Chapter 2

Theories and Literature Review

2.1 Theories

2.1.1 Types of slope failure

Slides may occur in almost every conceivable manner, slowly or suddenly, and with or without any apparent provocation (Terzaghi and Peck, 1967).

Slope failures depend on the geologic conditions, groundwater and slope geometry. Fig. 2.1 illustrates the types of slope movements, which can be subdividing into 3 major classes: falls, slides and flows. Falls are characterized by movement away from existing discontinuities, such as, joint, fault planes, etc. Slides comprise the movement of large generally

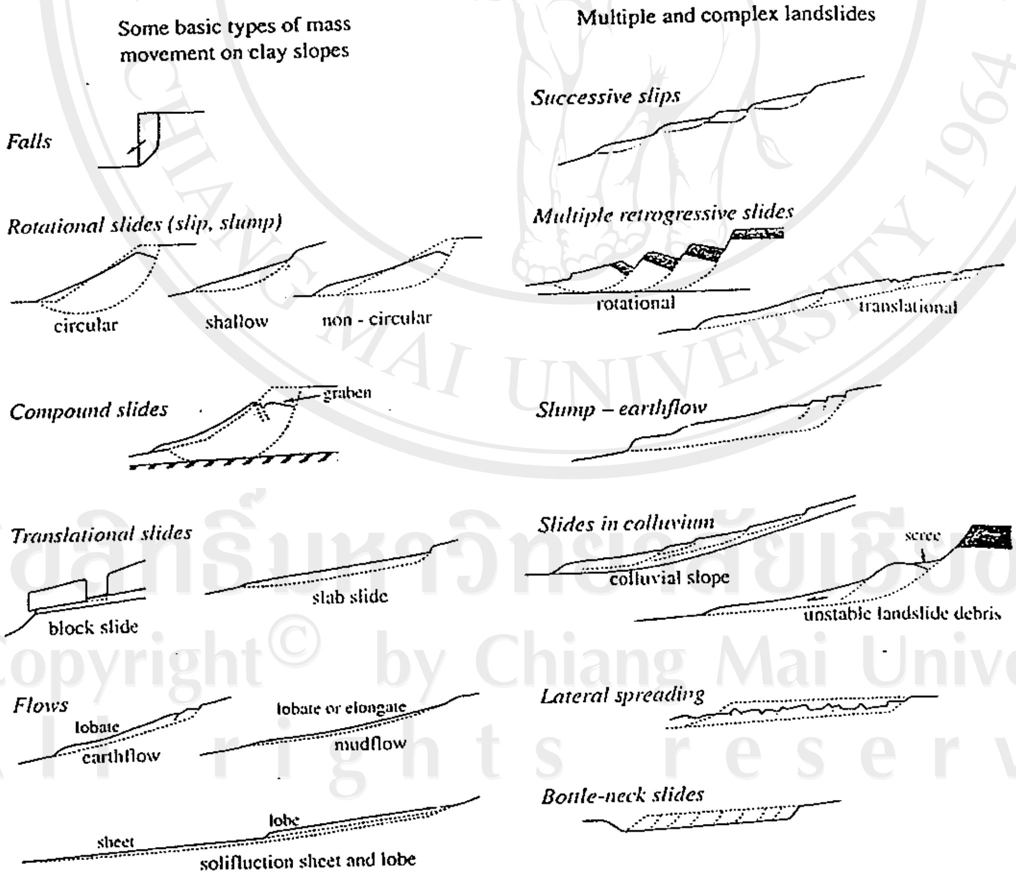
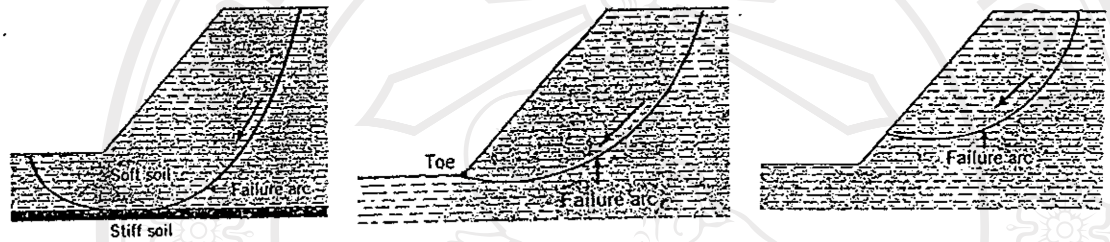


Fig. 2.1 Types of mass movements in soil slopes (Skempton and Hutchison, 1969)

Source : Barnes (2000)

continuous masses of soil on one or more slip surfaces, whereas flows consist of slow movement of softened or weathered debris, or somewhat faster movement of clay debris by water. These usually take the form of either, plane, circular, non-circular or a combination of these types.

The common characteristic of failure in homogeneous soil is almost rotational slip of which slip surface is maybe either circular arc or not. The location of slip surface can be classified by the base failure surface, toe failure surface, and slope failure surface. The modes of failure which often occur are shown according fig. 2.2.



(a) Base slide (b) Toe slide (c) Slope slide

Fig. 2.2 Mode of slope failure (sources from Budhu, 2000)

2.1.2 Causes of slope failure

Slope failures are caused by natural forces, human activities and misjudgment. Some of the main factors that provoke slope failures describe as follows:

a) Erosion (e.g., by rivers, winds, glaciers)

b) Precipitation (e.g., rainfall, snow)

c) Transitory effect (e.g., earthquakes)

d) Unidentified geological features

e) Overloading at the crest of the slope

f) Construction activities

- Excavated slopes

- Fill slopes

g) Effect of pore water pressure (e.g., rapid drawdown)

2.1.3 Basic concepts applied to slope stability analysis

a) Shear strength

The maximum internal resistance to shear on any plane in the soil or the value of the shear stress at which a soil fails in shear be called the shear strength. The shear strength of soils is usually expressed by Mohr-coulomb theory (fig. 2.3) as equation 2.1.

$$S = c + \sigma \tan \phi \tag{2.1}$$

In an effective stress analysis, the effective strength parameters; c', ϕ' are used and the pore pressure must be specified as an independent variable according equation 2.2.

$$S = c' + (\sigma - u) \tan \phi' \tag{2.2}$$

Where S = shear strength along failure surface

c, c' = soil cohesion in terms of total and effective stress

ϕ, ϕ' = angle of internal friction in terms of total and effective stress

σ, σ' = total and effective normal stress on the failure surface

u = pore water pressure

The determination of c -ϕ shear strength parameter can be made by field investigation and laboratory testing.

In order to understand the total stress analysis and effective stress analysis, the intensive understanding in the principles of total stress and effective stress, therefore, is need. In fully saturated soils, if pore water pressure occurred and can be absolutely measured, the effective stress should be used in the analysis. Because the water pressure is not induced the internal resistance. But sometimes in practice, it is difficult to estimate the pore water pressure; such as, immediately loading condition, unsaturated soils condition, etc. The total effective stress usually is applied in the analysis. Thus, the total stress is suitable for short-term analysis, and for long-term analysis of which pore water pressure can be known, the effective stress analysis is preferred.

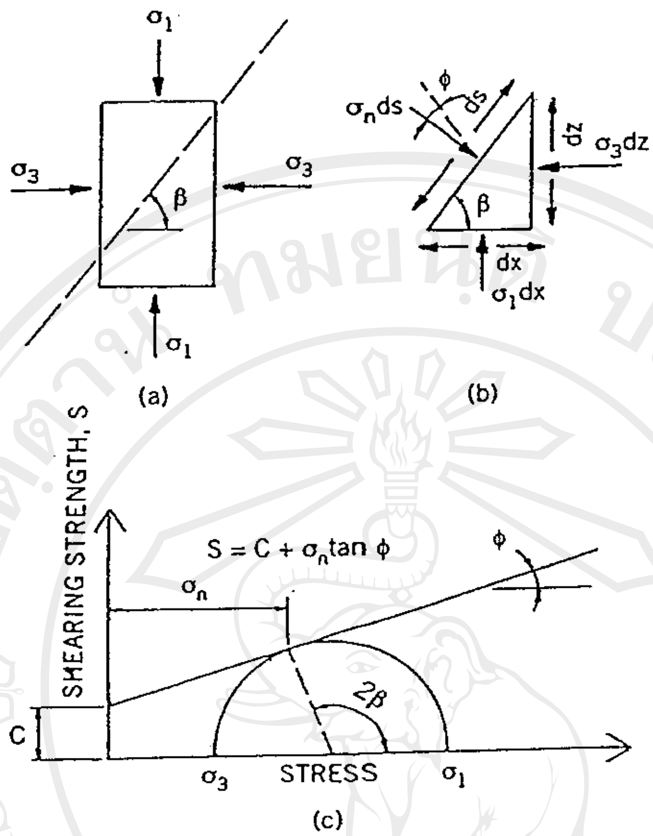


Fig. 2.3 Mohr-Coulomb envelop. (a) Soil element (b) Stress vector (c) Shear strength envelop

Source : Abramson et al. (1996)

