

Chapter 2

Review of Studies on Earthquake-Induced Landslides

Abstract This chapter reviews two aspects of existing studies on earthquake-induced landslides: slope stability analysis and landslide movement simulation. The merits and demerits of each method are stated.

Keywords Review • Slope stability • Landslides • Earthquake-induced landslides

2.1 Introduction

A large number of landslides can be caused by a strong earthquake. Appendix A shows a general list of 99 historical earthquakes causing substantial landsliding in the worldwide since the twentieth century. From Appendix A, a huge number of landslides were induced by the Chi-Chi earthquake in Taiwan (1999), by the Mid-Niigata Prefecture earthquake in Japan (2004), by the Northern Pakistan earthquake (2005), and by the Wenchuan earthquake in China (2008). Most recently the Great East Japan earthquake with magnitude 9.0 also caused a lot of landslides.

Earthquake-induced landslides have been the source of significant damage and loss of people and property. Landslides caused the majority of damage or casualties in many earthquakes. One of the most extreme examples is from the 1970 Peru earthquake, which triggered a huge rock avalanche that buried two cities and killed almost half the 54,000 fatalities (Rodriguez et al. 1999). Another example is, in the 1920 Haiyuan earthquake, numerous large landslides caused widespread devastation to buildings and infrastructure and killed at least 100,000 people, almost half of the total earthquake deaths (Keefer 2000).

Therefore, it is very important to predict the earthquake-induced landslides and to take countermeasures for potential landslides.

The problem of seismic landslide hazards has received considerable attention worldwide in recent years. Some special international symposiums for earthquake-induced landslides have been hold on and some special journal issues or proceedings on this field have been published (Table 2.1).

Table 2.1 Some special issues on earthquake-induced landslides from international journals

No.	Journal/Press	Issue/data	Name
1	Engineering Geology	2000, vol. 58, issue. 3–4	Special issue from the symposium on landslide hazards in seismically active regions
2	Surveys in Geophysics	2002, vol. 23, issue. 6	Special issue of the symposia on assessment and mitigation of collateral seismic hazards
3	Engineering Geology	2006, vol. 86 issue. 2–3	Landslides induced by earthquake and volcanic activity
4	Engineering Geology	2011, vol. 122, issue. 1–2	The next generation of research on earthquake-induced landslides
5	Springer	2013. Feb.	Earthquake-induced landslides Proceedings of the International Symposium on Earthquake-induced landslides, Kiryu, Japan, 2012

Main topics of earthquake-induced landslides are the following:

1. Investigation of recent and historical earthquake-induced landslides and their impacts so as to produce inventories of historical earthquake-induced landslides.
2. Prediction of potential earthquake-induced landslides, including (i) failure mechanism and stability analysis of seismic slopes, (ii) movement mechanism and behaviours of earthquake-induced landslides, and (iii) Instrumentation and monitoring technologies for potential earthquake-induced landslides or post-earthquake landslides.
3. Preventive countermeasures for earthquake-induced landslides, including (i) Stabilization and disaster mitigation of earthquake-related landslides, (ii) risk assessment and management of earthquake-related landslides, and (iii) hazard map and early warning system for earthquake-related landslides.

This chapter focuses on the prediction of potential earthquake-induced landslides. The prediction of potential landslide can be carried out using detailed geotechnical investigations and stability calculations. (i) Failure mechanism and stability analysis of seismic slopes, i.e., seismic slope stability analysis and (ii) movement mechanism and behaviours of earthquake-induced landslides, i.e., landslide run-out analysis are outlined firstly, and then, the merits and demerits of each method are clarified in this chapter.

2.2 Seismic Slope Stability Analysis

Methods developed to date to assess the stability of slopes during earthquakes can be divided into three categories: (1) pseudo-static methods, (2) dynamic sliding block methods, and (3) stress-strain methods. Each of these types of methods has

strengths and weaknesses, and each can be appropriately applied in different situations (Jibson 2011).

The Sect. 2.2 reviews these three categories methods and discusses their advantages and limitations.

2.2.1 Pseudo-Static Methods

Terzaghi (1950) first presented the pseudo-static method, which is a simple method for evaluating of seismic stability of a slope. It can be applied to natural or man-made slopes based on either analytical method or numerical method. The earthquake force acting on the whole or an element of the slope is represented by a horizontal force and/or a vertical force equal to the product of the gravitation force and a coefficient k , called the pseudo-static seismic coefficient as shown in Fig. 2.1.

Thus, the assumed seismic acceleration a is k times the gravitational acceleration g , i.e., $a = kg$. On a potential sliding mass of weight W , the assumed pseudo-static forces will be

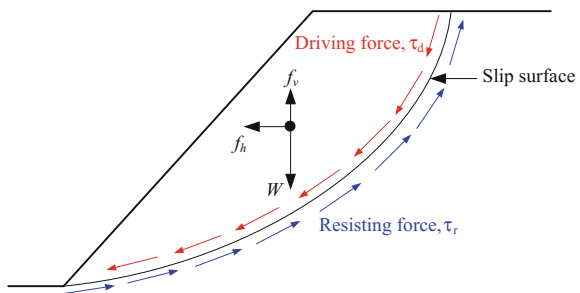
$$\begin{aligned} f_h &= \frac{a_h W}{g} = k_h W \\ f_v &= \frac{a_v W}{g} = k_v W \end{aligned} \quad (2.1)$$

where a_h and a_v are horizontal and vertical pseudo-static accelerations, respectively; k_h and k_v are horizontal and vertical pseudo-static coefficients, respectively. The FOS is expressed as the ratio of the resisting force to the driving force, Eq. 2.2.

$$\text{FOS} = \frac{\tau_r}{\tau_d} \quad (2.2)$$

There are three points for the pseudo-static seismic force mentioned: (1) **magnitude**, (2) **direction**, and (3) **acting point**, which should be determined in the application of the pseudo-static methods.

Fig. 2.1 Forces acting on a slope in pseudo-static slope stability analysis



2.2.1.1 Magnitude

From Eq. 2.1, the magnitude of the pseudo-static force is determined by the seismic coefficient. The key problem for the pseudo-static procedure is how to select an appropriate seismic coefficient under an acceptable FOS. There have been studies for determining the most appropriate pseudo-static coefficient by a matter of experience and judgment.

Terzaghi (1950)'s classical paper, probably the earliest recommendations on the values of the seismic coefficient published by a renowned geotechnical engineering or engineering geologist, made the original suggestion to use of $k_h = 0.1$ for **severe** earthquakes, $k_h = 0.2$ for **violent** and/or **destructive** earthquakes, and of $k_h = 0.5$ for **catastrophic** earthquakes.

