength parameters as listed in Table 3 have been considered. The program *Oasys* Slope was used to find the constant horizontal acceleration required to give a FoS of the slope of unity. This pseudo static acceleration is termed the critical acceleration, A_c of the slope (Table 3).

Table 3. Static FoS and critical acceleration of the two slope models

Soil strength	Static FoS	Critical Acceleration, A_c	Critical Blasting PPV_c (GEO Report No. 15)
CDG in Model 1 (c'=5 kPa, ϕ'=37^0)	1.2	0.11 g	20 mm/s
CDG in Model 2 (c'=7.5 kPa, ϕ'=39^0)	1.4	0.23 g	40 mm/s

3.3 Boundary Conditions and Initial Conditions

The boundary conditions of the slope model in the FLAC static and dynamic analyses are shown in Figure 3. The boundary at the base of the static model is fixed in both x- and y-direction and the vertical side boundaries are fixed in x-direction. For the dynamic model boundary conditions, free-field boundaries are coupled to simulate an infinite domain and to ensure that dynamic waves are not reflected at the side boundaries.

Gravity acceleration is applied to the model and run until an equilibrium state is obtained. The equilibrium state of the geometry can be assured from the horizontal and vertical displacement histories and when the unbalanced force diminishes. Once static equilibrium is reached, the displacements are redefined as zero. Earthquake or Blasting records are input along the base of the model in terms of a velocity time history. The time histories of the displacement, velocity, and acceleration at various locations of the slope face have been predicted. Soil damping is critical and the damping implied by the non-linear hysteretic soil model of the CDG is used in this study.

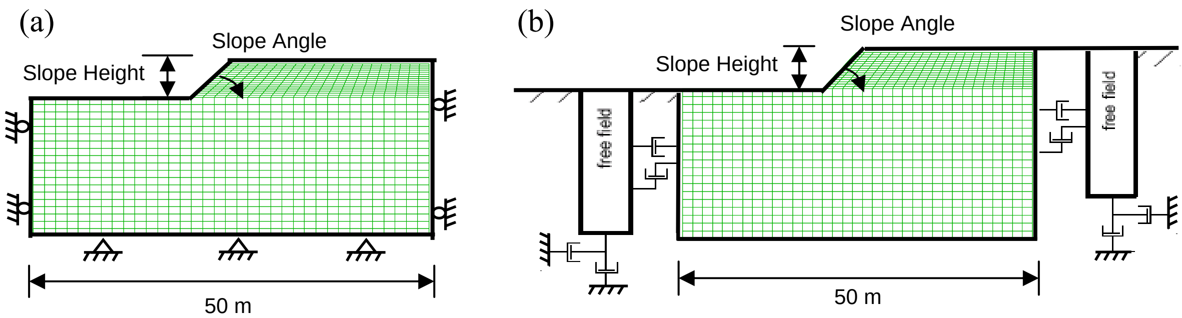

Fig. 3 Boundary conditions for the slope model: (a) static; (b) dynamic

4 INPUT DYNAMIC LOADING

4.1 Earthquake Motion

A probabilistic seismic hazard assessment study has recently been undertaken by Arup to estimate the potential seismic ground motion levels on bedrock in Hong Kong (Pappin et al., 2008). The calculated rock outcrop ground motion having a 10% probability of being exceeded in the next 50 years is presented in Figure 4 in terms of a velocity response spectrum. The peak acceleration and peak velocity of this earthquake ground motion are 12% g and 32 mm/s respectively. The input earthquake velocity time history is shown in Figure 5.