Most problems involving natural slopes may be classified as long term although failures generally result from small changes of loading. For example, the groundwater level may rise in a period of heavy rainfall resulting in a decrease of the effective stress and hence in the shear strength along a potential slip surface. Alternatively erosion at the toe of the slope may increase the shear stress and decrease the resistance. Usually an effective stress analysis is the most appropriate for these conditions, but in soils such as loose sand or quick clay in which small shear strains can cause a sudden built–up of pore pressure, great care is needed in deciding on appropriate shear strength parameters and pore pressures. In these circumstances a total analysis may be considered, although there is then the real difficulty of determining the appropriate undrained shear strength (Nash, 1987)

b) Factor of safety

Engineering design of slopes must realize the factor of safety for guarding against ultimate failure, avoiding intolerable deformations and uncertainties associated with the measurement of soil properties and with the analysis (Janbu, 1973). Factor of safety is defined as the ratio of the available shear strength to mobilized shear strength of soil at failure for maintaining equilibrium according equation 2.3. Another way to state this definition is that FS is "the factor by which the shear strength of the soil would have to be divided to bring the slope into a state of barely stable equilibrium" (Duncan, 1996).

$$FS = \frac{\tau_f}{\tau_m} \tag{2.3}$$

where FS = factor of safety

τ_f = shear strength of the soil

τ_m = shearing stress along failure surface

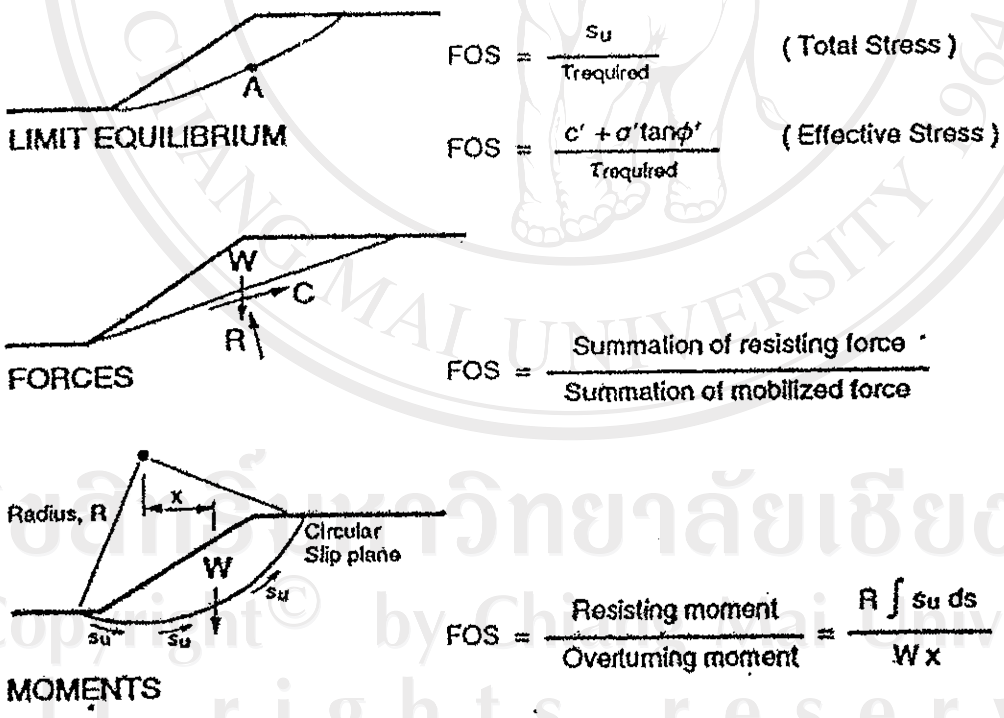


Fig. 2.4 Various definitions of factor of safety

Source : Abramson et al. (1996)

c) Limit equilibrium

Limit equilibrium assumes the soil is rigid-plastic, which means that there are no strains at any point until the failure occurred. A state of limit equilibrium is related by Mohr-Coulomb failure criterion.

From Mohr-Coulomb theory and the definition of factor of safety, the mobilized shear strength, τ can be expressed as

$$\tau = c_m' + (\sigma - u)\tan\phi_m' \tag{2.4}$$

$$c_m' = \frac{c'}{F}, \qquad \tan\phi_m' = \frac{\tan\phi'}{F}, \qquad \tau = \frac{\tau_f}{F}$$

A value of 1/F shows the degree of mobilization of the shear strength.

Fig. 2.5 illustrated the difference between the failure surface (FS=1) and the critical surface (F>1)

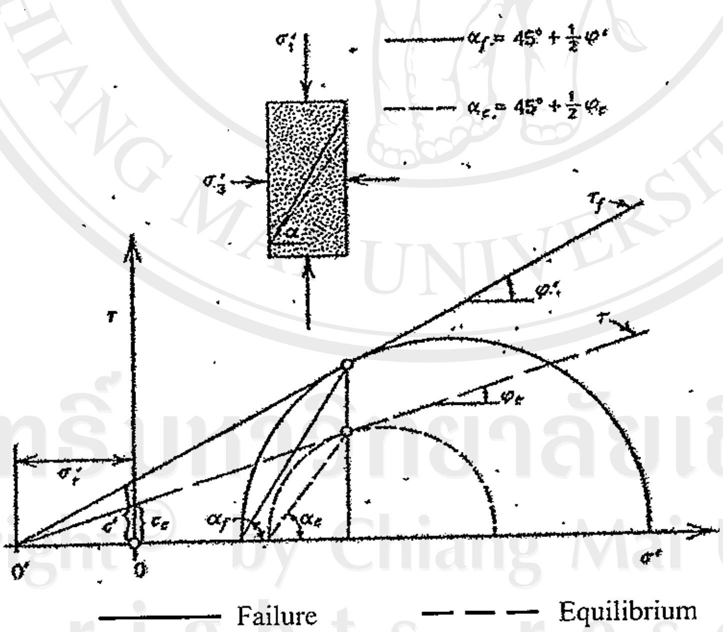


Fig. 2.5 Shear stress and critical shear surfaces at limit equilibrium and at failure

Source : Janbu (1973)

2.1.4 Method of Slices

The method of slices based on the limit equilibrium method, is analyzed by dividing the sliding mass into a number of vertical slices, **n**. The base of each slice is assumed to be a straight line. For any slice, the inclination of the base to the horizontal is α, the width of slice is **b** and the average height is **h**, as shown in fig. 2.6. The factor of safety is taken to be the same for each slice. It is implied that the inter-slice forces must be the mutual force between the slices.

The forces acting on a slice are :

- The total weight of slice, W
- The normal force on the base, N
- The shear force on the base, T
- The inter-slice normal force, E
- The inter-slice shear force, X

The shear forces on the base must be in equilibrium on each slice as well as on the overall sliding mass.

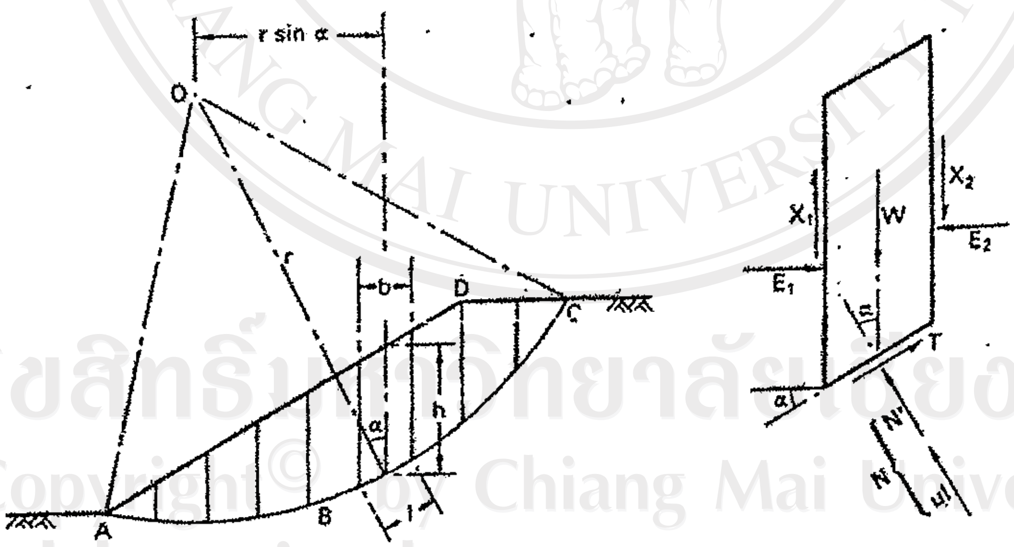


Fig. 2.6 Outline of method of slices

Source : Craig (1997)

Table 2.1 Equation and unknown associated with the method of slices

Source : reproduced from Espinoza et al. (1994)

Equations	Condition
n	Moment equilibrium for each slice
2n	Forces equilibrium in 2 directions for each slice
n	Mohr-Coulomb failure criterion
4n	Total number of equations

Unknowns	Description
1	Factor of safety
n	Nomal force at the base plane, N
n	Location of normal force at the base plane
n	Shear force at the base plane, T
n-1	Inter-slice normal force, E
n-1	Inter-slice shear force, X
n-1	Location of interslice force (line of thrust)
6n-2	Total number of unknowns

Based on the limit equilibrium, there are only ***4n*** equations as listed in table 2.1 while there are ***6n-2*** unknowns for sliding mass divided into n slices. Thus for ***n***>1, the solution is statically indeterminate and the additional assumptions are required to make the problem determinate. One of the common assumptions is the normal forces act at the midpoint of the base of slices, thus the number of unknowns reduces to ***5n-2***. The different types of assumptions about the inter-slice forces were made and led to various methods of slices. Table 2.2 lists the common methods of slices, their assumptions about the inter-slice forces and the conditions of static equilibrium.