Makdisi and Seed (1977) recommended a minimum pseudo-static factor of safety of 1.5 based on a soil strength reduction factor of 0.8 and the following values of acceleration associated with two different values of earthquake magnitude M . The same values of seismic coefficients for magnitude 6.5 and 8.25 earthquakes are recommended by Seed (1979), but with an acceptable FOS of 1.15.

$$\begin{aligned} a &= 0.1g \text{ for } M = 6.5 \text{ (implying } k = 0.1) \\ a &= 0.15g \text{ for } M = 8.25 \text{ (implying } k = 0.15) \end{aligned} \quad (2.3)$$

Seed (1979) gives a summary of the horizontal seismic coefficients and minimum factor of safety value for 14 large dams worldwide in the report published by the International Commission of Large Dams (ICOLD). The horizontal seismic coefficients range from 0.1 to 0.15 and the minimum factors of safety range from 1.0 to 1.5. The Corps of Engineers Manual (EM-1110-2-1902), published in 1982, recommended a seismic coefficient value of 0.1 or 0.15 for regions where earthquake threat is major and great, respectively, and a minimum FOS of 1.0 for all magnitude earthquakes. Historically, values of k in the range 0.05–0.15 have been adopted for seismic analysis and design of dams in the USA even in regions of very high seismic susceptibility such as California. In Japan, which also has high seismic susceptibility, typical values of k less than 0.2 have generally been adopted (Seed 1979).

Marcuson and Franklin (1983) and Hynes-Griffin and Franklin (1984) related the seismic coefficient value for a dam to the expected PGA at a site. Marcuson and Franklin (1983) recommended a seismic coefficient of $1/3$ – $1/2$ of the PGA at the crest of a dam, whereas Hynes-Griffin and Franklin (1984) recommended a seismic coefficient of $1/2$ of the PGA of bedrock (PGA_{rock}) with a minimum factor of safety of 1.0 and a 20% reduction in shear strength. Bray and Rathje (1998) also related the seismic coefficient to the PGA of bedrock (0.6 or $0.75PGA_{\text{rock}}$). This value is appropriate for seismic stability evaluations of solid-waste landfills, where the allowable levels of deformation are relatively small. Kramer (1996) pointed that although engineering judgment is required for all cases, the criteria of Hynes-Griffin and Franklin (1984) should be appropriate for most slopes.

Pyke (1991) showed that, for an 8.25 magnitude earthquake, if a pseudo-static analysis using a seismic coefficient equal to one-half the peak acceleration yields a factor of safety greater than 1.0, the displacements are likely to be acceptably small. Similarly, for earthquakes with magnitudes 7.5, 7.0, and 6.5, if the seismic coefficients are taken one-third, one-fourth, and one-fifth of the PGA, and the computed factors of safety are greater than 1.0, the displacements are likely to be acceptably small.

Krinitzky (1993) suggested one-half of PGA to use in an area of low seismicity (peak acceleration < 0.15 g) for the stability of earth embankments. This can be obtained from the peak horizontal motion (mean) from Modified Mercalli Intensity (MMI), magnitude-distance attenuation, and the probability of a 50-year, 90% non-exceedance. However, in an area of moderate to strong seismicity ($0.15 \text{ g} \leq \text{PGA} \leq 0.40 \text{ g}$), PGA is obtained from the peak horizontal motion, from MMI, magnitude-distance attenuation, and probability of 250-year, 90% non-exceedance.

Kavazanjian et al. (1997) suggested a minimum pseudo-static FOS of 1, also based on a soil strength reduction factor of 0.8 and the following values of 'a', expressed as a proportion of the PGA on soil sites: $a = 0.17$ PGA, if dynamic response analysis is also to be performed for the slope or earthquake structure, and $a = 0.5$ PGA, if dynamic response analysis is not to be performed for the slope or earth structure.

Stewart et al. (2003) used the data of Bray and Rathje (1998) to develop an expression for the seismic coefficient in terms of ground motion (PGA, duration) and earthquake parameters (magnitude). These seismic coefficient values generally range from 0.25 to $0.75\text{PGA}_{\text{rock}}$.

Table 2.2 lists several recommendations for selecting a pseudo-static coefficient. All of these recommended pseudo-static coefficients can fall into two categories: (1) magnitude-based coefficients and (2) peak ground acceleration (PGA)-based coefficients.

In particular, it is recommended that pseudo-static analysis, which provides only a very rough approximation of slope behavior during earthquake shaking, should be used only for preliminary assessments and screening procedures, then followed by more sophisticated analysis (Stewart et al. 2003; Wasowski et al. 2011; Jibson 2011).

2.2.1.2 Direction

It is almost common that only the horizontal acceleration is considered in evaluating the stability and deformation of a slope because the horizontal acceleration is the principal de-stabilizing force that acts on earth structures as well as the principal source of damage observed in earthquakes (Anderson and Kavazanjian 1995).

Table 2.2 Pseudo-static coefficient from several studies

Recommended pseudo-static coefficient (k/g)	FOS	Permanent displacement (D/m)	Original application	References
0.1 (R-F = IX) 0.2 (R-F = X) 0.5 (R-F = XI)	>1.0	–	Nature or man-made slope	Terzaghi (1950)
0.1 ($M = 6.5$) 0.15 ($M = 8.25$)	>1.5	<1	Earth dams	Makdisi and Seed (1977)
0.1 ($M = 6.5$) 0.15 ($M = 8.25$)	>1.15	<1	Earth dams	Seed (1979)
(1/3 ~ 1/2) PGA	>1.0	–	–	Marcuson (1981)
1/2 PGA	>1.0	<1	Earth dams	Hynes-Griffin and Franklin (1984)
1/2 PGA ($M = 8.25$) 1/3 PGA ($M = 7.5$) 1/4 PGA ($M = 7.0$) 1/5 PGA ($M = 6.5$)	>1.0	–	–	Pyke (1991)
0.15	>1.1	–	Dams	CDCDMG (1997)
(0.6 ~ 0.75) PGA_{rock}	>1.0	<0.15 ~ 0.3	Solid-waste landslides	Bray and Rathje (1998)
(0.25 ~ 0.75) PGA_{rock}	>1.0	<0.05 ~ 0.15	Urbanized slopes	Stewart (2003)

Note R-F is Rossi-Forel earthquake intensity scale, IX severe earthquake, X destructive earthquake, XI catastrophic earthquake, M is earthquake magnitude, PGA is peak ground acceleration, in terms of acceleration of gravity

From Fig. 2.1, the horizontal force clearly increases the driving force and decreases the FOS. The vertical pseudo-static force typically has less influence on the FOS than the horizontal pseudo-static force does because the vertical pseudo-static reduces both the driving force and resisting force. As a result, the effects of vertical accelerations are frequently omitted in pseudo-static analysis (Kramer 1996).

