EVALUATION OF LANDSLIDE SENSITIVE AREAS FOR CUT SLOPE IN PHUKET

INTRODUCTION

General Introduction

Landslides have become one of the major natural disasters over the past few years in our country. It is the most common natural hazard and threatening condition for people in mountainous area. Even when it happens away from the inhabited area, landslide can be a significant hazard and has a serious economic impact by blocking roads and river (Akbar, 1998).

In Thailand, many groups of researcher studied about the landslide occurrences and have developed landslide susceptibility map. The landslide susceptibility map is used for a hazard management. In order to develop the map property, factors related to slope instability need to be studied. Slope instability processes are the product of local geomorphic, hydrologic, and geologic conditions; modification of these conditions by geodynamic processes, vegetation, land use practices, human activities and frequency and intensity of precipitation and seismicity (Soeters and Van Westen, 1996). Recently, Geographic Information System (GIS) application is a powerful analysis tool to handle spatial data. Since the landslide hazard zonation is very much related to spatial information e.g. topography, geology, land cover, rainfall etc, GIS can be effective in analyzing these factors at various locations of a given area (Rajbhandari, 1995). This research is focused on the process of combining engineering soil properties and weighting factor method by using GIS application. An important thing in evaluating the hazard associated with the failure of landslide induced by cut slope is the probability of failure.

The development on Phuket island is rapidly growth and requires more infrastructure such as transportation route, resort projects, residential and commercial buildings. Building those structures in mountain area can trigger landslide. Therefore, this study is also focused on the determination of sensitive area for cut slope in Phuket.

Statement of Problems

The stability of cut slope on mountainous area is a major concern to the developed area as well as for the safety of those staying in these areas. Any kind of slope failure may lead to disruption in traffic, socio-economic activities, loss of property, injuries or sometimes even deaths of humans and/or livestock, and environmental degradation. Moreover, humans trigger landslide by carelessly cutting a slope for construction, especially at the toe slope.

Therefore, an assessment of the stability conditions in mountainous area is quite important especially as granitic and mudstone soil is the most common soil found in Thailand and has the highest rate of landslide (Geotechnical Engineering Research and Development Center, 2006). Several techniques can be used to evaluate landslide potential area such as infinite slope analysis, weighting factor method and logistic regression method. The slope mass rating (SMR) technique has been found to be quite useful where it can be practiced, and is effectiveness in interpreting stability and recommending control measures. The technique is based on the well established rock mass rating (RMR) technique. The RMR and SMR technique has been used earlier in many mining and engineering projects related to tunnels and cut slope.

In order to improve the landslide susceptibility map by weighting factor method, it is necessary to improve the parameter to predict landslide such as engineering soil properties factor, RMR and SMR factors.

Objective of Research

The objectives of this study are:

1. Determine the sensitive areas of landslide and cut slope failure due to urban development in Phuket area by combination engineering soil properties factor into weighing factor method using GIS application.
2. Develop and verify landslide susceptibility caused by cut slope failure by using field investigation data.
3. To propose a method in calculating probability of cut slope failure and to combine into landslide hazard map by using field investigation data.

Scope of Research

1. Study area located in Phuket province.
2. Engineering parameters of slope material were determined by rock mass classification method and used the analyzed data from previous study.
3. GIS application was used for data analysis.

LITERATURE REVIEW

Landslides

Varnes (1978) defined term Landslide “the movement of a mass of rock, debris or earth down a slope”. The criteria used in classification of landslides presented in emphasizing type of movement and type of material. The names for the type of materials are rock, debris, and earth. The movement has been divided into fall, topples, slides, spreads, and flows, as shown in Fig 1. This scheme considers fall, slides, and flows in bedrock, soils and unconsolidated deposits. The moisture content increases from rockfall to debris flow, and ultimately, a very wet debris flow grades into a very turbid stream.

A landslide is the mass movement, usually sudden, of soil and debris down a steep slope. Landslides can be triggered by heavy rainfall, earthquake or undercutting of the base of slopes by river (Ian Davis and Gupta, 1989).

The term landslide is defined as outward and downward movement of mass, consisting of rock and soil due to natural or manmade factors. High intensity rainfall triggers many landslides (Fauziah, 2004).

The processes involved in slope movements comprise a continuous series of events from cause to effect. Varnes (1978) provided a list of the causes of slides follows Varnes's distinction that the three broad types of landslide processes are which that increase shear stresses, contribute to low strength, and reduce material strength.

Varnes (1978) classified landslides according to the type of movement undergone on the one hand and the type of materials involved on the other (Fig 1). Types of movement were grouped into falls, slides and flows. The materials concerned were simply grouped as rocks and soils. Obviously, one type of slope failure may grade into another; for example, slides often turn into flows. Complex slope movements are those in which there is a combination of two or more principal types of movement. Multiple movements are those in which repeated failures of the same type occur in succession, and compound movements are those in which the failure surface is formed of a combination of curved and planar sections.

Falls are very common. The moving mass travels mostly through the air by free fall, saltation or rolling, with little or no interaction between the moving fragments. Movements are very rapid and may not be preceded by minor movements. A rockfall event involves a single block or group of blocks that become detached from a rock face; each block may be a falling block behaving more or less independently of other blocks. Blocks may be broken during the fall. There is temporary loss of ground contact and high acceleration during the descent, with blocks attaining significant kinetic energy. Blocks accumulate at the bottom of a slope as scree deposit. If a rockfall is active or very recent, then the slope from which it was derived is scarped. Frost thaw action is one of the major causes of rockfall.

Toppling failure is a special type of rockfall, which can involve considerable volumes of rock. The danger of slope toppling increases with increasing discontinuity angle, and steep slopes in vertically jointed rocks frequently exhibit signs of toppling failure.

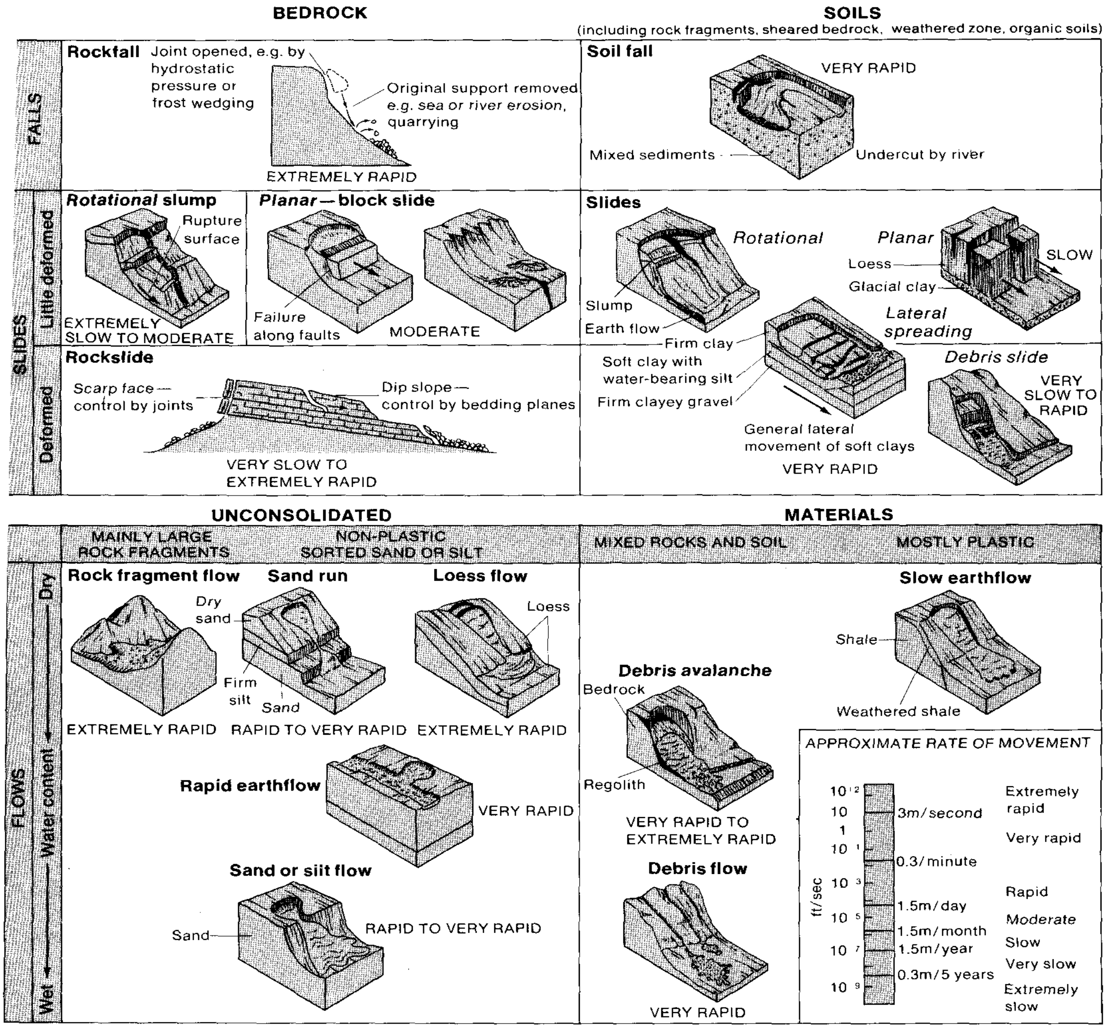

Figure 1 Landslide type
Source: Varnes (1978)

In slides, the movement results from shear failure along one or several surfaces, such surfaces offering the least resistance to movement. The mass involved may or may not experience considerable deformation. One of the most common types of slide occurs in clay soils, where the slip surface is approximately spoon-shaped. Such slides are referred to as rotational slides (Fig 2). They are commonly deep-seated (depth/length ratio = 0.15—0.33). Backward rotation of the failed mass is the dominant characteristic, and the failed material remains intact to the extent that only one or a few discrete blocks are likely to form.