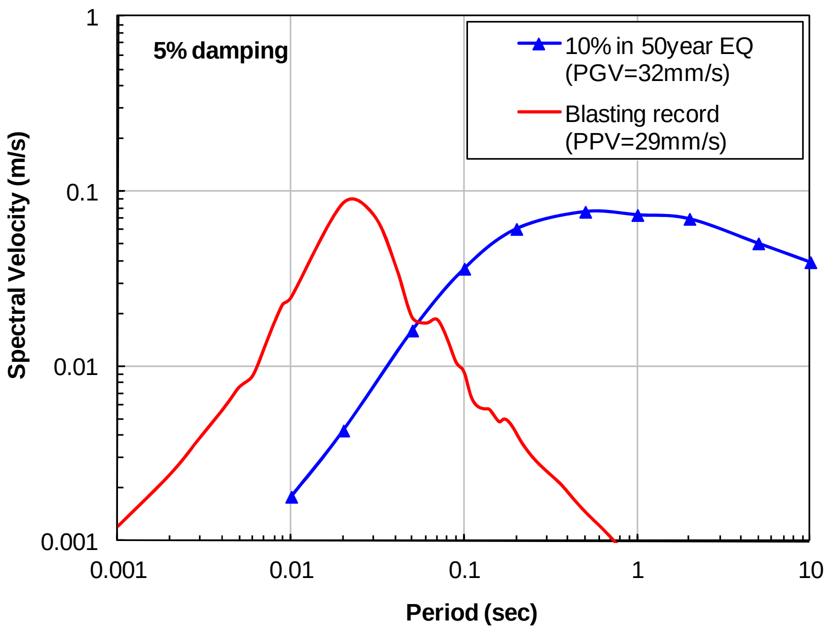

Fig. 4 Earthquake and blasting velocity response spectra on rock sites

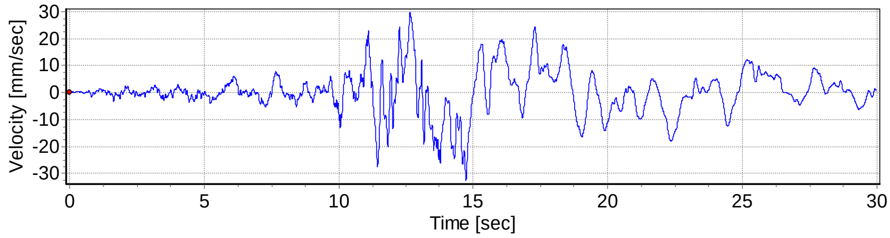

Fig. 5 Earthquake velocity time history

4.2 Blasting Motion

The blasting record used in this study was obtained from a detailed blast monitoring at a site formation development in Hong Kong in 2009. The blasting vibrations were recorded at the location of the geophones in three orthogonal directions. Figure 6 shows velocity response spectra of the site measurements on rock with peak particle velocity (PPV) between 23 mm/s to 101 mm/s. By normalising to a PPV value to 1 m/s a design normalised response spectral shape can be obtained as shown in Figure 7. A recorded event with a PPV of 29mm/s was chosen to match the peak ground velocity of the 10% in 50 year earthquake event for comparison purposes. The velocity time history and response spectrum of the blasting record from a rock outcrop are shown in Figure 8 and Figure 4, respectively. It is worthy to note that the peak acceleration of the blasting record is about 65% g. The time duration of a blasting event is normally about one or two seconds. Figure 7 shows that the frequency content of the different blasting records are quite consistent with a similar shape and a peak response at about 0.03 seconds which is consistent with the 30Hz frequency assumed in GEO Report No. 15.

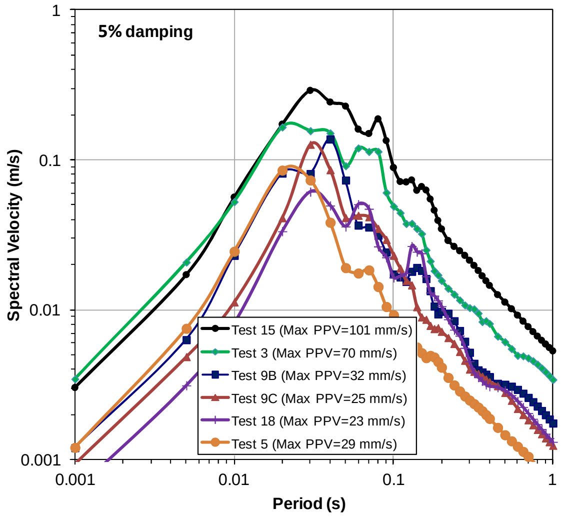

Fig. 6 Velocity response spectra of the site measured blasting records on rock outcrop