Analyses performed by several investigators with an inclined seismic force (i.e., coupled with vertical component of the earthquake force) have shown that the inclination can have a significant influence on the seismic slope stability analysis (Chopra 1966; Ling 1998).

The critical value of the FOS is not necessarily associated with the horizontal direction of the lateral force. In general, a horizontal direction is assumed for the lateral force. There may, however, be situations in which the vertical component of the earthquake force must be taken into consideration.

2.2.1.3 Acting Point

Incorporating the lateral force in an analysis requires a decision as to its point of application. Generally, it is applied at the centre of gravity of a potential sliding mass or of a typical vertical slice in any method of slices. Alternatively, the force may be applied at the level of the base of a slice. Different points of application of pseudo-static force can induce a significant difference in the result. Seed (1979) provided a well-known example, the analysis of Sheffield Dam. In his study, the seismic forces were applied at the base and the center of gravity of each slice, respectively, and the results of factor of safety were 1.21 and 1.32, respectively (Chowdhury et al. 2010).

In summary, pseudo-static method can be simply and directly used to identify the FOS and the critical seismic coefficient k_c , but it cannot simulate the transient dynamic effects of earthquake shaking, because it assumes a constant unidirectional pseudo-static acceleration. In addition, performance of slope is closely related to permanent displacement, but the results of pseudo-static method are difficult to interpret the performance of slope after a seismic event because this method provides no information about permanent displacement. Therefore, in particular, it is recommended that pseudo-static methods should be used only for preliminary assessments and screening procedures, which then should be followed by more sophisticated analysis (Stewart et al. 2003; Wasowski et al. 2011; Jibson 2011).

2.2.2 Dynamic Sliding Block Methods

Displacement-based dynamic sliding block method is another alternative approach to evaluate the seismic slope stability, as permanent displacement is a useful index of slope performance, especially for those man-made slopes constructed for special purposes such as dams and embankments. This method has been widely used in earthquake geotechnical engineering.

In the Rankine Lecture of 1965, Newmark (1965) first proposed the basic elements of a procedure for evaluating the potential displacements of an embankment due to earthquake shaking. Newmark envisaged that sliding would be imminent once the inertia forces on a potential failure block were large enough to overcome the yield resistance and that movement would stop when the inertia forces were reversed.

In his analysis, a soil mass moving downward along a failure surface under inertia force due to earthquake shaking is considered to be analogous to a rigid block with weight and an external force sliding on an inclined plane as shown in Fig. 2.2. In this method, a pseudo-static analysis is first used to calculate a critical or yield acceleration value a_c . The permanent displacement for a sliding mass is then calculated by double integration of the earthquake acceleration time history data above the critical acceleration. Newmark showed that the critical or yield

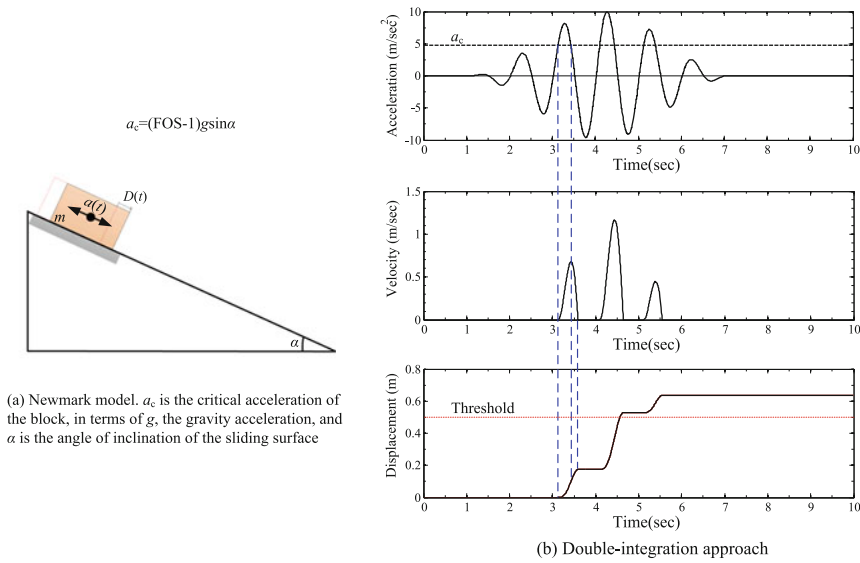


Fig. 2.2 Illustration of the original Newmark's method

acceleration, parallel to the slope, of a potential landslide block is a simple function of the static factor of safety and landslide geometry, expressed as:

$$a_c = (\text{FOS} - 1)g \sin \alpha \quad (2.4)$$

where a_c is in terms of the gravity acceleration g ; FOS is the static factor of safety; and α is the angle from the horizontal of the sliding surface.

It should be noted that his concept implied that movements would stop when the inertia forces were reversed. The velocity could remain positive even if the inertia forces were reversed or the inertia forces were not reversed but less than the yield resistance on the potential failure surface. Positive velocity thereby causes sliding on the surface. On the other hand, the velocity could be negative even though the inertia forces were greater than yield resistance. It all depends on the magnitude and direction of both velocity and inertia force, while not either one alone. Besides, as also indicated by Newmark, the uphill resistance without serious error in the calculations may be taken as infinitely large. In this situation, ground motions in the direction of the downward slope tend to move the mass downhill, but ground motions in the upward direction along the slope leave the mass without relative additional motion except where these are extremely large in magnitude. Thus, the negative velocity or velocity heading uphill is not allowed in this analysis.

Since then, the method has been numerous extended and applied. The Sects. 2.2.2.1 and 2.2.2.2 will give reviews for these two aspects, respectively. In addition, a regional scale application of the dynamic sliding block method is reviewed in Sect. 2.2.2.3.