Although the slip surface is concave upwards it seldom approximates to a circular arc of uniform curvature. For instance, if the shear strength of the soil is lower in the horizontal than vertical direction, the arc may flatten out; if the soil conditions are reversed, then the converse may apply. What is more, the shape of the slip surface is very much influenced by the existing discontinuity pattern.

Rotational slides usually develop from tension scars in the upper part of a slope, the movement being more or less rotational about an axis located above the slope. The tension cracks at the head of a rotational slide are generally concentric and parallel to the main scar. Undrained depressions and perimeter lakes, bounded upwards by the main scar, characterize the head regions of many rotational slides.

When the scar at the head of a rotational slide is almost vertical and unsupported, then further failure is usually just a matter of time. As a consequence, successive rotational slides occur until the slope is stabilized. These are retrogressive slides and they develop in a headward direction. All multiple retrogressive slides have a common basal shear surface in which the individual planes of failure are combined. Non-circular slips occur in overconsolidated clays in which weathering has led to the development of quasi-planar slide surfaces, or in unweathered structurally anisotropic clays. Both circular and non-circular shallow rotational slips tend to form on moderately inclined slopes in weathered or colluvial clays.

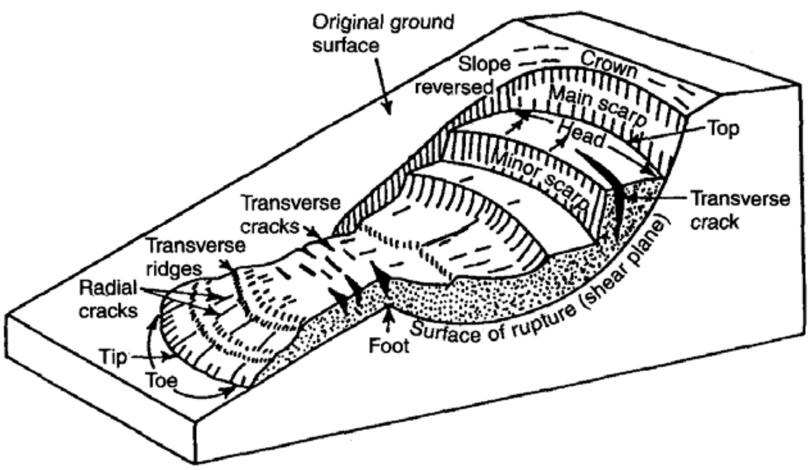

Figure 2 The main features of a rotational slide
Source: Varnes (1978)

Translational slides occur in inclined stratified deposits, the movement occurring along a planar surface, frequently a bedding plane. The mass involved in the movement becomes dislodged because the force of gravity overcomes the frictional resistance along the potential slip surface, the mass having been detached from the parent rock by a prominent discontinuity such as a major joint. Slab slides, in which the slip surface is roughly parallel to the ground surface, are a common type of translational slide. Such a slide may progress almost indefinitely if the slip surface is sufficiently inclined and the resistance along it is less than the driving force. Slab slides can occur on gentler surfaces than rotational slides and may be more extensive.

According to Skempton and Hutchinson (1969), compound and translational slides develop in clay deposits when rotation is inhibited by an underlying planar feature, such as a bedding plane or the base of a weathered boundary layer. Translational slides tend to be more superficial than compound slides, being governed by more shallow inhomogeneities. Clay that is subjected to part rotational, part translational sliding is often distorted and broken. Block slides may develop in the more lithified, jointed deposits of clay, blocks of clay first separating and then sliding on well-defined bedding, joint or fault planes. Slab slides are characteristic of more weathered clay slopes of low inclination. Material moves en masse with little internal distortion.

Weathered mantle and colluvial materials are particularly prone to slab failure, which rarely occurs with depth/length ratios greater than 0.1. If a sufficient number of overlapping slips develop, they may form a shallow translational retrogressive slide. Failures that involve lateral spreading may develop in clays, quick clays and varved clays. This type of failure is due to high pore water pressure in a more permeable zone at relatively shallow depth, dissipation of pore water pressure leading to the mobilization of the clay above. The movement is usually complex, being predominantly translational, although rotation and liquefaction, and consequent flow may also be involved. Such masses, however, generally move over a planar surface and may split into a number of semi-independent units. Like other landslides, these are generally sudden failures, although sometimes movement can take place slowly.

Rock slides and debris slides are usually the result of a gradual weakening of the bonds within a rock mass and are generally translational in character. Most rock slides are controlled by the discontinuity patterns within the parent rock. Water is seldom an important direct factor in causing rock slides, although it may weaken bonding along joints and bedding planes. Freeze—thaw action, however, is an important cause. Rock slides commonly occur on steep slopes and most are of single rather than multiple occurrence. They are composed of rock boulders. Individual fragments may be very large and may move great distances from their source. Debris slides are usually restricted to the weathered zone or to surficial talus. With increasing water content debris slides grade into mudflows. These slides are often limited by the contact between the loose material and underlying firm bedrock.

In a flow the movement resembles that of a viscous fluid (Bishop, 1973). In other words, as movement downslope continues, intergranular movements become more important than shear surface movements. Slip surfaces are usually not visible or are short-lived, and the boundary between the flow and the material over which it moves may be sharp or may be represented by a zone of plastic flow. Some content of water is necessary for most types of flow movement, but dry flows can and do occur. Consequently, the range of water content in flows must be regarded as ranging from dry at one extreme to saturated at the other. Dry flows, which consist predominantly of rock fragments, are simply referred to as rock fragment flows or rock avalanches and generally result from a rock slide or rockfall turning into a flow. They are generally very rapid and short-lived, and are frequently composed mainly of silt or sand. As would be expected, they are of frequent occurrence in rugged mountainous regions, where they usually involve the movement of many millions of tonnes of material. Wet flows occur when fine-grained soils, with or without coarse debris, become mobilized by an excess of water. They may be of great length.

Progressive failure is rapid in debris avalanches and the whole mass, either because it is quite wet or is on a steep slope, moves downwards, often along a stream channel, and it advances well beyond the foot of a slope. Lumb (1975) reported speeds of 30 m s for debris avalanches in Hong Kong. The main characteristics of many slips that occur in the residual soils (mainly decomposed granite) of Hong Kong are the rapid fall of debris (once movement starts the whole mass separates from the main slope within minutes) and the shallow depth of the slide, usually less than 3 m. The ratio of thickness to length of the scar is usually less than 1.5. There is rarely any prior warning that a slip is imminent. The prime cause of failure is direct infiltration of rainwater into the surface zones of slopes, leading to soil saturation and its loss of effective cohesion. Debris avalanches are generally long and narrow, and frequently leave V-shaped scars tapering headwards. These gullies often become the sites of further movement.

Debris flows are distinguished from mudflows on the basis of particle size, the former containing a high percentage of coarse fragments, while the latter consist of at least 50% sand-size or less. Almost invariably, debris flows follow unusually heavy rainfall or the sudden thaw of frozen ground. These flows are of high density, perhaps 60 to 70% solids by weight, and are capable of carrying large boulders. Like debris avalanches, they commonly cut V-shaped channels, at the sides of which coarser material may accumulate as the more fluid central area moves down-channel. Debris may move over many kilometres.