Table 2.2 Various assumptions in method of slices and static equilibrium condition

Source : applied from Nash (1987)

Method of slices	Assumptions about interslice force	Failure surface	Force equilibrium		Moment equilibrium
			X	Y	
Ordinary (Fellenius)	Resultant parallel to base of each slice	Circular	No	No	Yes
Simplified Bishop	Horizontal	Circular	No	Yes	Yes
Simplified Janbu	Horizontal	Any shape	Yes	Yes	No
Lowe and Karafiath	Define inclination	Circular	Yes	Yes	No
Spencer	parallel for each slice	Any shape	Yes	Yes	Yes
Morgenstern and Price	$X/E = \lambda.f(x)$	Any shape	Yes	Yes	Yes
Janbu rigorous	Define thrust line	Any shape	Yes	Yes	Yes
Fredlund and Krahn GLE	$X/E = \lambda.f(x)$	Any shape	Yes	Yes	Yes

Note E and X are horizontal and vertical components of inter-slice forces respectively.

a) Ordinary method of slices

Ordinary method of slices is sometimes called Swedish method or the conventional method or Fellenius method (Fredlund & Krahn, 1977). This method is developed from the study of failure condition of railways in Sweden in 1920 by Fellenius. Fellenius (1936) assumed the resultant of the inter-slice force is zero (the resultant of inter-slice forces is parallel to its base) for each slice and used the moment equilibrium of all sliding mass. For the forces acting on the slice in fig. 2.6,

The equilibrium normal to the base is,

$$N' = W\cos\alpha - ul \qquad (2.5)$$

The moment equilibrium for entire sliding mass is,

Disturbing moment = Resisting moment

$$\sum WR\sin\alpha = \sum(\tau_m l).R \qquad (2.6)$$

$$\sum WR\sin\alpha = \frac{R}{F}\sum(c'l + N'\tan\phi') \qquad (2.7)$$

$$F = \frac{c'L + \tan\phi'.\sum N'}{\sum W \sin\alpha} \tag{2.8}$$

Then,

$$F = \frac{c'L + \tan\phi'.\sum(W\cos\alpha - ul)}{\sum W \sin\alpha} \tag{2.9}$$

Equation 2.9 is easily solved by hand calculation. However, the false assumption about the inter-slice forces gives the results in error when compared with more accurate method. These errors are maybe as large as 60% (Whitman and Bailey, 1967).

b) Simplified Bishop Method

Bishop (1955) presented the method for analysis of circular slip surface. Unlike Fellenius' solution, the method does not entirely ignore inter-slice force. It is only assumed that the inter-slice shear forces are zero. This method satisfies the vertical force equilibrium for each slice and the overall moment equilibrium about the center of the circular slip surface (fig. 2.7).

For resolving the vertical force equilibrium,

$$W = N'\cos\alpha - ul.\cos\alpha - T\sin\alpha \tag{2.10}$$

$$N' = \frac{[W - ul.\cos\alpha - \frac{c'}{F}l.\sin\alpha)]}{\cos\alpha(1 + \tan\alpha.\frac{\tan\phi'}{F})} \tag{2.11}$$

The safety factor derived from the overall moment equilibrium is ,

$$\sum WR\sin\alpha = \sum(\tau_m l).R \tag{2.12}$$

$$F = \frac{\sum(c'l + N'.\tan\phi')}{\sum W\sin\alpha} \tag{2.13}$$

From equation 2.11 and 2.13, the factor of safety can be derived as,

$$F = \frac{1}{\sum W\sin\alpha}\sum\left[\frac{\{c'l.\cos\alpha + (W - ul.\cos\alpha)\tan\phi'\}}{\cos\alpha(1 + \tan\alpha.\frac{\tan\phi'}{F})}\right] \tag{2.14}$$

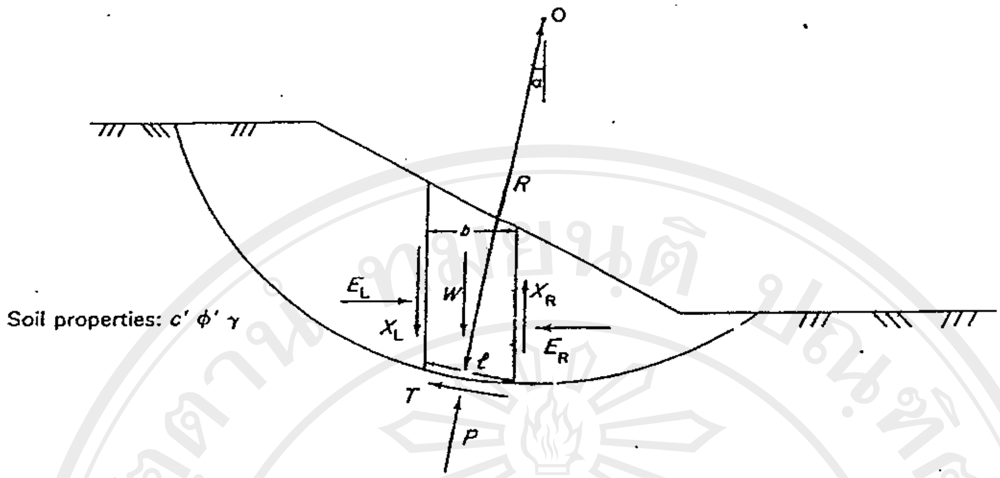


Fig. 2.7 Simplified Bishop method of slices

Source : Nash (1987)

As the equation 2.14 contains F on both sides, it has to be solved iteratively (trial and error). Since the convergence is usually quick, it can be solves by hand calculation although it is time consuming.

c) Simplified Janbu Method

This method was proposed by Janbu (1955, 1973) which are developed from the basis of Bishop's simplified method to suit for any shape of failure surface. It is assumed that a failure surface is non-circular and the inter-slice forces to be horizontal which means there is no inter-slice shear forces. Janbu method utilizes the force equilibrium of each slice and the horizontal force equilibrium of the entire sliding mass to derive a solution.

For force equilibrium in each slice,

$$N = W\cos\alpha, \qquad T = W\sin\alpha$$

Since

$$T = \frac{1}{F}(c'l + (N'-u)l.\tan\phi') = \frac{\tau l}{F} \tag{2.15}$$

$$\tau = \frac{\left(c' + (\frac{W}{b} - u).\tan\phi'\right)}{\left(1 + \tan\alpha.\frac{\tan\phi'}{F}\right)} \tag{2.16}$$

For horizontal force equilibrium of the entire sliding mass,

$$\sum T\cos\alpha = \sum N\sin\alpha$$

$$F = \frac{\sum \tau . l . \cos\alpha}{\sum W . \sin\alpha . \cos\alpha} = \frac{\sum \tau . b . \sec^2\alpha}{\sum W . \tan\alpha} \qquad (2.17)$$

From equation 2.15, 2.16 and 2.17, the factor of safety becomes,

$$F = \frac{1}{\sum W\tan\alpha}\sum\left[\{c'b + (W - ub)\tan\phi'\}.\{\frac{\sec^2\alpha}{1+\tan\alpha\dfrac{\tan\phi'}{F}}\}\right] \qquad (2.18)$$

The procedure for the result of equation 2.18 is similar to that of the Simplified Bishop Method (Bishop, 1955). Janbu uses the correction factor f_0 as in the equation 2.19 to compensate the assumption on the negligence of the inter-slice shear forces (fig. 2.8).

$$F_f = f_0 . F \qquad (2.19)$$

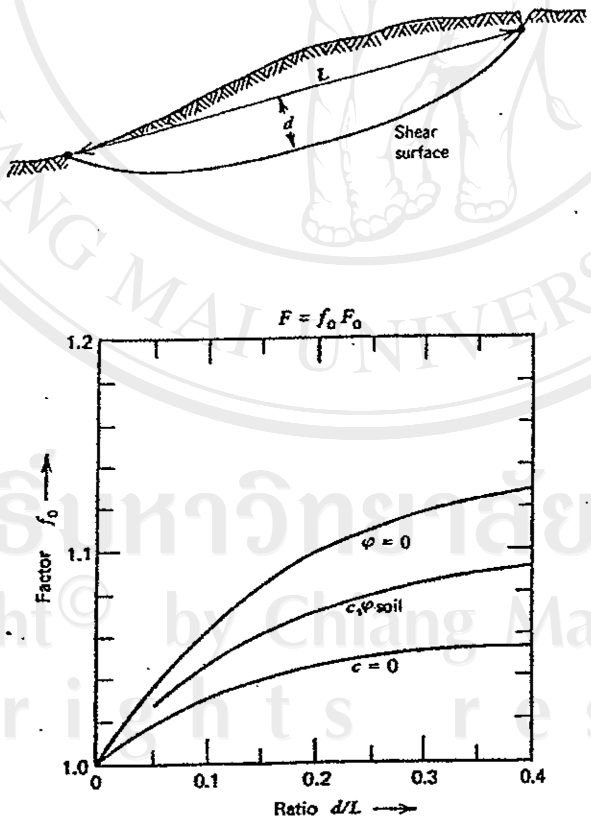


Fig. 2.8 Correction factor f_0 as function of curvature ratio d/L and type of soil

Source : Janbu (1973)

d) Spencer Method

Spencer's method (Spencer, 1967, 1973) is one of the rigorous limit equilibrium methods and satisfies static equilibrium by assuming that the direction of the inter-slice force is parallel for each slice. These ***n-1*** assumptions reduce the number of unknowns to ***4n-1***, but by an additive unknown factor, the number of unknowns equal to the required ***4n*** equations.