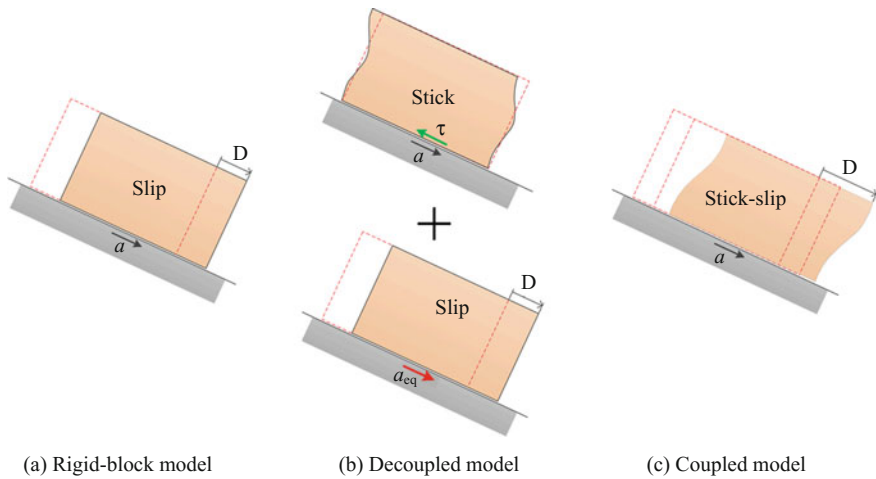


Fig. 2.3 Calculated models of permanent displacement

2.2.2.1 Extensions

Considerable attention has been focused over the last decades on developing procedures to more accurately analyze the seismic performance of a slope for dams, embankments, or other important structures by modeling the dynamic slope response more rigorously. Soon after Newmark published his rigid-block method, more sophisticated analyses were developed to account for the fact that sliding masses are not rigid bodies but deform internally when subjected to seismic shaking (Seed and Martin 1966; Lin and Whitman 1983). The most commonly used of such analyses was developed by Makdisi and Seed (1977) and calibrated to earth dam using a small number of strong motion records. They produced design charts for estimating seismic displacements as a function of slope geometry, earthquake magnitude, and the ratio of critical or yield acceleration to peak acceleration. Jibson (2011) grouped analytical procedures for estimating permanent coseismic landslide displacements into three types: (1) rigid-block model (Newmark 1965), (2) decoupled model (Makdisi and Seed 1978; Bray and Rathje 1998), and (3) coupled model (Lin and Whitman 1983; Rathje and Bray 2000; Bray and 2007) (Fig. 2.3).

2.2.2.2 Applications

Since the rigid-block method was published in 1965 by Newmark, it has seen numerous applications, four of which are shown in Fig. 2.4. As reviewed by Garini et al. (2011), the applications in recent years include (1) the seismic deformation analysis of earth dams and embankments (Seed and Martin 1966; Ambraseys and Sarma 1967; Sarma 1975, 1981; Franklin and Chang 1977; Makdisi and Seed 1977;

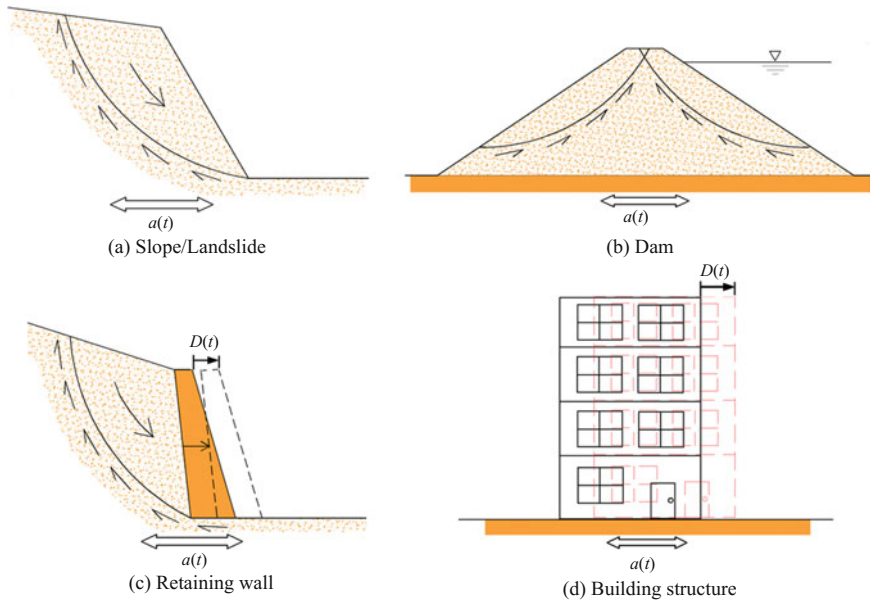


Fig. 2.4 Applications of dynamic sliding block method in geotechnical engineering

Lin and Whitman 1983; Constantinou and Gazetas 1987; Yegian 1991; Yegian et al. 1991; Sawada et al. 1993; Gazetas and Uddin 1994; Kramer 1996; Kramer and Smith 1997; Rathje and Bray 1999); (2) the displacements associated with landslides (Wilson and Keefer 1983; Jibson 1993; Harp and Jibson 1995; Del Gaudio et al. 2003); (3) the seismic deformation of landfills with geosynthetic liners (Bray and Rathje 1998; Yegian et al. 1998); (4) the seismic settlement of surface foundations (Richards et al. 1993); and (5) the potential sliding of concrete gravity dams (Leger and Katsouli 1989; Danay and Adeghe 1993; Fenves and Chopra 1986). The extension of the analogue by Richards and Elms (1979) to gravity retaining walls has met worldwide acceptance and has found its way into seismic codes of practice. Several other generalized applications have also appeared (e.g., Ambraseys and Menu 1988; Ambraseys and Srbulov 1994; Stamatopoulos 1996; Rathje and Bray 2000; Ling 2001; Fardis 2009; Wartman et al. 2003).

2.2.2.3 Regional Scale Analysis

Estimating regions where earthquake-induced landslides are likely to occur and what kind of shaking conditions will trigger them is an important topic in regional landslide seismic hazard assessment (Hsieh and Lee 2011).

Many slope stability analysis methods are not generally applicable for a regional scale analysis. For zonation purposes, it is necessary to quantify in a simple form

both the vulnerability of slopes and the triggering mechanisms that can induce instability (Varnes 1984; Rodriguez et al. 1999). With the development of geographic information systems (GIS) tools in recent years, regional scale analyses by the dynamic sliding block method have been proposed. These analyses are based on the integration of seismic shaking parameters, ground geotechnical and geomorphic data including landslide inventories (e.g., Mankelov and Murphy 1998; Miles and Ho 1999; Jibson et al. 1998, 2000; Miles and Keefer 2000; Del Gaudio et al. 2003; Saygili and Rathje 2009). Newmark displacement (Newmark 1965) is commonly taken as a measure of permanent displacement caused by seismic shaking along a slide surface.