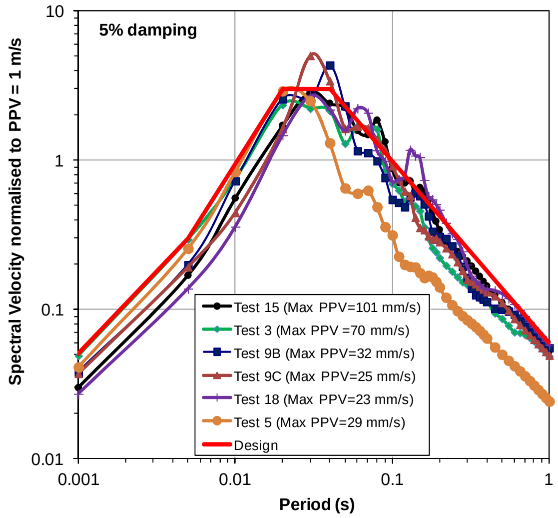

Fig. 7 Velocity response spectra normalised to a PPV of 1 m/s

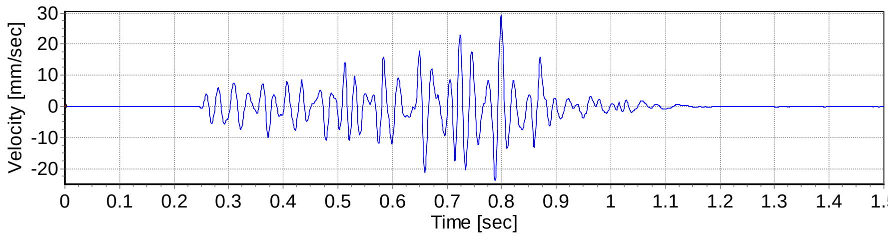

Fig. 8 Blasting velocity time history

5 FLAC ANALYSES

The results are presented in Table 4 and examples of the output displacement contour plots are presented in Figures 9 and 10. It was found that earthquake induced movements are generally shallow in the examples of this study. For a slope with a static FoS of 1.4, it was found that the maximum resultant displacements at slope crest subject to earthquake load of 475 years is 10mm. However, the maximum displacement increases to 60mm when the static FoS reduces to 1.2 (Table 4). When the slope is subjected to the blasting load, the movement is shallower (Figure 10) and the maximum resultant displacement at the slope crest is 8mm for the slope having a static FoS of 1.2 and the induced displacement for the slope having a static FoS of 1.4 is negligible being less than 1mm. A slope having a displacement in excess of 70 mm is usually considered to be at significant risk of failure. ASCE/SCEC (2002) recommends that a 50mm threshold displacement be used for typical slope construction.