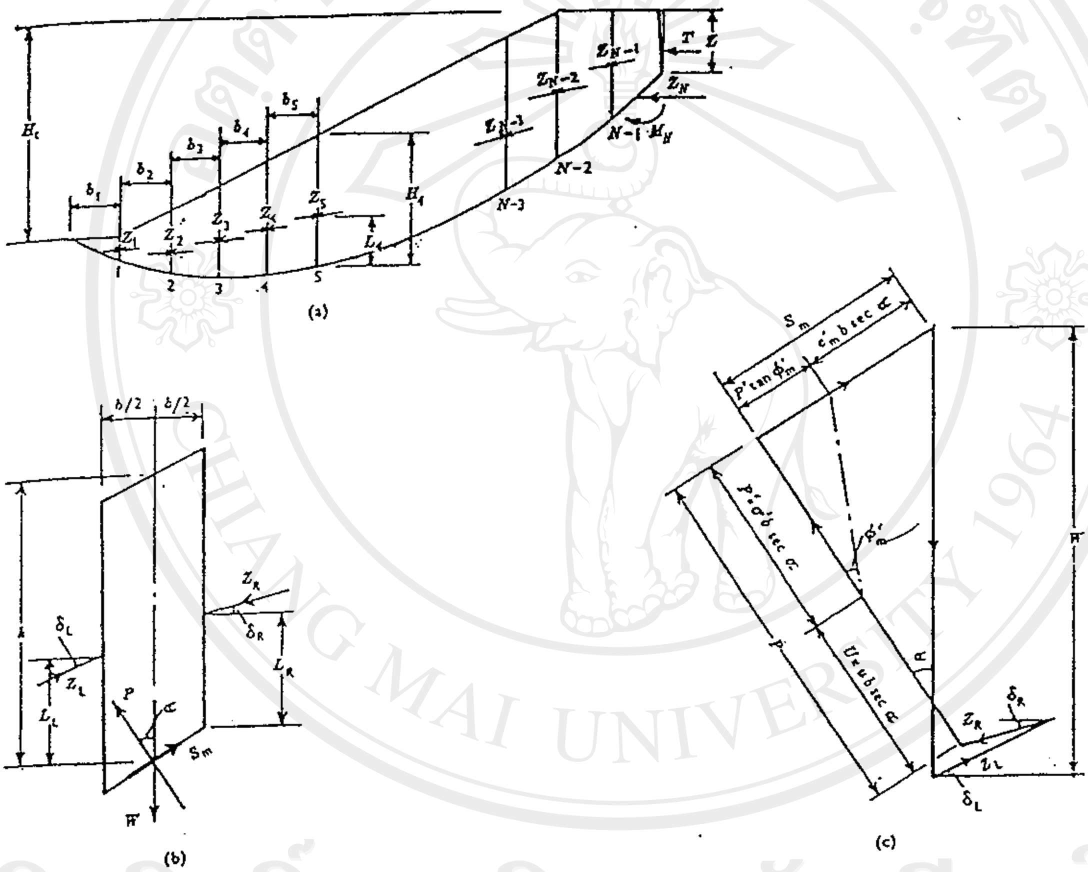


Fig. 2.9 Spencer method : (a) Cross-section through embankment,

(b) forces on typical slice, and (c) force diagram

Source : Spencer (1973)

Fig. 2.9 shows cross-section, forces on each slice and force diagram. The inter-slice resultant forces, Z_L and Z_R are inclined at δ_L and δ_R on the left and right sides of each slice.

For force equilibrium gives,

$$Z_R = \frac{\frac{c'}{F}b.\sec\alpha - W\sin\alpha + \frac{\tan\phi'}{F}(W\cos\alpha - ub.\sec\alpha)}{\cos(\alpha-\delta_R)\left[1+\frac{\tan\phi'}{F}.\tan(\alpha-\delta_R)\right]} + Z_L\left\{\frac{\cos(\alpha-\delta_L)\left[1+\frac{\tan\phi'}{F}.\tan(\alpha-\delta_L)\right]}{\cos(\alpha-\delta_R)\left[1+\frac{\tan\phi'}{F}.\tan(\alpha-\delta_R)\right]}\right\} \quad (2.20)$$

For moment equilibrium gives,

$$M_N = \gamma_w\frac{z^2}{2}\left[\frac{b_N}{2}\tan\alpha_N + \frac{z}{3}\right] - \sum_{i=1}^{N}[J] \quad (2.21)$$

$$J = \frac{1}{2}Z_{i-1}\left[\sin\delta_{i-1}(b_i + b_{i-1}) - \cos\delta_{i-1}(b_i\tan\alpha_i + b_{i-1}\tan\alpha_{i-1})\right] \quad (2.22)$$

As the problem is statically indeterminate, Spencer method solved by using an additive unknown value (k) as,

$$\tan\delta = k.\tan\theta \quad (2.23)$$

where δ is the inclination angle of inter-slice force and θ is a scaling factor. Since the assumption on the inter-slice force angles is parallel for each slice, thus k is constant throughout the slope.

To satisfy force and moment equilibrium conditions (equation 2.20, 2.21 and 2.22), 2 variables; F and θ are required and used in force and moment equilibrium equations. For trial the values of these 2 variables, the convergence procedure of Spencer method is illustrated in fig. 2.10.

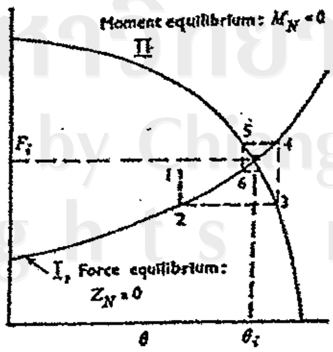


Fig. 2.10 Converge procedure of Spencer's Method

Source : Spencer (1973)

e) Morgenstern and Price method

Morgenstern and Price (1965) published a rigorous method of analysis for any shape slip surfaces by considering both force and moment equilibrium. It is similar to Spencer's method, but differs in the relationship between the inter-slice normal and shear forces, which assumes in the form of arbitrary function

$$f(x) = \frac{1}{\lambda} \cdot \frac{X}{E} \tag{2.24}$$

and λ is an addition unknown, whereas the function of Spencer's method is constant as shown in fig. 2.11.

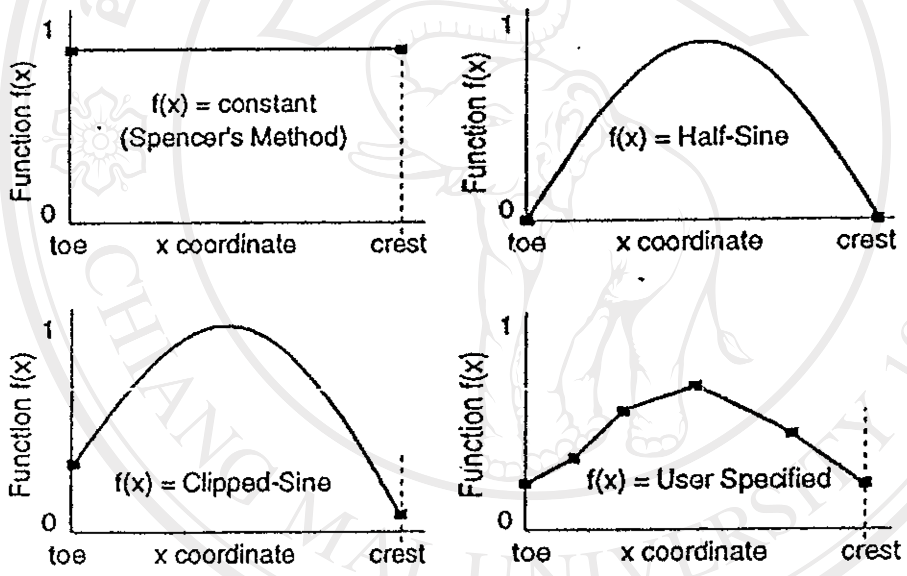


Fig. 2.11 Examples of functions described the variation of inter-slice force angles

Source : Abramson et al. (1996)

This function can compare with a similar expression by Spencer (1973).

$$\tan\delta = k.\tan\theta$$

, where $\tan\theta$ in this expression equals λ, k = f(x) and $\tan\delta$ = X/E.

f) Slice spring method

Kondo et al. (1999) proposed a new slope stability analysis method based on method of slices by using springs attached to the inter-slice planes which is call Slice Spring Method: SSM. Slice spring method assumes the soil mass to be elasto-plastic material and utilizes springs and sliders to represent the characteristic of elastic and plastic of soil as shown in fig. 2.12.