The Newmark analysis (which combines slope stability calculations with seismic ground motion records) is widely used to evaluate the potential for landslides that could be triggered by earthquake shaking (Jibson 2000; Jibson and Jibson 2003; Jibson 2007). In essence, Newmark-based seismic landslide hazard or, strictly speaking, spatial variation in slope failure probability or susceptibility, is expressed through regional distribution of Newmark displacements (D_n) that are predicted via empirical relations calibrated using actual seismic landslide records (e.g., Jibson 2000). Subsequent work by Jibson (2007), based on a larger data set of strong motion records (2270 strong motion records from 30 worldwide earthquakes), provided a series of new/updated regression equations to estimate D_n as a function of (i) critical acceleration ratio, (ii) critical acceleration ratio and magnitude, (iii) Arias intensity (I_a) and critical acceleration, and (iv) Arias intensity and critical acceleration ratio. Although all of the regressions have standard deviations of roughly ± 0.5 log units, which correspond to an order of magnitude variation in estimated Newmark displacements, this does not compromise their principal intended use as an indicator of relative hazard that can be quickly obtained for preliminary regional scale assessments (Jibson 2007). Most recently, Hsieh and Lee (2011) used strong motion data from the 1999 Chi-Chi earthquake, the 1999 Kocaeli earthquake, the 1999 Duzce earthquake, the 1995 Kobe earthquake, the 1994 Northridge earthquake, and the 1989 Loma Prieta earthquake to further refine the relationship among critical acceleration (a_c), Arias Intensity (I_a), and Newmark displacement (D_n). After testing different forms of the regression equations, it is shown that an addition of $a_c \log I_a$ term to the Jibson's (1993) empirical formula produces the best results in terms of higher goodness of fit ($R^2 = 0.89$) and lower range of displacements (standard deviation $D_n = 0.295$). Hsieh and Lee (2011) also indicated that it is useful to separate the Taiwanese dataset from the other data and thus developed Taiwan-specific regressions. Similarly, it was shown that the development of separate empirical equations for rock and soil sites leads to statistically better estimates of D_n . Meunier et al. (2007) investigated patterns of well-documented events of widespread seismic landslides in relation to the strong motion data, and their results can be used where accelerometer records and geotechnical information are not available.

2.2.3 Stress-Strain Methods

With the developments of computer technology and simulation approach in recent decades, the numerical simulation method is becoming increasingly used in engineering practice and more and more popular for the real dynamic analysis. These methods can be categorized into continuous methods, e.g., finite element method (FEM) (Clough 1960), finite difference method (FDM) (Mitchell and Griffiths 1980), boundary element method (BEM) (Brebbia and Wrobel 1980), and discontinuous methods, e.g., rigid-block spring method (RBSM) (Kawai 1977, 1978), discrete element method (DEM) (Cundall 1971), and discontinuous deformation analysis (DDA) (Shi and Goodman 1985, 1989).

2.2.3.1 Continuous Methods

Clough (1960) developed and named FEM of engineering analysis based on mathematical methods first developed by Courant (1943) (Jibson 2011). This method uses a mesh to model a deformable system as shown in Fig. 2.5. This method soon began to be applied to slopes, and it provided a valuable tool for modeling the static and dynamic deformation of soil systems.

Several applications of continuous modeling to earth structures have been developed and published; Kramer (1996) provided a good summary of various methods and their associated studies. Seed (1973) analyzed the failures of the Upper and Lower San Fernando dams during the 1971 San Fernando earthquake (M 6.6) by using a finite element model to estimate the strain potential at each node based on cyclic laboratory shear tests of soil samples. Lee (1974) and Serff (1976) used the strain-potential method to model the reduction in the stiffness of soils and thus the permanent slope deformation. More recently, nonlinear inelastic soil models have been developed and implemented in two-dimension (2D) and three-dimension (3D) models (e.g., Prevost 1981; Griffiths and Prevost 1988; Elgamal et al. 1990;

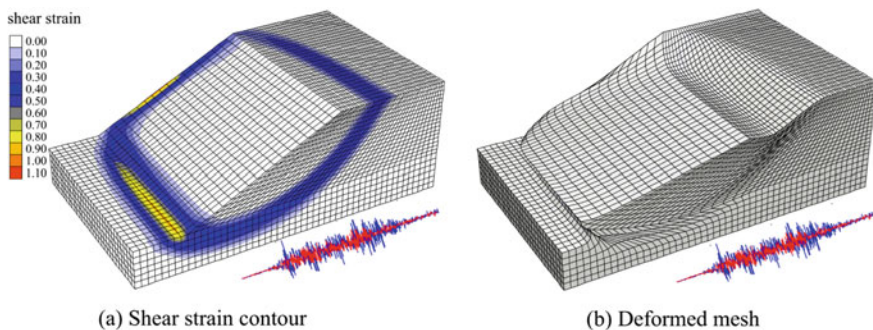


Fig. 2.5 Example of seismic slope analysis by stress-strain methods (Zhang et al. 2013)

Taiebat et al. (2011). In addition, Zheng et al. (2009), Latha and Garaga (2010) and Zhang et al. (2012a) studied the seismic slope stability by using FDM.

2.2.3.2 Discontinuous Methods

For the analysis of a sliding mass consisting of multiple discrete blocks, e.g., a jointed rock slope as shown in Fig. 2.6, the discontinuous methods are more applicable (Ning and Zhao 2012). In the discontinuous methods, the interactions between individual blocks, which will influence the reaction of each block under the seismic loadings, can be conveniently considered. Some applications of RBSM and DEM in such kind of analysis can be found in the literature (Kawai 1977; Kawai et al. 1978, 1981; Niwa et al. 1984; Hamajima et al. 1985) and (Zhang et al. 1997; Papantonopoulos et al. 2002; Psycharis et al. 2003; Bhasin and Kaynia 2004; Pekau and Cui 2004; Komodromos et al. 2008; Papaloizou and Komodromos 2009; Pal et al. 2011), respectively.

DDA is another discontinuous method developed for the modeling of the behaviors of block systems. Since the novel formulation and the numerical code of DDA were presented, DDA draws more and more attention and many modifications and improvements to the original formulas have been proposed to overcome some of its limitations (Ke 1996; Lin et al. 1996; Koo and Chern 1998; Cheng 1998; Doolin and Sitar 2004; Doolin 2005; Wang et al. 2012) and make it more efficient, suitable, and practical to seismic slope stability.