Mudflows may develop when a rapidly moving stream of storm water mixes with a sufficient quantity of debris to form a pasty mass. Because such mudflows frequently occur along the same courses, they should be kept under observation when significant damage is likely to result. Mudflows frequently move at rates ranging between 10 and 100m min and can travel over slopes inclined at 1° or less, although they usually develop on slopes with shallow inclinations, that is, between 5 and 15°. Skempton and Hutchinson (1969) observed that mudflows also develop along discretely sheared boundaries in fissured clays and varved or laminated fluvio-glacial deposits where the ingress of water has led to softening at the shear zone. Movement involves the development of forward thrusts due to undrained loading of the rear part of the mudflow, where the basal shear surface is Inclined steeply downwards. A mudflow continues to move down shallow slopes due to this undrained loading which is implemented by frequent small falls or slips of material from a steep rear scarp on to the head of the moving mass. This not only aids instability by loading but it also raises the pore water pressures along the back part of the slip surface (Hutchinson and Bhandari, 1971; Bromhead, 1978).

An earthflow involves mostly cohesive or fine-grained material, which may move slowly or rapidly. The speed of movement is to some extent dependent on water content in that the higher the content, the faster the movement. Slowly moving earthflows may continue to move for several years. These flows generally develop as a result of a build-up of pore water pressure, so that part of the weight of the material is supported by interstitial water with consequent decrease in shearing resistance. If the material is saturated, a bulging frontal lobe is formed and this may split into a number of tongues, which advance with a steady rolling motion. Earthflows frequently form the spreading toes of rotational slides due to the material being softened by the ingress of water. Skempton and Hutchinson (1969) restricted the term 'earthflow' to slow movements of softened weathered debris, as forms at the toe of a slide. They maintained that movement was transitional between a slide and a flow, and that earthflows accommodated less breakdown than mudflows.

Factors Affecting Landslide

Landslides in Relation to Geomorphology (Landform: Slope angle, elevation)

Mehortra, Sarkar and Dharmaraju (1992) analyzed that maximum number of landslides occur in the slope category of 31^{o}-40^{o} followed by slope category 21^{o}-30^{o}. These slope categories in the field have been found to consist predominantly of moderate to highly weathered rock types frequently jointed and fractured as well. Incidence of landslides have been found to be much less on the rocky slopes generally steep, falling in the category of 51^{o}-60^{o} more than 60^{o}.

The change of slope gradient may be due to natural or artificial interference i.e. to the undermining of the foot of the slope by stream erosion or by excavation. Exceptionally, the change of slope gradient may be produced by tectonic processes, by subsidence or uplift. The increase in slope gradient provokes a change of stress in the rock mass; the equilibrium is then distributed by the increase in shear stress. Upon the relief of lateral stress the rocks on the slope loosen and facilitate the penetration of water (Zaruba and Mencel, 1967).

Varnes (1984) noted that steepness of slope in relation to the strength of slope forming materials was very important: for zoning purposes, slope inclination was often grouped into range of degrees or percentages. He also pointed out that the interrelation between slope gradient and stability was not simple and that the steeper slope might not always be those most likely to fail. Many steep slopes of competent rock were more stable as compared to gentle slopes of weak material. The complex relationships between relative frequency of landslides, slope and lithology could be statistically examined.

The data suggested that while steeper slopes provided greater potential energy to induce failure, they were also indicative of higher strength materials. This trade-off between increased driving force and increased strength appeared to reduce the importance of slopes that were steeper than this threshold should be influenced to a greater degree by the remaining factors that affect landslide susceptibility.

Landslides in Relation to Geology (Lithology, Structural geology)

Lee and Min (2001) stated the landslide occurrence value was higher in granite gneiss and leucocratic gneiss areas, and was lower in quartz mica schist and biotite gneiss areas.

Khantaprab (1993) conducted a study on November 1988 landslides in southern Thailand and proposed the geology factors influencing the landslides. The areas underlain by granitic terrain with residual soil of weathered granite had higher landslides.

Landslides in Relation to Surface Drainage zone

It is observed that the incidence of landslides are more in areas having drainage density values between 3-4 km/km^2 characterized by medium to coarse texture having infiltration more or equal to runoff. The areas designated as low having drainage density values less than 3.0 km/km^2 and characterized by coarse texture with infiltration more than runoff. The frequency of landslides has been found to be comparatively much less in areas having drainage density values more than 4 km/km^2 having fine to medium texture (Mehrotra, Sarkar and Dharmaraju, 1992).

Landslides in Relation to Soil Characteristics

Collins and Znidarcic (2004) stated the relations between soil and rainfall parameters and the cause of failure for slopes subject to infiltration. Coarse-grained soils and high infiltration rates lead to the development of positive pore water pressures and failure will be caused by seepage forces within the slope. Fine-grained soils and low infiltration rates do not lead to the development of positive pore pressures and failure will more often occur due to the decrease in shear strength caused by the loss of suction. In general, shallower failures are associated with the development of positive pore pressures, while deeper failures are associated with a loss in suction. However, it should be noted that the failure depth is governed not only by the strength characteristics, but also by the hydraulic characteristics of the soil and that both should be investigated in performing detailed analyses.

Landslides in Relation to Land use and Land cover

Varnes (1984) stated effect of vegetation on slope stability appears to be complex in that depending on local conditions of soil depth, slope and type of vegetation, a vegetative cover in some ways definitely promotes stability and in other ways it may not.

Greenway (1987) also stated in the same way that vegetation that may be growing on a slope has traditionally been considered to have an indirect or minor effect on stability; and it is usually neglected in stability analysis. This assumption is not always correct and for certain forested slopes with relatively thin soil mantles has shown significantly in error.

The relationship of landslide activities with various land use types in the Himalayan region, India. The agricultural lands have occupied the maximum area and have also shown maximum proneness to landslide. The high rate of landslide event in this category of land use could be due to its locations commonly preferred by local people either in old/dormant slide area or close to populated areas where ill planned construction activities have already taken place. The barren and sparsely vegetated areas have shown more frequent occurrences of landslides as compared to thickly and moderately vegetated areas possibility due to insufficient growth of secondary vegetation on the slope and the ground (Mehrotra, Sarkar and Dharmaraju, 1992).

Landslides in Relation to Rainfall Intensity

Precipitation causes an increase or risk in the water level and increases the pore water pressure within the rock or soil. This action greatly reduces the shearing strength of the soil. This same water or an increase in moisture content adds weight to the mass and lubricates the slip planes. The actions will increase the chances for the down slope movement of the landslide mass.

Rain and melt water penetrate into the joints producing hydrostatic pressure; the increase in pour-water pressure in soil induces a change of consistence, which in turn causes a decrease of cohesion and internal friction. Recurrent sliding movement generally occurs in the years of usually high rainfall (Zaruba and Mencel, 1967).

Summerfield (1991) said that raindrops possess kinetic energy by virtue of their mass and velocity. Although the impact velocity of raindrops varies depending on the droplet size, wind speed and turbulence, raindrops of maximum size under normal conditions of around 6 mm diameter have an impact velocity of about 9 m/s. At this speed, rain drops can directly move particles more than 10 mm across and coarser material can be dislodged by the removal of down slope support provided by finer sediment. Rain splash erosion can occur wherever vegetation does not entirely cover the ground, although it is a more potent erosive agent in environments where there is little or no vegetation cover. Both slope gradient and surface characteristics influence the effectiveness of rain splash erosion. Experimental studies have shown that on low angle slope at 5^{o} only about 60% of the particles dislodged by the raindrop impacts move down slope but this percentage increases with gradient reaching 95% on 25^{o} slopes. It also appears that rain splash erosion is more effective on sandy surfaces than those containing a high proportion of clay and silt-sized material, apparently because the presence of finer particles contribute to cohesion.

Landslide Hazard Map in Thailand

Samran (1984) studied the rainfall erosivity-factor, R in Universal Soil Loss Equation, USLE, for mountainous areas in northern Thailand from automatic record rainfall intensity. He reported results that rainfall erosivity-factor, R indicated highly significant relationships between rainfall factor and rainfall amount in terms of annual, seasonal and monthly basis. And annual, wet seasonal and monthly rainfall had highly significant relationships with elevation and aspect.

Pantanahiran (1994) conducted research to identify landslide areas and to develop a predictive landslide model using various parameters from a limited data base. Pipun and Kiliwong areas in Thailand were selected for model development and validation, respectively. Information obtained from topographic maps and remotely sensed data were used in this study. The predictive model was formulated using logistic regression under TIN and GRID modules in ARC/INFO and SAS software. Land use/land cover and landforms were the primary factors affecting landslides in the study areas. The sensitive areas in Pipun occur at an elevation of 400-600 m which had slopes of 16-30°. In addition, approximately 75% of all landslides in Pipun occurred within 140 m of a stream channel. Eight parameters including elevation, aspect, vegetations (TM4), flow accumulation, soil characteristics (Brightness), soil moisture (Wetness), slope, and flow direction were selected as significantly contributing to the model. The logistic model was represented by the equation:

$$\begin{aligned} Y &= 1.8914 - 0.00281(\text{Elevation}) + 1.4215(\text{Adjusted aspect}) \\ &\quad - 0.00505(\text{TM4}) + 0.00073(\text{Flow accumulation}) \\ &\quad - 0.0042(\text{Brightness}) - 0.00504(\text{Wetness}) + 0.00698(\text{Slope}) \\ &\quad - 0.00165(\text{Flow direction}) \end{aligned}$$

and $P = 1/(1 + \exp(-Y))$ is the estimated probability (P) of landslide presence at a given cell.