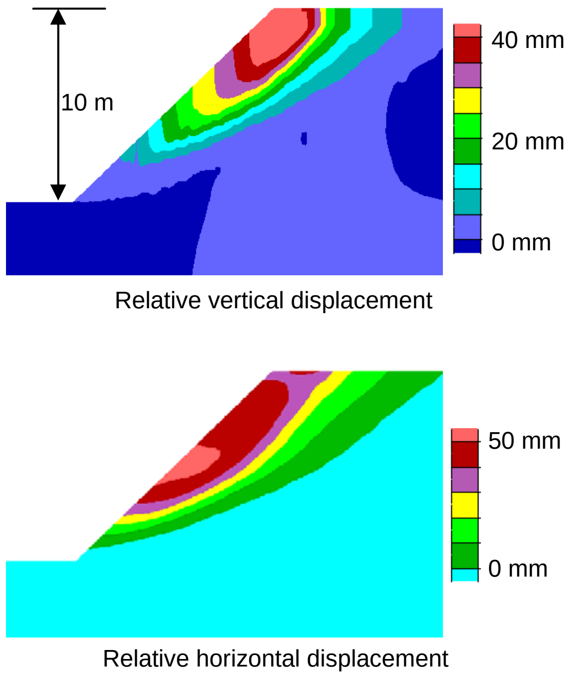

Fig. 9 Static FoS 1.2 subject to earthquake motion

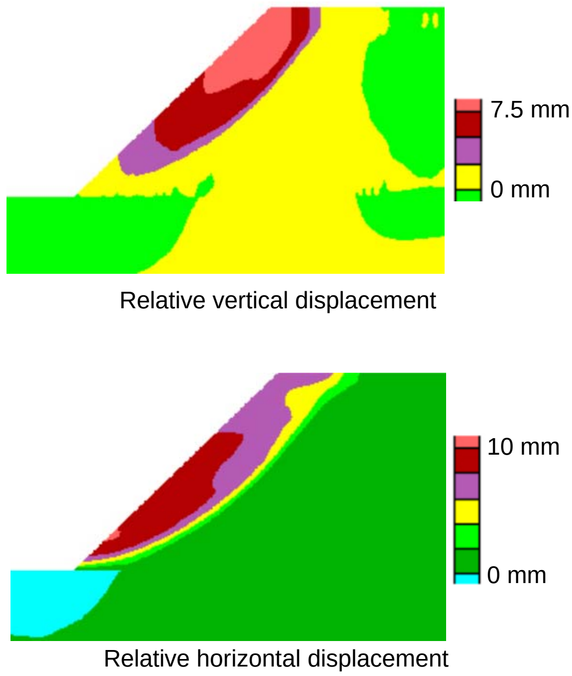

Fig. 10 Static FoS 1.2 subject to blasting motion

Table 4. Maximum relative resultant displacement at the slope crest

Dynamic Load	PGA/ PPA (g)	PGV/ PPV (mm/s)	FoS=1.2 (mm)	FoS=1.4 (mm)
Earthquake	0.12	32	60	10
Blasting	0.65	29	8	0.6

The results indicate that with similar peak ground / particle velocity, the blasting induced slope displacement is significantly less than that from earthquake loading. This is thought to be due to the significant differences in the frequency content of the two vibration types. Figure 4 shows that the blasting spectrum has a much higher frequency content than the earthquake spectrum with similar peak spectral velocity. Therefore, in addition to PPV, the frequency content of the loading is also critical to the determination of the dynamic load induced displacement. An additional factor is the length of time that the slopes are subject to the vibration load. Comparing the time histories shown in Figures 5 and 8 indicates that the earthquake motion duration is at least 10 times that of the blasting.

The FLAC results imply that conventional pseudo static rigid block analysis may not be a good predictor of slope displacement as the soil slope model is not behaving as a rigid structure over a well defined failure surface. The slopes behave in a more diffuse manner, more similar to a stack of cards than a solid block.

6 SITE MEASUREMENT AND NUMERICAL ANALYSIS OF BLASTING

At an ongoing site formation project in Hong Kong a trial has been carried out to measure the response of a 4.5 m high slope cut into CDG with an angle of 70°, located approximately 25 m from a blast. The slope was monitored with geophones at the toe and the crest, and an additional geophone was placed on a nearby rock outcrop. Figure 11 shows the set up of the monitoring locations and the inferred geological section from borehole data is shown in Figure 12. A back analysis using FLAC dynamic (Figure 13) has been carried out to compare the calculated slope crest response spectrum to that measured on site. The input blasting time history used in the dynamic analysis was based on the rock outcrop measurement (Location A in Figure 11) at a similar level to the blast rock surface and at a similar distance from the blast source as that of the slope location, since no direct measurement could be made at the rock surface under the slope. The input is that of Blast 15 shown in Figure 6.