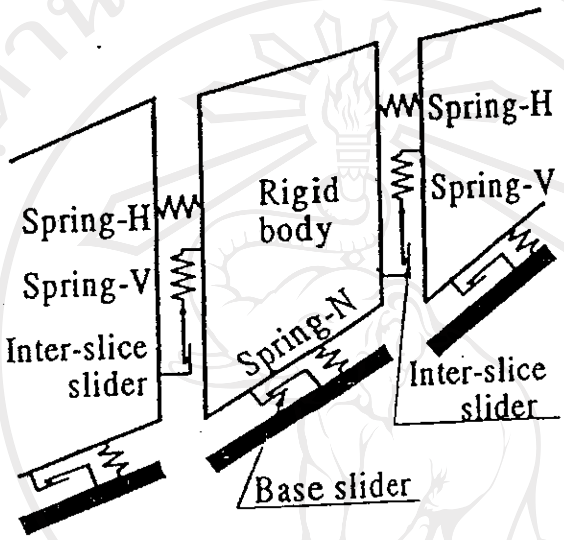


Fig. 2.12 Model of Slice-spring method

Source : Kondo et al. (2001)

SSM satisfies force and moment equilibrium conditions for each slice and the whole slope. This method also satisfies the stress-strain relationship of soil and determines the inclination angle of the inter-slice forces by using the displacement of slices caused by the action of the spring (Kondo et al., 2001). As the displacement of slices can be determined, the relative vertical displacement (Vi) between the adjacent slices can be known.

Based on the stress-strain relationship of spring-V, the virtual shear forces Z_{DVi} along the inter-slice plane can be obtained from the relative vertical displacement Vi. On the other hand, horizontal inter-slice force Z_{Hi} is determined by the limit equilibrium condition. The virtual inclination angle of the inter-slice force δ_{Di} is defined as equation 2.25,

$$\tan\delta_{Di} = Z_{DVi} / Z_{Hi} \qquad (2..25)$$

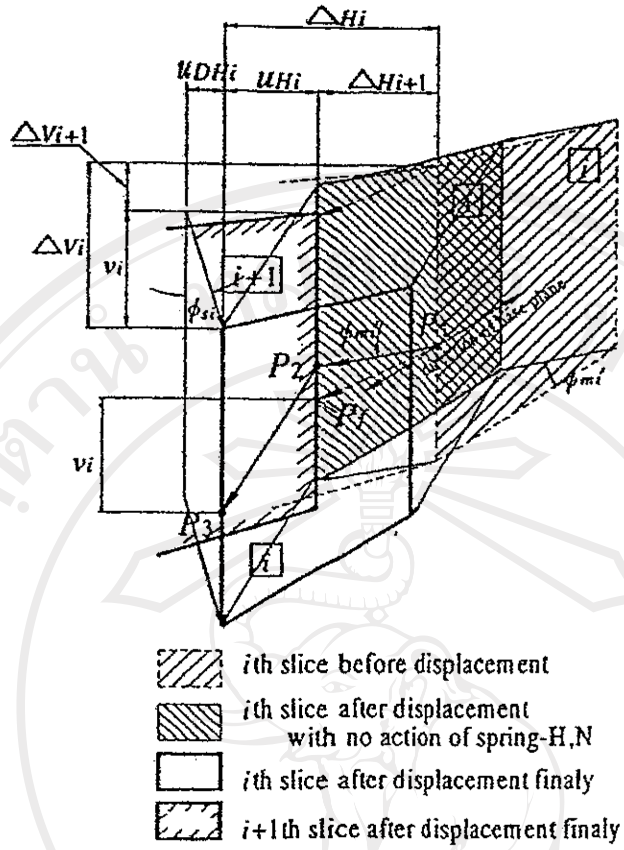


Fig. 2.13 Slice displacement in the Slice-Spring Method

Source : Kondo et al. (2001)

Accordingly, to convert the virtual inclination angle to the angle which satisfies the limit equilibrium condition by an iterative calculation using k (scaling factor).

$$\tan\frac{\pi/2-\delta_i}{2}=k.\tan\frac{\pi/2-\delta_{Di}}{2} \qquad (2.26)$$

The way how to evaluate the safety factor is basically based on the Spencer Method. The SSM can be applied to soil failure problems; earth pressure, bearing capacity, and slope stability, even on a non-uniform ground surface.

2.1.5 Method of Discrete Element

a) Finite Element Method (FEM)

Generally, methods of slices based on limit equilibrium conditions are commonly used in slope stability problems, because of its simplicity. However, such method needs some assumptions to solve the indeterminate problem. As the computer technology has been developed, finite element method (FEM) is leaded to analyze the slope stability problems.

FEM was first introduced to geotechnical engineering by Clough and Woodward, and was further studied and applied by many researchers (Duncan, 1996).

FEM is the numerical method using discrete element which is based on the principle of limit analysis. FEM divides the soil continuum into discrete units (finite element) as shown in fig. 2.14. These elements are connected at their nodes and boundaries of the continuum (Abramson et.al., 1996). Some soil variables and properties are input distributive to each element in the forms of mathematics formula. The stiffness matrix is computed for each element using the virtual work theory.

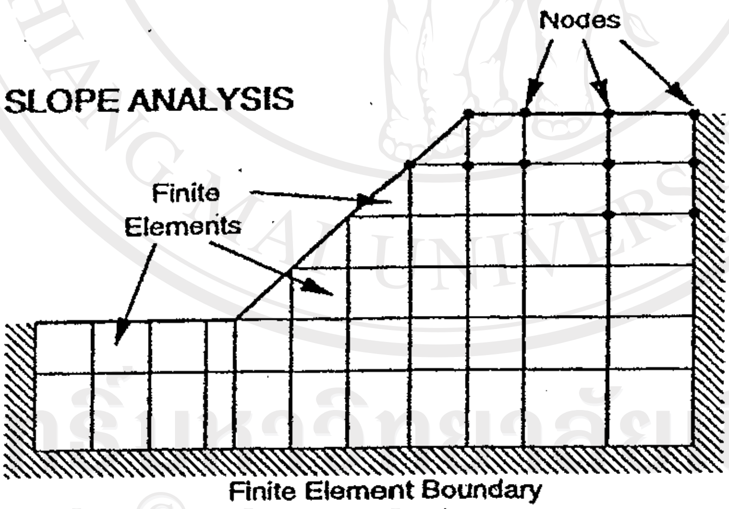


Fig. 2.14 Definitions of terms used for Finite Element Method

Source : Abramson (1996)

For geo-technical applications, FEM is used to analyze the relationship of displacement, stresses and strains at any point in soil mass.

b) Rigid Bodies-Spring Method (RBSM)

Rigid bodies-spring method (RBSM) is proposed by Kawai (1977). The principles of RBSM are based on discrete element and limit analysis, which are similar to the finite element method (FEM). RBSM is sometimes called rigid finite element method (RFEM) by Zhang (1999).

In the RBSM, a slope is discretized by a number of small rigid bodies with arbitrary shape, commonly used triangular shape. The rigid bodies are connected by a group of springs distributed over the contact area of adjacent bodies according to fig. 2.15. These springs represent the relative normal displacement, relative shear displacement and rotational displacement (Takeuchi and Kawai, 1988).

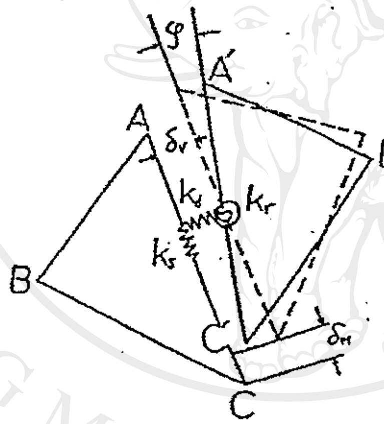


Fig. 2.15 Model of rigid bodies-spring method (RBSM)

Source : Kawai (1977)

2.2 Literature review

2.2.1 Comparison of slope stability analysis methods

From the past until the present, various slope stability analysis methods have been developed. The geotechnical engineers would like to know the best method that is suitable for each condition. Then, the comparisons of these methods have been studied.

Whitman and Bailey (1967) studied the results of analysis of Ordinary Method of slice, Simplified Bishop method and compared with Morgenstern and Price method which is the most reliable method in that time. The results of the analysis of Bishop's simplified method show 7% or less error of the values of safety factor. And those of Ordinary method of slice show a great error (40%) of the factor of safety, although the safety factor is still in the safety level. The error will be increase, sometimes 60% in the case with pore water pressure.

Fredlund and Krahn (1977) compared the various methods of slices commonly used for slope stability analysis in the terms of the normal force equation. This study is concerned with Fellenius method, Simplified Bishop method, Spencer's method, Janbu's simplified method, Janbu's rigorous method and Morgenstern-Price method. According to the comparison, all methods except Fellenius method, have the same form of the normal force equation but differ in the static equilibrium conditions satisfied for whole slope and the assumption to make the problem determinate.

Fig. 2.16 and table 2.3 illustrate the example problems and the comparison in some cases. Then, compared by plotting factor of safety F_m and F_f versus scaling factor λ that is called "best-fit regression graph" according to fig. 2.17.