The results indicated that the predictive model correctly classified 82% of the landslides at a 0.4 cutoff probability.

Table 1 The landslide potential and the rang of probability

Landslide Susceptibility Classes	Range of probability
Very low to nil susceptibility to landslide	0-20
Low susceptibility to landslide	21-40
Moderate susceptibility to landslide	41-60
High susceptibility to landslide	61-80
Very high susceptibility to landslide	80-100

Source: Pantanahiran (1994)

Auathaveepon (1995) reported application of satellite data on classification of landslide risk area in Amphoe Phipun, Changwat Nakhon Si Thammarat. Also the total of 226 square grid selected each 1x1 square kilometer corresponding with active landslide which occurred in 1989. The slope, landform, geological characteristics, soil characteristic, rainfall and landuse were investigated as independent variable coinside with appearant landslide on sattellite image. The relationships between the percentage of landslide and independent variables were formulated by stepwise method. The best multiple regression equation is

$$\text{Log } Y = 1.3285 - 0.0101(\text{Slope}) - 0.1021(\text{Landform}) + 0.9178(\text{Land use}) + 0.5189(\text{Geology}) - 0.8939(\text{Soil}) + 0.3213(\text{Rainfall})$$

in which the coefficient of determination (R^2) is equal 0.6538.

For landslide susceptibility study Department of Land Development used weighting factor index. Five factors such as rock type, slope, land use, soil properties and rainfall precipitation intensity were identified as the main factors governing slope instability in Thailand.

Table 2 The detailed descriptions of different weighted factor values

Parameter	Weight Value	Rating Value		Score
		Description	Rating	
1. Rock type	10	1. Sedimentary rock 2. Sandstone/Shale 3. Limestone/Dolomite/Pyrite 4. Metamorphic of Igneous rock/Quartzite 5. Granite/Slate	1 2 3 4 5	1x10=10 2x10=20 3x10=30 4x10=40 5x10=50
2. Slope (%)	9	1. 0-8% 2. 8-16% 3. 16-35% 4. 35-50% 5. >50%	1 2 3 4 5	1x9=9 2x9=18 3x9=27 4x9=36 5x9=45
3. Land used and Land cover	8	1. Forest 2. Grassland/Deforest 3. Vacant land/Orchard 4. Agriculture 5. Open area	1 2 3 4 5	1x8=8 2x8=16 3x8=24 4x8=32 5x8=40
4. Soil properties	7	1. Fine grain soil +deep 2. Medium +deep/ Fine grain soil +intermediate 3. Fine grain soil +shallow/ Coarse grain soil +deep 4. Medium + intermediate 5. Coarse grain soil +shallow	1 2 3 4 5	1x7=7 2x7=14 3x7=21 4x7=28 5x7=35
5. Rainfall intensity	6	1. < 1,800 mm/yr 2. 1,801-2,100 mm/yr 3. 2,101-2,400 mm/yr 4. 2,401-3,200 mm/yr 5. 3,201-4,000 mm/yr	1 2 3 4 5	1x6=6 2x6=12 3x6=18 4x6=24 5x6=30

Source: Department of Land Development (1996)

Table 3 The landslide potential and the rang of total score

Landslide Susceptibility Classes	Range of Score
Very low to nil susceptibility to landslide	40-72
Low susceptibility to landslide	73-104
Moderate susceptibility to landslide	105-136
High susceptibility to landslide	137-168
Very high susceptibility to landslide	169-200

Source: Department of Land Development (1996)

Naramngam (1996) applied GIS and factor of safety (F.S.) in determining landslide risk area sub-watershed Klong Kathu and Klong Dindaeng of Tapi watershed, Changwat Nakhon Si Thammarat. The F.S. value was calculated using the equations proposed by Mairaing, Abe, Gray and Megahan, Gray and Leiser, Wu et al. and Coppin and Richards. Applicability and efficiency of those equations were evaluated based on the concided value (CV) representing percentage of the overlaps in terms of size and location of landslide area between actual and simulated landslide maps. The most feasible equation in determining and mapping landslide risk area is Wu et al.'s equation when soil depth was given at 1.5 m. and 2.0 m.

Chalermpong (2002) conducted to identify landslide risk area and communities that might be affected by landslides in the East-Coast Gulf Watershed. Landslide statistics and factors were investigated. The landslide risk factors were employed together with the geographic information system to prepare, analyze, and map landslide risk area. The land use map, geology map, and soil group map were used to analyses landslide risk.

Junkhiaw (2003) applied the technique of geographic information system and Artificial Neural Network (ANN) to create modal flash flood and landslide risk area. The modal was conducted under the influence parameters such as the topographical, geomorphology, land use characteristics, and hydrometeorology. The Phuket Island was the study area. High level hazard of landslide was found on granite mountain.

Thaijeamaree (2003) studied the landslide behaviors for Nam Kor Watershed, Nam Kor subdistrict, Lom Sak district, Phetchabun Province. The studies were done by field survey on landslide area, field tests, and laboratory tests such as strength. Finite Element Method on soil slope during heavy rainfall was performed using these test results for infiltration analyses. The relationship of rainfall patterns and the stability of slope gave the critical rainfall causing landslide. This report found direct shear test showed when the moisture content of the samples increased, the shear strengths decreased. These relationships can establish the critical rainfall envelope when the Factor of Safety (FS.) is equal to unity. With the various rainfall patterns from 1-14 raining days, the critical rainfall envelope can be established and used as future warning levels for the villager.

Table 4 The detailed descriptions of different weighted factor values

Parameter	Weight Value		Rating Value		Score
	Weight	Sub	Description	Rating	
1. Geology	5				
1.1 Rock type		3	A. Igneous rocks	5	5x3=15
			B. Sedimentary rocks	3	3x3=9
			C. Metamorphic rocks	1	1x3=3
1.2 Lineament zone		2	A. Inside lineament zone	3	3x2=6
			B. Outside Lineament zone	1	1x2=2
2. Landform	4				
2.1 Slope (%)		3	A. >70%	5	5x3=15
			B. 50-70%	4	4x3=12
			C. 30-50%	3	3x3=9
			D. 15-30%	2	2x3=6
			E. 0-15%	1	1x3=3
2.2 Elevation-m		1	A. >401 m	5	5x1=5
			B. 301-400 m	4	4x1=4
			C. 201-300 m	3	3x1=3
			D. 101-200 m	2	2x1=2
			E. 0-100 m	1	1x1=1
3. Surface drainage zone	2		A. Inside	2	2x2=4
			B. Outside	1	1x2=2
4. Soil characteristics	2		A. Gravel loam/Gravelly sand	5	5x2=10
				4	4x2=8
			B. Sand	3	3x2=6
			C. Sandy loam	2	2x2=4
			D. Clayey loam/loam	1	1x2=2
			E. Clay, Mud		
5. Land used and Land cover	3		A. Agriculture area	4	4x3=12
			B. Urban and build-up area	3	3x3=9
				2	2x3=6
			C. Other deforestation	1	1x3=3
			D. Forest area		
6. Rainfall intensity (mm)	5		A. > 2,826 mm/yr	3	3x5=15
			B. 2,726-2,825 mm/yr	2.5	2.5x5=12.5
			C. 2,626-2,725 mm/yr	2	2x5=10
			D. 2,476-2,675 mm/yr	1.5	1.5x5=7.5
			E. 2,325-2,475 mm/yr	1	1x5=5

Source: Thassanapak (2001)

Table 5 The landslide potential and the rang of total score

Landslide Susceptibility Classes	Range of Score
Very low to nil susceptibility to landslide	21-33
Low susceptibility to landslide	34-45
Moderate susceptibility to landslide	46-58
High susceptibility to landslide	59-70
Very high susceptibility to landslide	71-82

Source: Thassanapak (2001)

Study susceptibility of landslide by Thassanapak (2001) use weighted factor index. The influencing parameter of geology including rock types and lineament zone, slope gradient and elevation, surface drainage zone, land use and land cover, soil characteristics, and rainfall intensity were identified as the main factors governing slopes instability in Phuket Thailand.