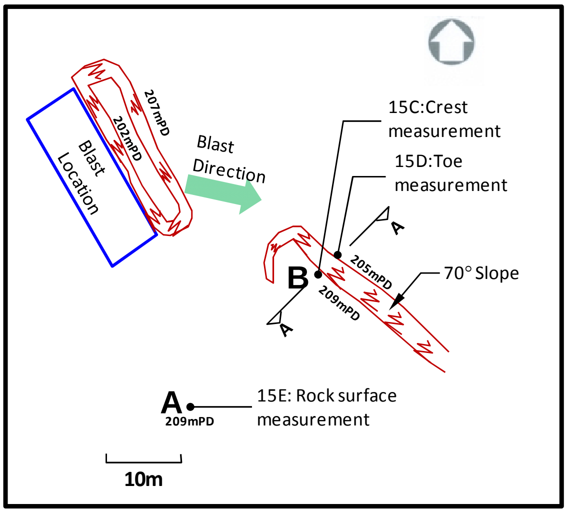

Fig. 11 Set up of the site measurement

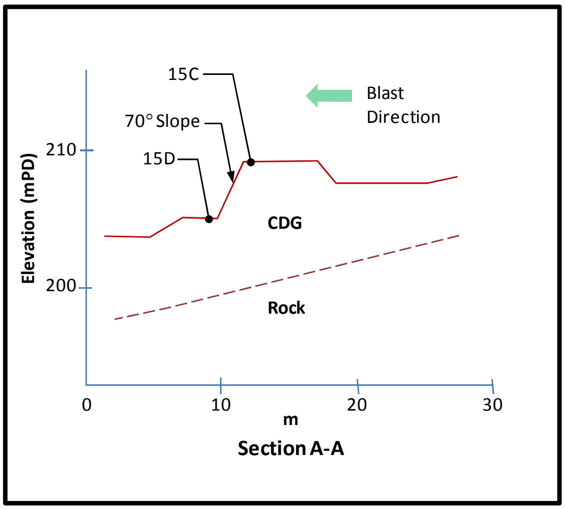

Fig. 12 Inferred geological section

The results are summarised in Table 5 and Figure 14 shows the comparison of response spectrum between the FLAC analysis and the site vibration measurement on site at the crest of the slope (Location B in Figure 11). The residual soil slope lateral displacement relative to rock calculated by FLAC is about 10mm. This is consistent with the site observation that some small centimetre sized fragments ravelled on the slope surface, but no slope failure was noted. However, it seems that the estimated critical PPV_c based on GEO Report No. 15, as listed in Table 5, is very conservative for determining the allowable PPV for slope blasting.

Table 5. Measured and predicted PPV's

Soil parameters	Site measurement		FLAC analysis		GEO Report No. 15
$A_c = 0.135$ g c'=7.5 kPa phi'=39° $V_s = 300$ m/s	Rock surface 101 mm/s	Slope crest 28 mm/s	Rock input 101 mm/s	Slope crest 35 mm/s	Critical PPV_c 9 mm/s

Figure 14 shows the comparison of the slope crest response spectrum between the FLAC analysis and the site measurement. They agree quite well in the low period range up to 0.1 seconds. It can be seen that there is a soil amplification of about 1.3 in the period range of 0.1 and 0.3 seconds and at larger periods the FLAC result shows that the slope crest and rock input spectra are very close to each other. However, the FLAC result for the slope crest shows significantly higher spectral velocity than the site measurement at periods above about 0.1 seconds.