From table 2.3 and figure 2.17, the factor of safety obtained by Spencer's method and Morgenstern-Price method are similar (i.e. the differences are not more than 0.4%) to that by the simplified Bishop method. The factors of safety obtained by various methods remain similar whether the failure surface is circular of composite. It can be concluded that the assumption of the inter-slice force has small influence on the factor of safety respected to moment equilibrium. In contrast, the factor of safety F_f satisfied force equilibrium is very sensitive to λ.

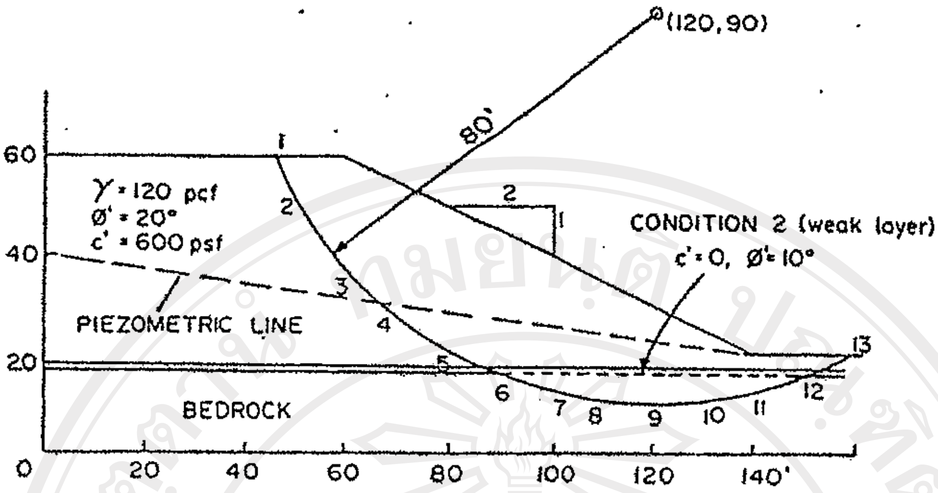


Fig. 2.16 Example problem

Source : Fredlund and Krahn (1977)

Table 2.3 Comparison of factors of safety for example problems

Source : Fredlund and Krahn (1977)

Case no.	Example problem*	Ordinary method	Simplified Bishop method	Spencer's method			Janbu's simplified method	Janbu's rigorous method**	Morgenstern–Price method $f(x)$ = constant	
				F	θ	λ			F	λ
1	Simple 2:1 slope, 40 ft (12 m) high, $\phi' = 20°$, $c' = 600$ psf (29 kPa)	1.928	2.080	2.073	14.81	0.237	2.041	2.008	2.076	0.254
2	Same as 1 with a thin, weak layer with $\phi' = 10°$, $c' = 0$	1.288	1.377	1.373	10.49	0.185	1.448	1.432	1.378	0.159
3	Same as 1 except with $r_u = 0.25$	1.607	1.766	1.761	14.33	0.255	1.735	1.708	1.765	0.244
4	Same as 2 except with $r_u = 0.25$ for both materials	1.029	1.124	1.118	7.93	0.139	1.191	1.162	1.124	0.116
5	Same as 1 except with a piezometric line	1.693	1.834	1.830	13.87	0.247	1.827	1.776	1.833	0.234
6	Same as 2 except with a piezometric line for both materials	1.171	1.248	1.245	6.88	0.121	1.333	1.298	1.250	0.097

*Width of slice is 0.5 ft (0.3 m) and the tolerance on the nonlinear solutions is 0.001.
**The line of thrust is assumed at 0.333.

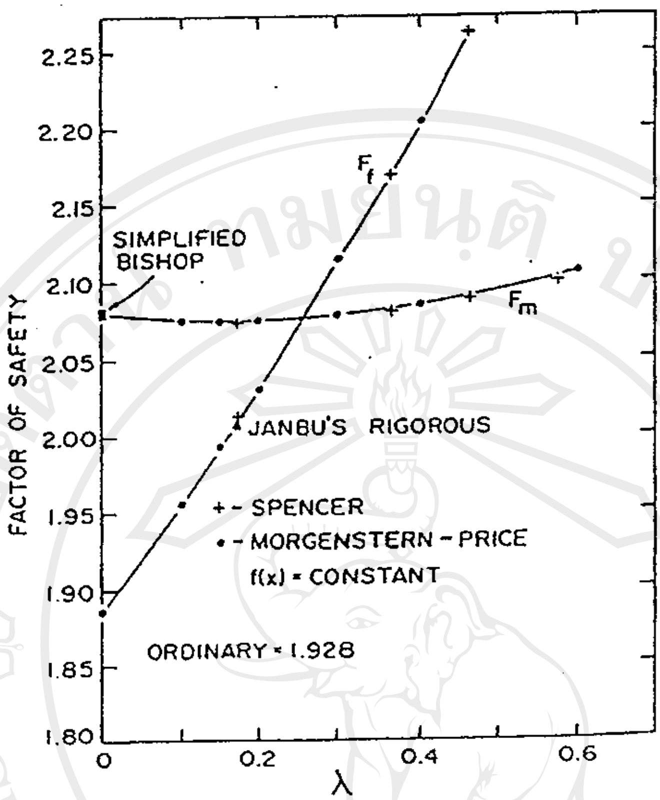


Fig. 2.17 Comparison of factor of safety by plotting versus λ

Source : Fredlund and Krahn (1977)

Cheng (1997) performed the back analysis of three actual slope failure cases in Singapore and Hong Kong: homogeneous fill slope, layered soil slope and in-situ soil slope. The characteristics and comparison of 6 slope stability analysis methods are shown in table 2.4. According the result of this study, it is found that Bishop's simplified method is the most suitable method for homogeneous soil. In the case of layered soil slope, the most conservative method is Janbu's simplified method, and the wedge method is most suitable for soil slope with weak layer. However, the factors of safety obtained from these methods in all 3 cases are in the level that can be admitted.

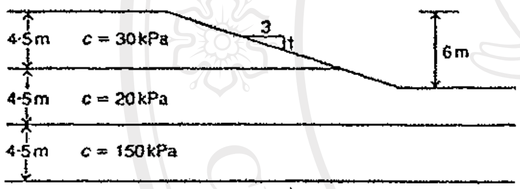
Table 2.4 Comparison and characteristics of various slope stability analysis methods

Source : Cheng (1997)

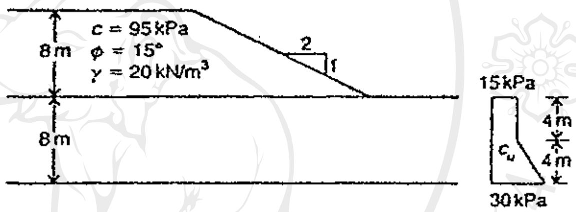
Method	Geographical Conditions			Preferred Shape of Failure Surface	No. of Wedges/Slices	Speed Rank	Recommendations
	Homogenous	Layered	Hard or weak layer encountered (ie slip surface predefined)				
Wedge	Suitable - interface cohesion ≯ 50 %	Suitable -interface cohesion = 0 %	Most Suitable - interface cohesion = 100%	any shape	Usually ≯ 8	2	- Can be used in most kinds of problems with 0% interface cohesion and number of wedges not greater than 8 - Particular suitable where there is a weak stratum or where the slope rests upon a very strong stratum. - Can be used in rock slope problem. - Fast to converge. - Multi value of FOS will be obtained. Only the greatest FOS can be used
Janbu's Rigorous	Suitable - slightly overestimated	Suitable	Suitable -slightly under estimate	any shape	Usually ≮ 10	4	- Very useful for the majority of soil slope in Hong Kong
Janbu's Simplified	Suitable - underestimated	Suitable - most conservative	Suitable -slightly under estimate	any shape	Usually ≮ 10	5	- Highly affected by fo factor. Published fo factor are for homogenous materials. Great care should be made when using this method in non-homogenous soil. - Factor of safety usually underestimated. - Large number of iteration required.
Bishop's Simplified	Most suitable	Suitable	Suitable -slightly under estimate	circular	Usually ≮ 10	1	- Most useful where circular failure surface can be assumed
Spencer's	Suitable - slightly underestimated	Suitable	Suitable -slightly under estimate	any shape	Usually ≮ 10	6	- Suitable in most problems. - Take long time to obtain FOS(moment) = FOS(force)
Sarma's	Suitable - slightly underestimated	Suitable - Slightly underestimated	Suitable -slightly under estimate	any shape	Usually ≮ 10	3	- If assessing the problem with dynamic loading modifying of equation is need. - No iteration required (ie faster)

Jiang and Magnan (1997) presented the application of limit analysis methods in geo-technical engineering. They also demonstrated some differences and similarities between limit analysis and limit equilibrium methods. The comparison of these two theories is carried out by several computations of stability of model slopes as shown in fig. 2.18. Based on the results according to table 2.5, it illustrated that the limit analysis methods gives the factor of safety close to the values obtained by methods of limit equilibrium. Thus, the limit analysis can be well applied to calculate stability of embankments. Another advantage of the limit analysis over the methods of slices is that the finite element solution of the limit analysis provides failure mechanisms that may not be appropriately simulated by methods of slices.