Kunsuwan (2005) studied the landslide behavior for Khlong Krating, Khlong Takhian and Klong Thung Phen, in Chantaburi sub-basin during the heavy rainfalls and floods in 1999 and 2001. The hazard map was created by the relationships between the rainfall patterns, rainfall duration, return period, the slope stability and the critical rainfall envelop in order to use for landslides warning. The results showed that the failure slopes were on the area of 25-35 degree slopes and the depth of 2.5-3.5 meters. The soil profiles were on the weathered granite rock with high natural moisture contents. The shear strength of soil decreased with increase of the degree of saturation. The study of the distribution of the sediment carried from the landslide areas along the rivers found that the sediment of rocks decreased with increasing of the distance from the source. The critical F.S. occurred right after the end of heavy rainfall. The correlation of the slope stability analyses with the historical rainfall data lead to landslide critical rainfall envelope of the F.S. equal to 1.1.

General Method of Evaluating Landslide Hazard Zonation.

Definition of Hazard Zonation

To differentiate between the terms hazard; and risk, following definitions (given by Varnes, 1984) have become generally accepted:

NATURAL HAZARD (H): The probability of occurrence of a potentially damaging phenomenon within a specified period of time and within a given area.

VULNERABILITY (V): The degree of loss to a given element or set of elements at risk resulting from the occurrence of a natural phenomenon of a given magnitude. It is exposed on a scale from 0 (no damage) to 1 (total loss).

SPECIFIC RISK (Rs): The expected degree of loss due to a particular natural phenomenon. It may be expressed by the product of H and V.

ELEMENT AT RISK (E): The population, properties, economic activities, including public services, etc. at risk in a given area.

TOTAL RISK (Rt): The expected number of lives lost, persons injured, damage to property, or disruption of economic activity due to a particular natural phenomenon. It is therefore the product of specific risk (Rs) and elements at risk (E).

Hazard Assessment

Disaster result from vulnerable conditions being exposed to a potential hazard. Therefore, the first step in taking any mitigation measures is to assess the hazard. Hazard assessment aims to come to grips with: (a) the nature, severity and frequency of the hazards; (b) the area likely to be affected; and (c) the time and duration of impact. (Ian Davis and Gupta, 1989)

Landslide Hazard Zonation

Landslide hazard is commonly shown on maps, which display the spatial distribution of hazard classes (landslide hazard zonation). Zonation refers to " the division of the land in 'homogeneous' areas or domains and their ranking according to degrees of actual/potential hazard caused by mass movement" (Varnes, 1984).

Anbalagan (1992) stated that Landslide Hazard Zonation (LHZ) map depicts the division of land surface in to zones of varying degree of stability based on the estimated significance of the causative factors in inducing instability. He pointed out the usefulness of the LHZ map as follow.

The LHZ maps are useful for the following purposes

l. LHZ map help the planners to choose favourable location for site development schemes such as building and road construction. Even if the hazardous areas can not be avoided altogether, their recognition in the initial stages of planning may help to adopt suitable precautionary measures.

2. As the LHZ map delineates the areas into zones of varying degree of stability, the environmental regeneration measures can be initiated in high hazard areas by adopting suitable mitigation measures.

Mapping Scale

Van Westen (1994) stated selection of the working scale for a slope instability analysis project is determined by the purpose for which it is executed. He followed the scale of analysis presented in the International Association of Engineering Geologists monograph on engineering geological mapping (IAEA, 1976) in his study of landslide hazard zonation in Andes of Colombia. The scales are
National scale (< 1: 1,000,000) Synoptic or regional scale (< 1:100,000) Medium scale (1:25,000 - 1:50,000) Large scale (1:5,000 - 1: 10,000)

Mapping Framework of Landslide

Einstein (1988) introduced the framework of mapping landslide in to five levels

1. State of nature map
2. Danger maps
3. Hazard maps
4. Risk maps
5. Landslide management maps

Hazard Mapping Analysis

Van wester (1993) stated in his publication that the most straightforward type of hazard map is a landslide inventory map displaying present and past landslides. Assessment of the area extent of landslides and their evolution in the recent past can be made with the use of multi-temporal photo interpretation and geomorphological fieldwork.

The report stated that the prediction of hazard in areas presently free of landslides requires different methods, based on the assumption that hazardous phenomena that have occurred in the past can provide useful information for the prediction of occurrences in the future. Therefore, mapping these phenomena and the factors thought to be of influence is very important in hazard zonation. He cited the two general approaches used for such mapping

1. Many of the geomorphology-based hazard zonation studies can be called hazard mapping studies, since the hazard is basically assessed in the field during mapping. This method is also called direct approach (Hansen, 1984).

2. Indirect methods calculate the importance of the combinations of parameters occurring in landslide locations, and extrapolate the results to landslide-free areas with similar combinations, mostly by statistical techniques (Hansen, 1984)

The report cited Hartlen and Viberg (1988) who differentiated between relative hazard and absolute hazard assessment techniques. The relative hazard assessment techniques differentiate the likelihood of occurrence of mass movements for different areas on the map, without giving exactly exact values.

Absolute hazard maps display an absolute value for the hazard, either as a factor of safety or a probability of occurrence. A combination is also possible, indicating the probability that the factor of safety is below one.

Absolute hazard assessment techniques can be divided into three main groups (Carrara, 1983; Hartlen and Viberg, 1988):

1. White box model, based on physical models (slope stability and hydrological models) also referred to as deterministic models;
2. Black, box models, not based not on physical models but on statistical analysis;
3. Grey box models, based partly on physical models and partly on statistics.

Principles of Hazard Zonation

According to Varnes (1984) Landslide Hazard Zonation is still in a stage of experimentation. He has indicated at least three basic principles or fundamental assumptions that have guided all zonation studies.

1. The past and present are keys to the future
2. The main conditions that cause landslide can be identified
3. Degree of hazard can be estimated

General Trend in Landslide Hazard Zonation Techniques

A large amount of research on hazard zonation has been done over the last 30 years as the consequences of and urgent demand for slope instability hazard mapping. Several types of landslide hazard zonation techniques have been developed in which Van westen (1994) has listed the summary of the various trends in the development of techniques as follow

Type of landslide analysis	Main characteristic
A. Distribution analysis	Direct mapping of mass movement features resulting in a map which gives information only for those sites where landslides have occurred in the past
B. Qualitative analysis	Direct, or semi-direct, methods in which the geomorphological map is renumbered to a hazard map or in which several maps are combined into one using subjective decision rules, based on the experience of the earth scientist
C. Statistical analysis	Indirect methods in which statistical analysis are used to obtain predictions of the mass movement hazard from a number of parameter maps
D. Deterministic analysis	Indirect methods in which parameter maps are combined in slope stability calculations

E. Landslide frequency analysis	Indirect methods in which earthquake and/or rainfall records or hydrological models are used for correlation with known landslide dates, to obtain threshold values with a certain frequency

Data Required for Input in GIS for Landslide

Van Westen (1994) pointed out the list of various input data needed to assess landslide hazard at regional, medium and large scale. The list is extensive, and only in a ideal case will all type of data be available. However, the amount and type of data that can be collected, determine the type of hazard analysis that can be applied ranging from qualitative assessment to complex statistical methods.

The data layer needed to analyze landslide hazard can be subdivided into five main groups; geomorphical; topographic; engineering geological or geotechnical; land use; and hydrological data. A data layer in a GIS can be seen as one digital map, containing one type of data composed of one type of element (points, line, units) and having one or more accompanying Tables. The layers that have to be taken into account vary for different environment.

Phases of Landslide Hazard Analysis Using GIS (Van westen, 1993)

The following phases can be distinguished in the process of mass movement hazard analysis using GIS:

1. Choice of working scale and the methods of analysis which will be applied;
2. Collection of existing maps and reports with relevant data;
3. Interpretation of Images and creation of new input maps;
4. Design of the data base and definition of the way in which data should be collected and stored;
5. Fieldwork to verify the photo-interpretation and to collect relevant quantitative data;
6. Laboratory analysis of soil and rock samples for classification;
7. Digitizing of maps and attribute data;
8. Validation of the entered data;
9. Manipulation and transformation of the raw data to a form which can be used in the analysis;
10. Analysis of data for preparation of hazard maps;
11. Evaluation of the reliability of the output maps and inventory of the errors which may have occurred during the previous phases.
12. Final production of hazard maps and adjoining reports.

Weighting Factor Method

A numerical rating system or a weight-rating system is based on the theory of logical combination. A weighting or a measure of relative importance, must be assigned each influencing factor. Each influencing factor was subdivided into subclasses and given index numbers. Although the index numbers are for identification only, the subclasses should be arranged in a logical sequence, such as from gentle to steep or small to large. The product of these factors was the potential of the area indicated susceptibility to landslide.