Early applications of DDA to static rock slope engineering can be found in Ohnishi et al. (1995), Ke (1996), Zhao et al. (1997), Wu et al. (1997), Chen and Ohnishi (1999), and Luan et al. (2000). Hatzor et al. (2002, 2004) and Hatzor (2003) demonstrated a fully dynamic 2D DDA stability analysis of a highly discontinuous rock slope using simple rock bolting pattern based on results from dynamic DDA.

In summary, stress-strain method represents a powerful alternative approach for seismic slope stability analysis which is accurate and versatile and requires fewer a

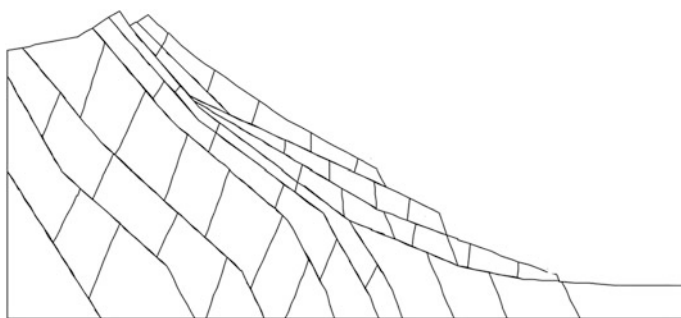


Fig. 2.6 A jointed rock slope (modified from Bhasin and Kaynia 2004)

priori assumptions, especially, regarding the shape of failure surface. However, stress-strain methods use highly complex models that, to be worthwhile, require a high density of high-quality data and sophisticated soil-constitutive models to predict the stress-strain behavior of the soils. For this reason, stress-strain methods are generally practical only for critical projects such as earth dams and slopes affecting critical lifelines or structures. Stress-strain methods are innately site-specific and cannot be applied to regional problems.

2.3 Landslide Run-Out Analysis

The estimation of the movement behavior of a potential landslide is very important for mitigating the landslide disaster. Especially, the run-out distance is one of the major parameters in landslide risk assessment and preventive measure design. Long run-out is one of the major characteristics of earthquake-induced landslides. Many researchers have made great effort to understand how and why large falling masses of rock can move unusually long run-out distance. Researchers have repeatedly revisited the problem using a wide variety of approaches. These efforts can fall into four categories: (1) experimental methods, (2) empirical methods, (3) analytical methods, and (4) numerical simulation methods. The states of the art of these methods are reviewed in the following four Sects. [2.3.1](#)–[2.3.4](#).

2.3.1 *Experiment Methods*

Physical modelling typically involves using scale models to capture the motion of landslides. Physical experiments are usually preferred to models because models require more assumptions than direct measurements. But for landslides, direct experiment is difficult, dangerous, expensive, and of limited utility. Based on laboratory experiments and field investigation data, there are many different available models developed for calculating run-out zones.

Some full-scale direct experiments with artificial landslides have been completed (Okura et al. [2000a, b, 2002](#); Ochiai et al. [2004](#); Moriwaki et al. [1985](#); and others). However, since landslides are frequently heterogeneous and single event cannot be repeated carefully through adjusting only one factor, direct experiment is difficult, dangerous, expensive, and of limited utility. And observing conditions are complicated by the danger of being in close proximity to a landslide and the difficulty of measuring a material with properties that change when observed in-situ or when isolated for measurement. But laboratory experiments are still the first qualitative and quantitative observations on the obtained results became fundamental for a better understanding of movement run-out behaviour.

2.3.2 Empirical Methods

Several empirical methods for assessing landslide travel distance and velocity have been developed based on historical data and on the analysis of the relationship between parameters characterizing both the landslide, e.g., the volume of the landslide mass, and the path, e.g., local morphology, and the distance travelled by the failure mass (Hungr and Evans 2004). Regression model-based methods and geomorphology-based methods are two kinds of common methods.

2.3.2.1 Regression Model-Based Methods

The regression model-based methods are developed on an apparent inverse relationship between landslide volume and angle of reach [also called as *fahrböschung* by Heim (1932)]. Several linear regression equations have been proposed (Scheidegger 1973; Li 1983; Corominas 1996). Introduced by Heim (1932), the angle of reach is the inclination of the line connecting the crest of the source with the toe of the deposit, as measured along the approximate streamline of motion as shown in Fig. 2.7. The angle of reach is considered an index of the efficiency of energy dissipation, and so is inversely related to mobility. Similar correlations between volume and other simple mobility indices have been proposed (Hsü 1975; Davies 1982; Rickenmann 1999). Given estimated source location, volume, and path direction, these methods provide estimates of the distal limit of motion (McDougall 2006a, b).

An alternative principle suggested by Evans and Hungr (1993) is the *minimum shadow angle*, following Lied (1977). This is the angle of a line between the highest point of the talus slope and the stopping point of the longest run-out. According to an investigation of 16 talus slopes in British Columbia by Evans and Hungr (1993), the *minimum shadow angle* is more preferable, but both should only be used for a first approximation of run-out zone.

Keylock and Dommas (1999) tested three empirical models on their ability to predict the maximum length of run-out zone. Their first model was the *height*

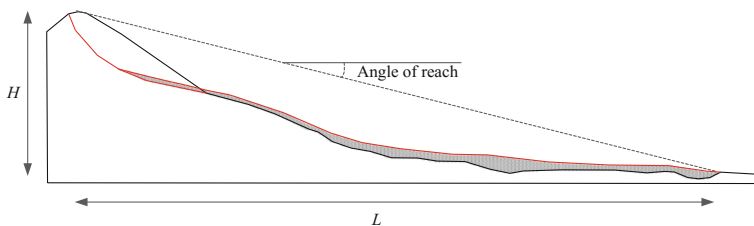


Fig. 2.7 Schematic definition of the angle of reach. H is the elevation difference between the crest of the source and the toe of the deposit, and L is the length of the horizontal projection of the streamline connecting these two points

function model, in which the run-out distance derived from the combined vertical height of the free rock face and the talus slope. The second was the α - β *model* following Heim (1932), Hsü (1975), and Köner (1980), which is based on the correlation between the average energy of an extreme rockfall event. Their third model was *run-out ratio model*, developed by McClung and Lied (1987), which describes the ratio between the horizontal length of the run-out zone to the combined horizontal length of the talus slope and the free rock face. After investigated using rockfall data presented by Domaas (1994), the most accurate model of the three appeared to the *run-out ratio model*.