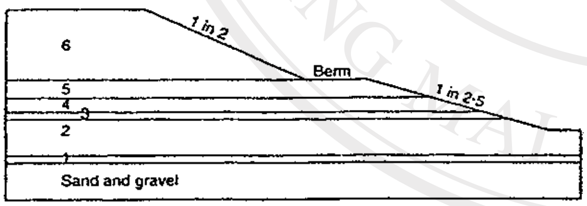
Example 1



Example 2



Example 3



Stratum No.	Soil	Thickness: m	c_u: kN/m²	γ: kN/m³
6	Fill	5·8	12·0	17·3
5	Brown silt	1·5	23·9	18·5
4	Grey silt	1·5	22·7	18·1
3	Upper Peat	0·6	33·5	11·0
2	Buttery Clay	4·3	22·7	16·0
1	Lower Peat	0·6	33·5	11·0

Fig. 2.18 All example in the analysis

Source : Jiang and Magnan (1997)

Table 2.5 Comparison of the results of limit analysis and those of limit equilibrium analysis

Source : Jiang and Magnan (1997)

Case	Method of limit equilibrium	Author	F_s^1 from limit equilibrium	F_s^2 from limit analysis	$\frac{F_s^1 - F_s^2}{F_s^2}$
Example 1	Semi-analytical Fellenius and Bishop	Low Low	1·45 1·44	1·48	−2·0% −2·7%
Example 2, with $\phi = 15°$ embankment	Semi-analytical Fellenius Bishop	Low Low Low	1·38 1·36 1·14	1·25	10·4% 8·8% −8·8%
Example 2, with $\phi = 0°$ embankment	Fellenius and Bishop	Low	1·31	1·25	4·8%
Example 3, with 6·1 m berm	Slip circle Morgenstern & Price Bishop	Skempton Chen Chen	1·0 0·864 0·899	0·905	10·5% −4·5% −0·7%
Example 3, with 12·2 m berm	Slip circle Morgenstern & Price Bishop	Skempton Chen Chen	1·2 0·974 1·068	1·007	19·2% −3·3% 6·1%

Kondo (1996) proposed the method of deriving the overall safety factor using Rigid bodies-spring method (RBSM) and applied RBSM to analyze the stability of landslide slopes considered pore water pressure. From this study, the method of analysis of slopes concerning pore water pressure was proposed, and comparing the overall safety factor with limit equilibrium methods. All 5-example cases of slope, the results show the overall safety factor of RBSM is almost close with that of Spencer method. The differences from Bishop method, Janbu method and Fellenius method are 1%, 2% and 7% respectively.

Kondo (2000) proposed the "Slope Stability Analysis using Spring attached to the inter-slice planes which is call "Slice Spring Method" and evaluated the proposed method on the typical soil failure problems. In the case of bearing capacity problem, it is shown that for the example of Nq=30°, the coefficient of bearing capacity obtained from Slice spring method (19.6) agrees very well with the correct value (18.4). But Spencer method (9.5), Janbu method (13.0), Bishop method (31.1) and Fellenius method (5.6) result in much error when applied to problems of bearing capacity. And in the case of slope with the anchoring force, it is shown that the difference of the factor of safety between the Slice spring method and Spencer method when attached a big value of anchoring force is larger than the case of no anchoring force.

2.2.2 Classification of slope stability analysis methods

A number of methods of slices have been proposed for the stability analysis of slopes. These methods differ in the addition 2n-2 assumptions to solve the problems determinate. Espinoza et al. (1992) classified various methods of slices into three categories describing the hypotheses made on the inter-slice forces:

Hypothesis I : the direction of the inter-slice force resultant (Bishop, 1955; Morgenstern and Price, 1965; Spencer, 1967)

Hypothesis II : the height of the line of thrust (Janbu, 1954)

Hypothesis III : the shape of the inter-slice force distribution (Sarma, 1973; Correia 1989)

Espinoza et al. (1992) studied the influence of the inter-slice shear forces to the factor of safety. Several case examples are determined. It is found that for circular slip surfaces without external forces either homogeneous or non-homogeneous soils, the different assumptions on the inter-slice forces give the minimal variation in the value of factor of safety. This conclusion is in agreement with Fredlund and Krahn (1977) who used λ functional relationship in comparisons. Nevertheless, the simplified Bishop method gives the most acceptable results in such conditions.

For non-circular failure surfaces, the different hypotheses give the large variation (10-34%) of safety factor. It can be concluded that for such conditions, the shape of slip surfaces and the hypothesis of inter-slice forces pay a significant influence on the value of safety factor (Espinoza et al., 1994).

Chowdhury (1988) divided the methods of slope stability analysis into two major categories: Deterministic and Probabilistic. Deterministic methods include limit equilibrium methods and finite element methods. The conventional deterministic approach is concerned with the calculation of the magnitude of the factor of safety F under given conditions, where the factor of safety F is defined as the ratio of available shear strength along a slip surface to the shear strength required for critical equilibrium. Probabilistic method is the risk assessment of failure that is not calculated as such, but used the common sense of the relationship between the slope reliability and the value of factor of safety. The slope reliability increases while the value of factor of safety increases and the reliability decreases while the value of factor of safety decreases. In such method, the factor of safety is a function of several variables and constant,

$$F = f(c, \phi, \gamma, u, H, \beta) \tag{2.27}$$

The mean of F and the standard deviation σ_D of the factor of safety can be calculated and a reliability index defines as follow.

$$RI = (F_{mean} - 1) / \sigma_D \tag{2.28}$$

2.2.3 Measuring the normal forces acting on the slip surface

In the way of slope stability analysis, the normal forces acting on the slip surface occupied one of the significant functions. Ritthisom et al. (2002) made an experiment of measuring the normal forces acting on the base plane by model tests. The experiment included 3 slope models, and each model is divided into three parts. One is the model base made of the plywood boards. Another is the slip plane made of the plywood boards covered by sand paper. The other is moving mass made of the straw filled with sand inside formed to be a slope. The experimentation of this study is done in two cases: normal slope and slope attached the anchoring force under loading conditions. The values of normal forces acting on the slip surface are collected by 20 load cell sensors which were set under the sliding mass. Figure 2.19 and 2.20 illustrate the model cross-section and outline of the experimentation.

Base on the results, it is found that the normal forces acting on the slip surface of every slices change when the surcharge is gained. The normal force of slice 4 increases in the highest rate because of position of the surcharge. And the normal force of the other slices increase in the similar rate. From this study, it is implied that there are the mutual inter-slice forces between the slices.

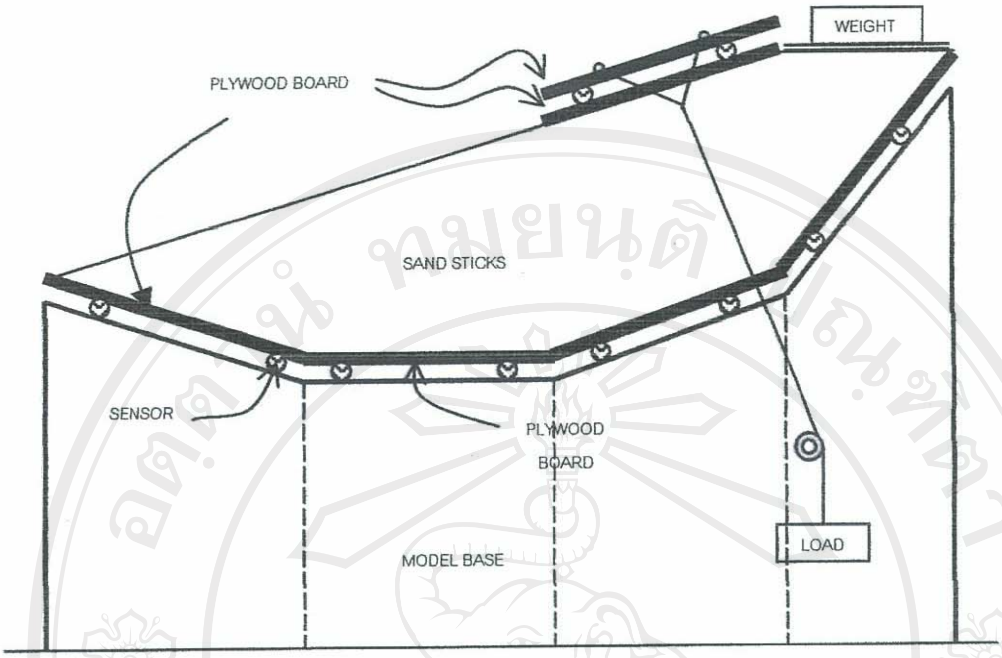


Fig. 2.19 Cross-section of model slope

Source : Ritthisom et al. (2002)

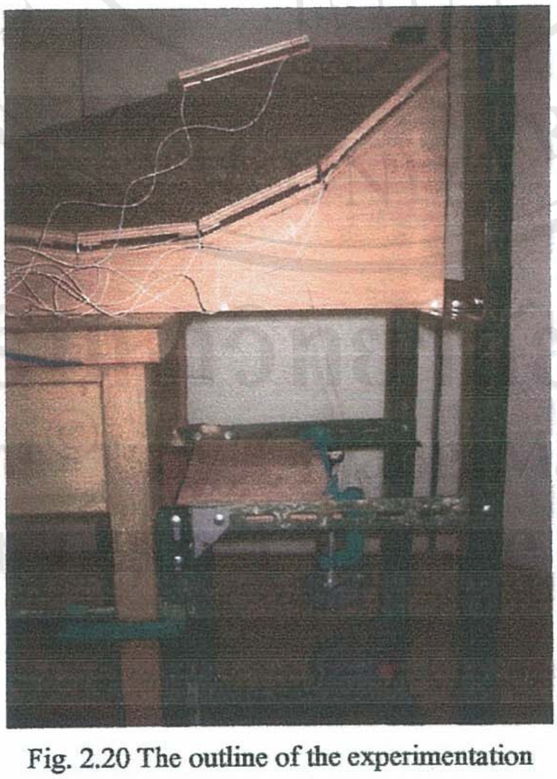


Fig. 2.20 The outline of the experimentation

Source : Ritthisom et al. (2002)

2.2.4 Physiography, Geology and slope failure in Northern Thailand

The northern part of Thailand is full of mountain and upland areas, consisted of high terraces, low plateaus, hills and mountain ranges alternated by river basin plains. The main rivers in this area are Ping, Wang, Yom and Nan rivers. Most of the mountain ranges are oriented at the north-south direction.