A simplified formula to predict the susceptibility to landslide is defined as follows;

$$M_t = M_1W_1+M_2W_2+ M_3W_3+ M_4W_4+\ldots\ldots\ldots\ldots\ldots\ldots+ M_nW_n$$

Where M_t = Total scores
M = Value of the importance factor
W = Value of subclasses of the importance factor

Rock Mass Qualitative System

Rock masses have been described from the earliest geological maps onwards. The descriptions of the rocks were initially in lithological and in other geological terms. With increasing knowledge of geology, geological features and the influence of geology on engineering the amount of information to be included in a description for geotechnical purposes increased, leading to sets of rules for the description or characterization of a rock mass geotechnically. Parallel with this development, a movement took place in mining and engineering geology to combine the characterization of a rock mass with direct recommendations for tunnel support. This resulted in rock mass classification systems. The systems were developed primarily empirically by establishing the parameters of importance, giving each parameter a numerical value and a weighting. This led, via empirical formulae, to a final rating for a rock mass. The final rating was related to the stability of the underground excavation. In systems that are more elaborate, the rating was also related to the support installed in the excavation and to stand-up times. The success of classification systems in underground excavations resulted in classification systems also being used for slopes. Classifications systems have been designed following many different calculation methods and also the used parameters and their influence on the final result differ widely from system to system. This obviously sets some question marks to the validity of classification systems. The correlation between the results of some systems is often quoted to prove that the systems do work, but also this on detailed investigation seems not to be so convincing.

Rock Mass Rating

In 1973 Bieniawski introduced the Geomechanics Classification also named the Rock Mass Rating (RMR), at the South African Council of Scientific and

Table 6 Rock mass rating

A. CLASSIFICATION PARAMETERS AND THEIR RATINGS									
Parameter			Range of values						
1	Strength of intact rock material	Point-load strength index	>10 MPa	4-10 MPa	2-4 MPa	1-2 MPa	For this low range - uniaxial compressive test is preferred		
		Uniaxial comp. strength	>250 MPa	100-250 MPa	50-100 MPa	25-50 MPa	5-25 MPa	1-5 MPa	< 1 MPa
	Rating		15	12	7	4	2	1	0
2	Drill core Quality *RQD*		90%-100%	75%-90%	50%-75%	25%-50%	< 25%		
	Rating		20	17	13	8	3		
3	Spacing of discontinuities		> 2 m	0.6-2 . m	200-600 mm	60-200 mm	< 60 mm		
	Rating		20	15	10	8	5		
4	Condition of discontinuities (See E)		Very rough surfaces Not continuous No separation Unweathered wall rock	Slightly rough surfaces Separation < 1 mm Slightly weathered walls	Slightly rough surfaces Separation < 1 mm Highly weathered walls	Slickensided surfaces or Gouge < 5 mm thick or Separation 1-5 mm Continuous	Soft gouge >5 mm thick or Separation > 5 mm Continuous		
	Rating		30	25	20	10	0		
5	Ground water	Inflow per 10 m tunnel length (l/m)	None	< 10	10-25	25-125	> 125		
		(Joint water press)/ (Major principal σ)	0	< 0.1	0.1,-0.2	0.2-0.5	> 0.5		
		General conditions	Completely dry	Damp	Wet	Dripping	Flowing		
	Rating		15	10	7	4	0		

B. RATING ADJUSTMENT FOR DISCONTINUITY ORIENTATIONS (See F)						
Strike and dip orientations		Very favourable	Favourable	Fair	Unfavourable	Very Unfavourable
Ratings	Tunnels & mines	0	-2	-5	-10	-12
	Foundations	0	-2	-7	-15	-25
	Slopes	0	-5	-25	-50	

C. ROCK MASS CLASSES DETERMINED FROM TOTAL RATINGS					
Rating	100 ← 81	80 ← 61	60 ← 41	40 ← 21	< 21
Class number	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

D. MEANING OF ROCK CLASSES					
Class number	I	II	III	IV	V
Average stand-up time	20 yrs for 15 m span	1 year for 10 m span	1 week for 5 m span	10 hrs for 2.5 m span	30 min for 1 m span
Cohesion of rock mass (kPa)	> 400	300-400	200-300	100-200	< 100
Friction angle of rock mass (deg)	> 45	35-45	25-35	15-25	< 15

E. GUIDELINES FOR CLASSIFICATION OF DISCONTINUITY conditions					
Discontinuity length (persistence) Rating	< 1 m 6	1-3 m 4	3-10 m 2	10-20 m 1	> 20 m 0
Separation (aperture) Rating	None 6	< 0.1 mm 5	0.1-1.0 mm 4	1-5 mm 1	> 5 mm 0
Roughness Rating	Very rough 6	Rough 5	Slightly rough 3	Smooth 1	Slickensided 0
Infilling (gouge) Rating	None 6	Hard filling < 5 mm 4	Hard filling > 5 mm 2	Soft filling < 5 mm 2	Soft filling > 5 mm 0
Weathering Ratings	Unweathered 6	Slightly weathered 5	Moderately weathered 3	Highly weathered 1	Decomposed 0

F. EFFECT OF DISCONTINUITY STRIKE AND DIP ORIENTATION IN TUNNELLING**			
Strike perpendicular to tunnel axis		Strike parallel to tunnel axis	
Drive with dip-Dip 45-90°	Drive with dip-Dip 20-45°	Dip 45-90°	Dip 20-45°
Very favourable	Favourable	Very favourable	Fair
Drive against dip-Dip 45-90°	Drive against dip-Dip 20-45°	Dip 0-20-Irrespective of strike°	
Fair	Unfavourable	Fair	

*Some conditions are mutually exclusive. For example, if infilling is present, the roughness of the surface will be overshadowed by the influence of the gouge. In such cases use A.4 directly.

Source: Bieniawski (1989)

Industrial Research (CSIR). The rating system was based on Bieniawski´s experience in shallow tunnels in sedimentary rocks. Originally, the RMR-system involved 49 unpublished case histories. Since then the classification has undergone several significant changes. In 1974 there was a reduction of parameters from 8 to 6 and in 1975 there was an adjustment of ratings and reduction of recommended support requirements. In 1976 a modification of class boundaries took place (as a result of 64 new case histories) to even multiples of 20 and in 1979 there was an adoption of the ISRM rock mass description. The newest version of RMR is from 1989, where Bieniawski published guidelines for selecting the rock reinforcement. In that version, Bieniawski suggested that the user could interpolate the RMR-values between different classes and not just use discrete values. Therefore, it is important to state which version is used when RMR-values are quoted. Since the Hoek-Brown, Yudhbir and Sheorey rock mass criteria suggest and prefer that the 1976 version of RMR should be used. When applying this classification system, one divides the rock mass into a number of structural regions and classifies each region separately. The RMR-system uses the following six parameters, whose ratings are added to obtain a total RMR-value.

i. Uniaxial compressive strength of intact rock material;
ii. Rock quality designation (RQD);
iii. Joint or discontinuity spacing;
iv. Joint condition;
v. Ground water condition; and
vi. Joint orientation.

The first five parameters (i-v) represent the basic parameters (RMRbasic) in the classification system. The sixth parameter is treated separately because the influence of discontinuity orientations depends upon engineering applications. Each of these parameters is given a rating that symbolizes the rock quality description.

Slope Mass Rating

Most of the empirical rating methods apply adjustment factors to their basic rock mass rating. These adjustment factors account for such things as defect orientation, excavation method, weathering, induced stresses and major planes of weakness. Bieniawski (1976 and 1989) applies the adjustments by subtracting them from the rock mass rating. Table 1 show that the defect orientation adjustment can dominate the RMR. If the defect orientations are deemed “very unfavourable” an adjustment of -60 is required to the basic rock mass rating. Even for defect orientations denoted as “fair” this adjustment is -25. There is no guideline as to what “very unfavourable” means. Bieniawski (1989) recommends the use of the Romana (1985) SMR corrections for slopes. Romana used the same basic rock mass rating as RMR_{89} but developed new adjustment factors for joint orientation and blasting to account for the lack of guidelines in the RMR methods. The equation for SMR is shown below. The joint orientation weighting includes a factor for the difference between joint dip and slope angle, F_3. This requires an iterative approach for design.

Table 7, 8 and Table 9 show the adjustment ratings.

$$SMR = RMR + F_1F_2F_3 + F_4$$

Romana (1985) developed his factors not only for rock mass failures but also for wedge and planar failure. A rock mass rating method should not be used for these two cases as they are defect controlled and can be assessed using such measures as stereographic projection. Even if the method was applicable, the ratings for planar failure are questionable. F_2 depends on defect dip and must account for the defect shear strength. However, the method seems to assume that friction angles are quite high. For example, bedding surface shears may attain strengths of ϕ' below 12° yet these would be given a 'very favourable' rating of 0.15.