Improved empirical model notable performing regressions on subsets with varying scopes were presented by Cannon (1993), Corominas (1996), Jakob and Hungr (2005), and others.

Regression model-based models play a valuable role in landslide run-out analysis due to the regression model-based methods are simple. But the regression model-based methods are difficult to apply in practice with a high degree of certainty. For example, the correlation coefficients for some of regression models are 0.7–0.8, while a value of larger than 0.95 generally indicates a strong correlation. And it is difficult in this method to take account of influences of the ground condition, the micro-topography, the degree of saturation of the landslide mass. For this point, geomorphology-based method is another alternative approach to predict the run-out of landslide.

2.3.2.2 Geomorphology-Based Methods

Field work and photograph interpretation are the main sources of the geomorphological analysis for determining the travel distance of landslides (Hungr et al. 2004). The assessment of the extent of both ancient and recent landslide deposits is the basis for defining future travel distances. The outer margin of the landslide deposits gives an appraisal of the maximum distances that landslides have been able to reach during the present landscape (Fig. 2.8). Several authors have provided these studies (e.g., Costa 1984; Kostashuk 1987; Copons et al. 2004; Copons and Vilaplana 2008).

The geomorphological approach does not give any clue of the emplacement mechanism. Furthermore, the slope geometry and the circumstances responsible for past landslides might have changed. Therefore, results obtained in a given place cannot be easily exported to other localities.

In summary, empirical methods, both regression model-based methods and geomorphology-based methods, typically predict travel distances, while the deformation characteristics or the slide velocities of the landslide are not predicted. These models may be applied to establish initial hazard characteristics for preliminary run-out analysis, which may be later refined by other models.

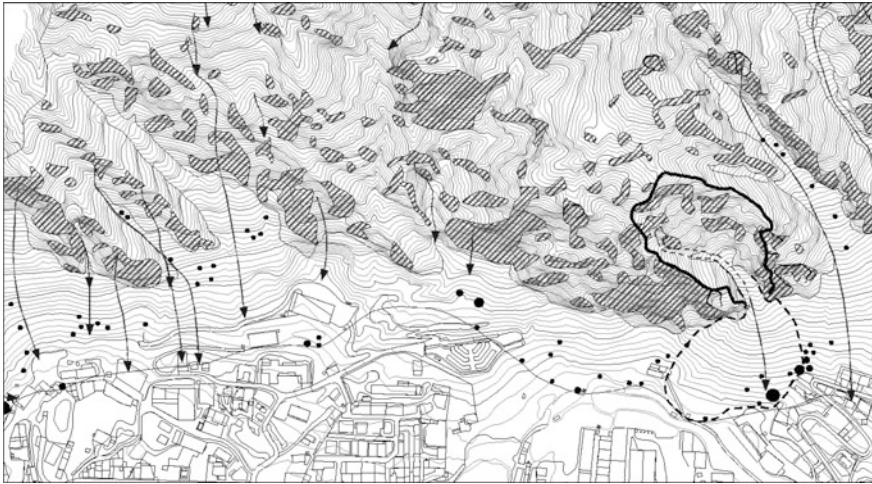
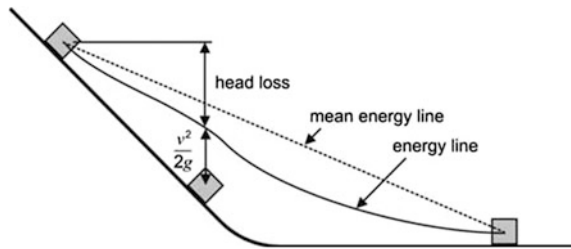


Fig. 2.8 Boundary of the potential rockfall run-out area in Santa Coloma (Principality of Andorra), defined by the line that links the farthest fallen blocks observed in the field (Copons 2004). Arrows indicate historical rockfall paths and solid circles are large fallen boulders (Hungre et al. 2004)

2.3.3 Analytical Methods

In contrast to empirical methods, analytical methods are based on mechanics and involve the solution of motion equations (McDougall 2006a, b). The simplest analytical model is the classical sliding block model as shown in Fig. 2.9, which is based on work-energy theory (Kirby and Statham 1975; Bozzolo and Pamini 1986; Hungre and Evans 1988; Pfeiffer and Bowen 1989; Kobayashi et al. 1990; Evans and Hungre 1993; Azzoni et al. 1995; Chau et al. 1996; Bozzolo and Pamini 1986; Azzoni et al. 1995; Müller-Bernet in Heim 1932; Sassa 1988). Internal deformation and its associated energy dissipation are neglected and the landslides is treated as a lumped mass. At any position along the path, the sum of the energies including the potential energy, kinetic energy, and net energy loss equals the initial potential energy. This energy balance can be visualized using the concept of energy grade lines, as shown in Fig. 2.9. The concept of energy grade lines is useful for visualizing the energy balance. v is the velocity of the block, g is the vertical acceleration due to gravity, and $v^2/2g$ is known as the velocity head, which is the kinetic energy of the block normalized by the product of its mass and g . The same normalization of net energy loss is known as head loss. Note that the positions of the energy lines are referenced to the centre of mass of the block and that the true energy line and mean energy line do not necessarily coincide. Given the initial position of the center of mass and a suitable relationship to approximate the energy losses, the position and velocity of the block can be determined at any given time.

Fig. 2.9 Classical sliding block model, based on work-energy theory (McDougall 2006a, b)



To compute the velocity before and after bounce, two principle approaches were used both considering the basis of energy loss after a bounce. One approach defined energy loss by a coefficient for the efficiency of collision and the other calculated energy loss on the basis of a tangential coefficient of restitution and a normal coefficient.

Three dimensional for investigating run-out at a slope scale were also developed (Descœudres and Zimmermann 1987; Guzzetti et al. 2002; Dorren et al. 2004; Lan et al. 2007 and Masuya et al. 2009). These models require a high-resolution digital elevation model (DEM).

Generally speaking, the use of analytical methods is somewhat motivated by the limitations of purely empirical methods, as the unique geometry and materials involved in each case can be accounted for explicitly and a statistically significant database of previous events is not necessarily required. The simplicity of a lumped mass allows analytical solutions, fast and effectively (Hürlimann et al. 2008). However, because the landslide is reduced to a single point, lumped mass models cannot provide the exact maximum run-out distance, but only the displacement concerning the centre of mass (Evans et al. 1994; Hungr 1995).

2.3.4 Numerical Simulation Methods

The single-block model should be only applied to the motion of the center of mass of a rigid body, but more complex continuum deformable mass or multi-block system is often appeared in practice. Some numerical simulation methods have been developed to account explicitly for deformation during motion.