The basic igneous rocks are widely distributed throughout this region. The major parent rocks and rock outcrops are granite, gneiss, granodiorite, andesite, schist and shale (Ruenkrairergsa, 1978). Because of the weather and climate of the region, the degree of weathering of this area is so high. Thus, many parent rocks have been transformed into decomposed materials, especially in the upper part of the ground. According to fig. 2.21, it can be seen that decomposed granite is widely distributed in the area of Northern Thailand.

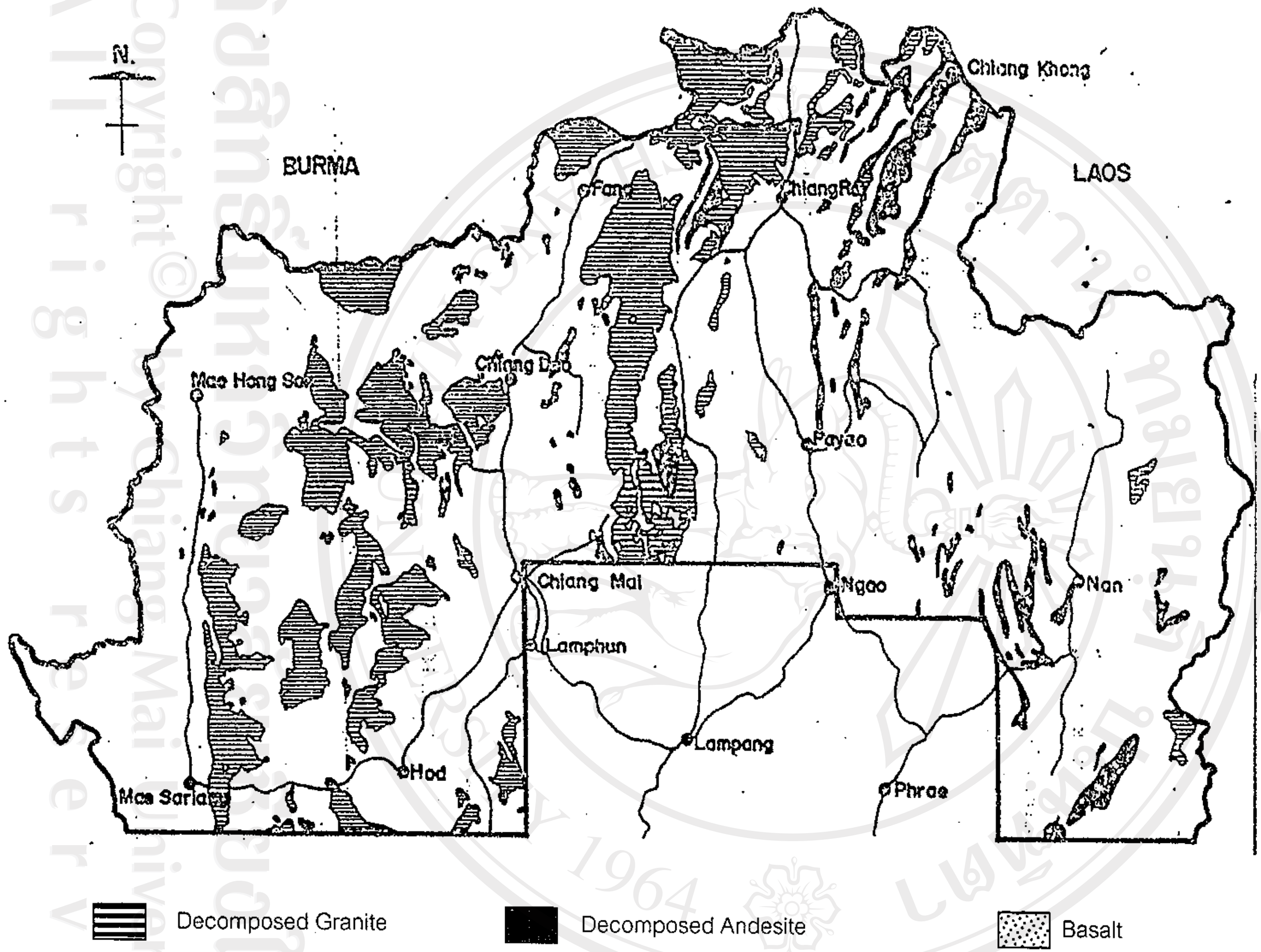
The main component minerals of granite are quartz and feldspar associated with biotite and hernblende. In the process of chemical weathering, quartz is hard enough to endure with this weathering, but feldspar has been transformed into kaolinite which is clay minerals with relatively low plasticity. After weathering process, the products are mainly quartz and kaolinite clay. The color varies from white to light brown, and the thickness of weathering layer varies from 8-12 meters depending on the mountain height and other environmental factors (Ruenkrairergsa, 1993).

Northern Thailand is affected by many cycles of orogenic events in the historic time. A lot of fault zones and weak planes exist all over the area. These weak planes and low plasticity granatic soils which are easily weathered reduce the stability of slope and lead to the occurrence of slides, especially in rainy season.

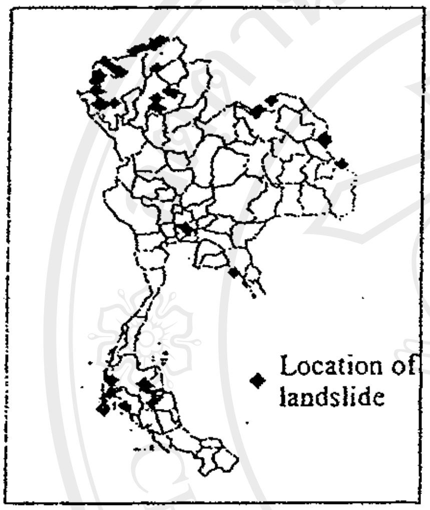
Yamsai and Mairaing (2000) collected landslide data from site visits and previously reports according to geological conditions. All 119 landslides have been recorded and their locations are illustrated in fig. 2.22. A number of landslides for each region and for type of work are also shown.

It can be noticed that landslides more than half are caused by slope cutting and the rate of landslides in the northern part is 54.6% of all landslides in Thailand. The geological formation is mostly residual soil and weathered rocks of granite, shale, phyllite, sandstone and

Fig. 2.21 Distribution of decomposed granite, andesite and basalt in Northern Thailand (Source : applied from Ruenkrairergsa ,1978)



quartzite which have high water absorbability. Type of slope failures are shallow seated rotational slides which has slope slide failure mode. The location of landslides in Northern Thailand is almost similar to the distribution area of decomposed granite. Almost slope failures in this area are caused by slope cutting through the areas of granite mountains or low strength areas during heavy rainfalls. As the time is gone, the granite is rapidly weathered and the slope is failed finally.



Location of landslides in Thailand

Number of landslides in Thailand

Work Type	The divisions of Thailand					
	Center	North	East	Northeast	South	Total
Cut Slope		40			21	61
Excavation						
Bank Protection				13		13
Filling	2	8				10
Earth Dam		2				2
Natural Slope	-	11			18	29
Misc.		4			0	4
Total	2	65	0	13	39	119

Fig. 2.22 Location and number of investigated landslides

Source : Yamsai and Mairaing (2000)

Table 2.6 Varnes classification system

Source : Mairaing (2000)

TYPE OF MOVEMENT			TYPE OF MATERIAL		
			BEDROCK	DEBRIS (coarse soil and rocks)	EARTH (fine soil)
I	FALLS		rock fall	debris fall	earth fall
II	TOPPLES		rock topple	debris topple	earth topple
III	SLIDES	ROTATIONAL	rock slump	debris slump	earth slump
		TRANSLATIONAL	a. rock block slide b. rock slide	debris slide	earth slide
IV	SPREADS		rock spread		earth lateral spread
V	FLOWS		bedrock flow	a. debris flow b. debris avalanche c. block stream d. solifluction e. soil creep	a. wet sand flow b. rapid earth flow c. earth flow d. loess flow e. dry sand flow
VI	COMPLEX		combination of above movements		

The types of movement, which is classified by Varnes system (table 2.6) are also presented in the study. Earth slump is the type that has the most chance to occur, the next is debris flow, debris slide, earth slide and others. The slides which include earth slump, earth slide, debris slide and debris slump occurred at the rate of 58.8% in all landslides.