Table 7 Adjustment rating for joints

Case		Very Favourable	Favourable	Fair	Unfavourable	Very unfavourable
P	$\|\alpha_j - \alpha_s\|$	>30°	30°-20°	20°-10°	10°-5°	<5°
T	$\|\alpha_j - \alpha_s - 180^\circ\|$					
P/T	$F_1 = (1 - \sin\|\alpha_j - \alpha_s\|)^2$	*0.15*	*0.4*	*0.7*	*0.85*	*1.00*
P	$\|\beta_j\|$	<20°	20°-30°	30°-35°	35°-45°	>45°
P	$F_2 = \tan^2 \beta_j$	*0.15*	*0.4*	*0.7*	*0.85*	*1.00*
T	F_2	*1.00*	*1.00*	*1.00*	*1.00*	*1.00*
P	$\beta_j - \beta_s$	>10°	10°-0°	0°	0°-(-10°)	<-10°
T	$\beta_j - \beta_s$	<110°	110°-120°	>120°	-	-
P/T	F_3	*0*	*-6*	*-25*	*-50*	*-60*

P - Planar failure α_s - Slope dip direction α_j - Defect dip direction

T - Toppling failure β_s - Slope dip β_j - Defect dip

Source: Romana (1985)

Table 8 Adjustment rating for methods of excavation of slopes

Method	Natural Slope	Presplitting	Smooth Blasting	Blasting or Mechanical	Defficient Blasting
F_4	+15	+10	+8	0	-8

Source: Romana (1985)

Table 9 Tentative description of SMR classes

SMR	0-20	21-40	41-60	61-80	81-100
Class	V	IV	III	II	I
Description	Very Bad	Bad	Normal	Good	Very Good
Stability	Completely Unstable	Unstable	Partially Stable	Stable	Completely Stable
Failures	Big planar or soil like	Planar or big wedges	Some joints or many wedges	Some blocks	None
Support	Reexcavation	Important/ Corrective	Systematic	Occasional	None

Source: Romana (1985)

The CSMR method (Chen, 1995) is based on the SMR method. The CSMR applies a discontinuity condition factor, λ, that describes the conditions of the controlling discontinuity on which the ratings F1, F2 and F3 are based (Table 10). This factor ranges from 0.7 to 1.0. The CSMR method also assumes that the SMR method is applicable for a slope height of 80m but must be adjusted for other slope heights, H, using the slope height factor, x. The relationship for x, based on an extensive survey and rigorous analysis of slopes in China, is shown in Figure 3. With the addition of the two new factors, the equation for CSMR is defined as:

$$CSRM = \xi RMR + \lambda F_1 F_2 F_3 + F_4$$

$$\xi = 0.57 + 34.4H$$

where, H = Slope height in metres

Table 10 Discontinuity condition factor λ

λ	Defect Condition
1.0	Faults, long weak seams filled with clay
0.8 to 0.9	Bedding planes, large scale joints with gouges
0.7	Joints, tightly interlocked bedding planes

Source: Chen (1995)

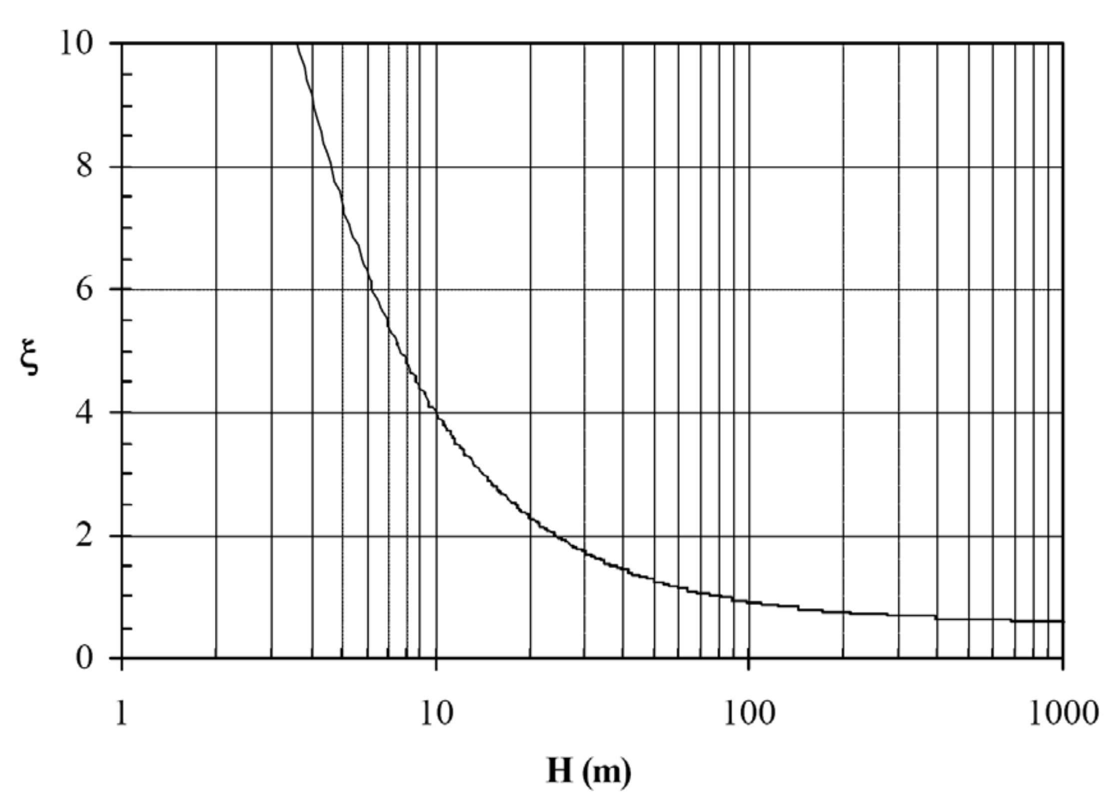

Figure 3 Slope height, H, vs slope height factor, x
Source: Chen (1995)

The CSMR has been based on the SMR and thus has similar problems. CSMR acknowledges the affect of slope height. It is the authors view that height should not be grouped with the rock mass rating (a defacto strength estimate) but should be addressed during the stability analysis where it will contribute to the stresses acting.

Cut slope

Japan Society of Engineering Geology (1992) stated that in order to design for the earthwork or tunnels, it is essential to probe ahead and to grasp geological conditions, soil and rock properties which make up the object of rock mass. But considering the complex and varied conditions of topographies and geologies in Japan, it is impossible to grasp all conditions at the stage of probing ahead. After the construction started pratically, problems which we have unexpected at the stage of probing ahead often rises. So original design, classified geological conditions strictly and designed each geological conditions minutely, often does not mean anything. For this reason, Japan Highway Public Corporation classifies familiar type of soil and geological conditions roughly, and tries to design or construct efficiently and rationally. Japan Society of Engineering Geology (1992) reported on the standard rock mass classification for the choice of cut slope gradient for earthwork design.

Table 11 Range of standard cut slope gradients for bedrock soil

Bedrock soil		Cut Height	Gradient
Hard rock			1:0.3 - 1:0.8
Soft rock			1:0.5 - 1:1.2
Sand	Those not dense, not solid and of bad grade distribution		1:1.5 -
Sandy soil	Those that are dense and solid	less than 5 m	1:0.8 - 1:1.0
		5 - 10 m	1:1.0 - 1:1.2
	Those not dense, not solid	less than 5 m	1:1.0 - 1:1.2
		5 - 10 m	1:1.2 - 1:1.5
Sandy soil mixed with gravel or rock mass	Those that are dense and solid or of good grade distribution	less than 10 m	1:0.8 - 1:1.0
		10 - 15 m	1:1.0 - 1:1.2
	Those not dense, not solid or of bad grade distribution	less than 10 m	1:1.0 - 1:1.2
		10 - 15 m	1:1.2 - 1:1.5
Cohesive soil		0 - 10 m	1:0.8 - 1:1.2
Cohesive soil mixed with rock mass or cobblestone		less than 5 m	1:1.0 - 1:1.2
		5 - 10 m	1:1.2 - 1:1.5

Note: 1) Silt is placed under cohesive soil. Individual consideration is given to soils not indicated in the table.

2) The gradient in the table is the gradient of a single slope not including the beam.

3) The indication of gradient 1:n= 

Source: The Japan Highway Public Corporation (1992)

After construction starts, cut slope becomes weathered from surface as time goes by, and become unstable gradually. And generally speaking, natural ground is often complicated and ununiform. On this account, despite examining the cut slope stability for every individual geological condition in detail, the examination is often meaningless, regarding it as the whole road design. Generally, The Japan Highway Public Corporation (1992) adopted the value of cut slope gradient indicated in Table 11. It indicates the standard range of cut slope gradient produced by our experiences on the condition that the face of slope is protected from erosion to a certain degree.