2.3.4.1 Continuous Methods

When considering that the dimensions of a typical particle is much smaller than the depth and length of the debris, the debris mass is treated as continuum. According to depth-averaged Saint Venant approach, the material is assumed to be incompressible and the mass and momentum equations are written in a depth-averaged form. Many numerical methods now exist to investigate the run-out process of

landslide (e.g., Savage and Hutter 1989; Sousa and Voight 1991; Takahashi et al. 1992; Hungr 1995; Chen and Lee 2000; Denlinger and Iverson 2001; Crosta et al. 2003, 2005, 2007; McDougall and Hungr 2004; Pirulli 2005; McDougall 2006a, b). These methods are usually based on continuum mechanics and assume that the avalanche thickness is very much smaller than its extent parallel to the bed, i.e., thin layer depth-averaged models. The primary differences are their representation of basal resistance force and the constitutive relations describing the mechanical behavior of the considered material. These models can accurately take account of detailed topography effects, shown to be significant, with a reasonable computational time, making it possible to perform sensitivity studies of the parameters used in the model. They can provide effective properties that make it possible to roughly reproduce not only the deposit shape, but also the dynamic as shown in Favreau et al. (2010) and Moretti et al. (2012) for examples. However, conventional continuum approaching models, which neglects the contact between rocks, makes it impossible to trace the position of individual rock during a landslide.

2.3.4.2 Discontinuous Methods

When the landslide mass consists of large fragments and boulders, the run-out mass is modelled as an assembly of blocks moving down a surface. By applying known individual constitutive properties, contact laws, velocities, displacements, and body forces, their dynamic behavior can be studied over a selected period in time. Some authors take circular shape models in their run-out analysis to evaluate maximum run-out and final deposit position of past or potential events (e.g., Poisel et al. 2008). Although polygonal shapes have the disadvantages due to the complexity of the contact patterns and penalty in computational time, methods using non-circular shapes will be required for more real-world problems. It is more appropriate when problems are limited in finite blocks. Discontinuous numerical simulation methods are powerful tools in simulation of failure and run-out process of rock avalanche controlled by weakness surface. DEM (Cundall 1971) and DDA (Shi and Goodman 1985, 1989; Shi 1988) are two of the most commonly used methods.

Both DEM and DDA employ the equations of dynamic motion which are solved at finite points in time, in a series of time steps, but there are some subtle but significant differences in their formulations of the solution schemes and contact mechanics. In the solution schemes, equations of motion in DDA are derived using the principle of minimization of the total potential energy of the system, while the equations of motion as implemented in DEM are derived directly from the force balance equations, which still resultant unbalanced force after a time step and damping is necessarily used to dissipate energy. In the contact mechanics, the DDA used a penalty method in which the contact is assumed to be rigid. No overlapping or interpenetration of the blocks is allowed as the same as real physical cases, whereas soft contact approach is used in DEM. The soft contact approach requires laboratory or field measured joint stiffness, which may be difficult to obtain in many

cases. Many comparisons of basic models (sliding, colliding, and rolling models) between the DEM and DDA were carried out and show that the results from DDA are more close to the analytical values than that from DEM (Zheng 2010). Compared to DEM, DDA has a simpler and more straightforward physical meaning (Wu 2003).

Applications of DEM can be found in some literatures, such as Zhang et al. 1997, Papantonopoulos et al. 2002, Psycharis et al. 2003, Pekau and Cui 2004, Komodromos et al. 2008, Papaloizou and Komodromos 2009.

DDA is a dynamic numerical analysis method capable of evaluating the impacts area of an earthquake-induced landslide when seismic impacts are integrated into simulations. Hatzor and Feintuch (2001) was the first to validate the use of DDA in simulating dynamic landslides by studying the dynamic of block sliding on an inclined plane, in which they assumed that the base block was fixed and earthquake accelerations were directly considered as body force and added to the sliding block in DDA. Based on the same input model of seismic loadings, Makris and Roussos (2000), Shi (2002), Kong and Liu (2002), Ishikawa et al. (2002), Hatzor et al. (2004), Tsesarsky et al. (2005), Yagoda and Hatzor (2010), and Bakun-Mazor et al. (2012) studied the dynamic response or/and stability analysis of tunnel, slope, dam, foundation, or ancient masonry structure by using DDA. Alternatively, Sasaki et al. (2004) developed an acceleration input method different from the original DDA algorithm to simulate the dynamic behavior of a slope with sliding block. In his method, the seismic accelerations were applied to the base block, which is different from the former seismic loadings input model. Sasaki et al. (2007) applied the same earthquake input model to analysis several cases of simple block structures under harmonic accelerations to acquire the relationships between natural frequencies of elastic block structures and applied accelerations. Later, Wu group (2009, 2010, 2011) applied DDA to simulate the kinematic behavior of sliding rock blocks in the Tsaoling landslide and the Chiu-fen-erh-shan landslide induced by the 1999 Chi-Chi earthquake. Recently, Zhang et al. (2012b) applied newest DDA program to simulate the largest landslides induced by the 2008 Wenchuan earthquake.

2.4 Comparisons of Various Methods and Conclusions

The studies in the field of the earthquake-induced landslides are generally reviewed. Two parts of contents (i) seismic stability analysis and (ii) run-out analysis are reviewed and compared. Some conclusions can be drawn:

- (1) Three categories of methods can be used to analyze the seismic stability of a slope. Each of these types of methods has strengths and weaknesses and each can be appropriately applied in different situations. In detail, pseudo-static methods can simply and directly determine the FOS and the critical coefficient k_c of a slope, while the widely used Newmark's methods and its extensions can

determine the coseismic deformation of a slope. And the Newmark's methods can be used to estimate where earthquake-induced landslides are likely to occur and what kind of shaking conditions will trigger them based on the GIS technology. More sophisticated analysis for real dynamic process of a seismic slope should be carried out by stress-strain methods, including both continuous methods and discontinuous methods.

- (2) Four kinds of methods can be used to analyze the run-out of a landslide. In detail, experiment method can provide the qualitative and quantitative observations on the obtained results although this method is difficult, dangerous, expensive, and of limited utility. Empirical method can be directly used for assessing landslide travel distance and velocity based on historical data and on the analysis of the relationship between parameters characterizing both landslide and the path. Analytical method can be more directly used without the need of statistically significant database of previous events. Numerical simulation method can be used to provide more information for the landslide composed by the complex continuum deformable mass or multi-blocks.

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