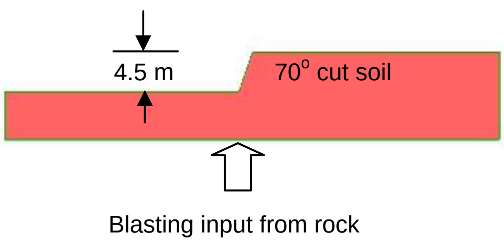

Fig. 13 FLAC simplified model

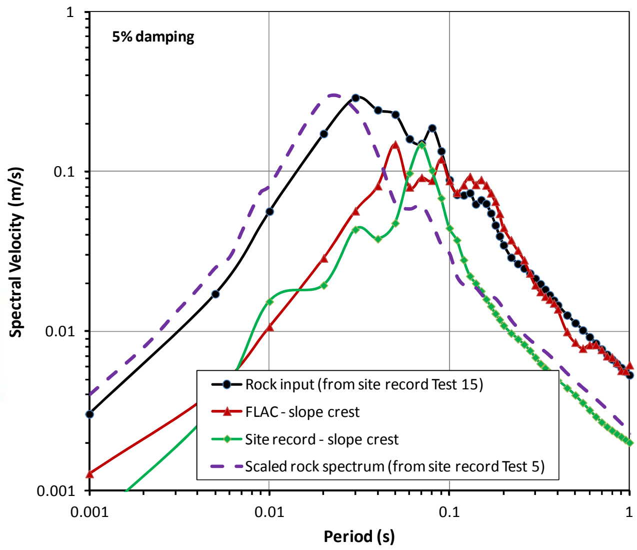

Fig. 14 Response spectrum comparison

This discrepancy at higher periods may be because the rock input motion to the FLAC model is too large at periods larger than 0.1 seconds. The FLAC input rock motion was recorded at Location A (see Figure 11) which is a different direction from the blast source, although the distance from the blast source is similar. Site observation also shows that the rock motion recorded to the side of the blast source (Location A) may excite higher vibrations at period range of 0.1 and 1 second than those at Location B in front of the blast due to trapping of the blast waves behind the rock slope. The rock motion recorded in Blast 5 (shown in Figure 6) was recorded at about 30m in front of the blast. If the rock response spectrum of Test 5 is scaled to match the recorded maximum spectral velocity of the rock input measured in Blast 15 (see Figure 14), it can be seen that the shape of the response spectrum between Test 5 and recorded slope crest match quite well at higher periods. Therefore, the frequency content of the rock input response spectrum may significantly dominate the soil slope response. Further studies could be considered to identify what effect source orientation has on the vibration response. Also, the geometry and soil parameters of the FLAC model cannot be exactly the same as the site conditions leading to other differences between the numerical analysis and the site measurement.

7 CONCLUSIONS

The dynamic performance of slopes subjected to earthquake and blasting ground motions have been investigated in this study. A simple and rational time history approach has been presented and numerical analysis using dynamic FLAC has been carried out to determine the dynamic load induced displacement in slopes. The advantage of using the time history analysis is that it allows the designer to assess the slope performance using realistic earthquake or blasting records in the analysis and the soil non linearity response can be considered. The output can be presented in terms of maximum displacement, displacement contours and time histories which are useful to assess the effect of dynamic loading to any foundations or earth-retaining structures located within or nearby the slopes.

Slopes with two different shear strength parameters have been considered in this study. It was found that the frequency content and duration of the dynamic load is critical to the resulting displacement in slopes. When subjected to a seismic ground motion having a 10% chance of being exceeded in the next 50 years, the maximum resultant displacement at the crest of a 10 m high slope having a static FoS of 1.2 is calculated to be 60 mm. For a blasting record obtained from recent blasting works in Hong Kong, having a PPV of 29 mm/s (similar to the PGV of the earthquake loading), the maximum resultant displacement of the same slope crest is 8 mm. For a slope having a static FoS of 1.4, the earthquake induced maximum displacement is 10 mm and the blasting induced displacement is negligible. A slope having a displacement in excess of 70 mm is usually considered to be at significant risk of failure. ASCE/SCEC (2002) recommends that a 50mm threshold displacement be used for typical slope construction.

Velocity response spectra of the site measurements at rock outcrop for PPV recorded from 23 mm/s to 101 mm/s have been presented and a normalised design response spectral shape was obtained. The comparison of the slope crest response spectrum between FLAC analysis and site measurement for a 4.5 m high cut slope agree quite well in the short structural period range less than 0.1 seconds. A more extensive sensitivity study considering blasting orientation and slope parametric study, such as pore water pressure, different slope geometry is proposed for the next stage of study.

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