However when engineering plane civil engineering design, it is necessary to consider the whole earthwork planning, and in filling section sometimes choosing gentle slope gradient to increase cumulative cut. In waste section, on the other hand, it is necessary to choose steep slope gradient protected stability by structure for decreasing cumulative cut to compare with standard slope gradient and many cutting. And in such places, large cut slope, slope in landslide area, or slopes with soil which may collapse, it is necessary to examine slope stability more minutely (The Japan Highway Public Corporation, 1992).

Logistic Regression

The Multiple Linear Regression Model

Multiple linear regression is in some ways a relatively straightforward extension of simple linear regression allowing for more than one independent variable. The objective of multiple regression is the same as that of simple regression; that is, we want to use the relationship between a response (dependent) variable and factor (independent) variables to predict or explain the behavior of the response variable. This chapter will illustrate the similarities and the differences between simple and multiple linear regression, as well as develop the methodology necessary to use the multiple regression model.

The multiple linear regression model is written as a straightforward extension of the simple linear model. The model is specified as

$$y = \beta_0 + \beta_1 x_1 + \beta_2 x_2 + \ldots + \beta_m x_m + \varepsilon,$$

where

y is the dependent variable

x_j ,j = 1,2,..., m, represent m different independent variables

β_0 is the intercept (value when all the independent variables are 0)

β_j ,j = 1, 2, ..., m, represent the corresponding m regression coefficients

ε is the random error, usually assumed to be normally distributed with mean zero and variance α^2

Although the model formulation appears to be a simple generalization of the model with one independent variable, the inclusion of several independent variables creates a new concept in the interpretation of the regression coefficients. For example, if multiple regression is to be used in estimating weight gain of children, the effect of each of the independent variables—dietary supplement, exercise, and behavior modification—depends on what is occurring with the other independent variables. In multiple regression we are interested in what happens when each variable is varied one at a time, while not changing values of any others. This is in contrast to performing several simple linear regressions, using each of these variables in turn, but where each regression ignores what may be occurring with the other variables. Therefore, in multiple regression, the coefficient attached with each independent variable should measure the average change in the response variable associated with changes in that independent variable, while all other independent variables remain fixed. This is the standard interpretation for a regression coefficient in a multiple regression model.

Multiple Logistic Regressions

The simple logistic regression model can easily be extended to two or more independent variables. Of course, the more variables, the harder it is to get multiple observations at all levels of all variables. Therefore, most logistic regressions with more than one independent variable are done using the maximum likelihood method. The extension from a single independent variable to m independent variables simply involves replacing $\beta_0 + \beta_1 x$ with $\beta_0 + \beta_1 x_1 + \beta_2 x_2 + ... + \beta_m x_m$ in the simple logistic regression equation given in Section 10.4. The corresponding logistic regression equation then becomes

$$\mu_{y/x} = \frac{\exp(\beta_0 + \beta_1 x_1 + \beta_2 x_2 + ... + \beta_m x_m)}{1 + \exp(\beta_0 + \beta_1 x_1 + \beta_2 x_2 + ... + \beta_m x_m)}$$

Making the same logit transformation as,

$$\mu_p = \log\left[\frac{\mu_{y/x}}{1 - \mu_{y/x}}\right],$$

we obtain the multiple linear regression model:

$$\mu_p = \beta_0 + \beta_1 x_1 + \beta_2 x_2 + ... + \beta_m x_m$$

General Information of Phuket province

The areas under study cover Phuket Island, about 900 km south of Bangkok on the west coast of peninsular Thailand. It is bound by latitudes 7^o 52’ 12” and 7^o 57’ 36” N and longitudes 9^o 15’ 24” and 9^o 26’ 48” E, encompassing an area of approximate 549 km^2. This includes three major districts, namely Amphoe Muang Phuket, Amphoe Thalang, and Amphoe Kathu. The mapped area covers the 1:50,000 topographic map of Changwat Phuket, sheet no 4624i, 4625ii.

The area studied covers approximately 549 km^2 in the Phuket Island. At least 60 percent of the area is granitic rocks of the Phuket Plutons. The ages of the granitices range from Cretaceous to Tertiary. The granites from composite plutons is elongated shape in the N-S direction. They have been divided, based upon field observation, into 5 types: from the older to the younger as coarse-grained porphyritic biotite granites (G-1), fine-to medium-grained biotite granites (G-2), medium-to coarse-grained biotite granite slightly porphyritic (G-3), fine-to-medium-grained biotite-muscovite granites locally porphyritic (G-4), and fine-grained biotite-muscovite-tourmaline granites (G-5) (Charusiri, 1980).

The permo-Carboniferous sedimentary rocks of the Phuket Group are wholly clastic and composed mainly of mudstone, laminated mudstone, diamictite, siltstone and sandstone. The stratified rocks are slightly metamorphosed due to tectonic effects and granitic intrusions. The general strike of the Phuket Group is from N-S to NE-SW with gentle dip. Structurally, both granitic and sedimentary rocks are considered principally to be faulted, and fractured by the tectonic episode developed from late Palezoic to Tertiary and locally by igneous activities.

Climate

The Phuket-Island climate can be classified as tropical rainforest climate with fairly uniform high temperatures and heavy rainfall throughout the year without distinct dry-cold season. The statistics produced by the Royal Thai Meteorological Department for Phuket during 1995 to 2004 reveal that the highest and lowest temperatures are about 36.2 ^{o}C and 16.9 ^{o}C, respectively. There are at least 6 months of heavy rainfall which are predominated by southwest monsoon rather than northeast monsoon. The yearly average rainfall is about 2,379 mm. The two highest rainfall peaks develop during the periods of transitional directions of monsoons. Monthly precipitation averages for Phuket is given below:

Table 12 Rainfall (m.m.) in Muang Phuket

year	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	TOTAL
1998	T	0.0	0.0	5.0	121.1	295.7	212.0	453.4	494.3	388.6	399.6	67.3	2437.0
1999	64.2	90.5	111.0	265.4	152.1	229.8	224.3	337.9	381.6	426.3	242.1	25.1	2550.3
2000	59.1	104.4	112.7	183.9	234.0	240.9	65.3	367.7	290.5	416.9	167.9	127.7	2371.0
2001	69.2	36.2	189.9	75.9	164.1	267.9	222.1	225.8	495.3	224.6	112.6	118.8	2202.4
2002	9.1	0.0	59.2	86.8	202	223.5	201.6	239.3	361.9	223.3	178.1	114.4	1899.2
2003	13.3	0.0	147.2	72.3	92.6	230.7	356.7	393.0	352.3	658.6	112.3	36.0	2465.0
2004	21.3	2.7	10.1	51.8	195.1	338.8	350.7	266.8	173.9	387.8	127.1	66.7	1992.8
2005	1.2	3.8	8.2	84.2	311.7	158.3	72.4	138.3					773.1
1971-2000	21.7	30.3	59.2	135.4	282.6	244.0	283.5	293.5	381.4	305.0	173.8	59.4	2269.8
Rainfall (mm.) 1998-2005 and return period 30 year (1971-2000) (" T " = Trace)													

Source: The Meteorological Department (2006)

Table 13 Relative humidity (%) in Muang Phuket

year	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	MEAN
1998	68	67	67	68	73	80	82	83	83	85	84	79	77
1999	74	70	73	80	80	79	78	79	82	83	81	72	78
2000	73	71	75	81	79	81	77	79	80	82	79	80	78
2001	73	71	77	75	78	77	78	76	82	83	75	73	77
2002	67	64	68	73	76	78	75	76	81	81	79	77	75
2003	69	66	71	72	75	78	81	78	82	85	78	84	77
1971-2000	69	67	68	73	79	78	79	78	81	81	78	73	75
relative humidity (%) monthly 1998-2003 and return period 30 year (1971-2000)													

Source: The Meteorological Department (2006)

Table 14 Mean temperature (°C) in Muang Phuket

year	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	Annual
1998	29.2	29.8	30.0	31.2	30.7	28.7	28.1	27.7	27.4	27.1	27.0	26.9	28.7
1999	27.7	28.3	28.9	28.1	28.1	27.8	28.0	27.8	27.3	27.1	27.0	26.9	27.8
2000	28.0	28.5	28.6	28.2	28.6	27.8	28.5	27.9	28.0	27.4	27.2	27.9	28.1
2001	28.1	28.6	28.3	29.7	28.9	29.1	28.4	29.3	27.4	27.5	27.8	28.5	28.5
2002	28.2	29.0	29.8	29.7	29.4	28.9	29.2	28.6	27.6	27.7	28.0	28.4	28.7
2003	28.5	29.4	29.6	29.7	29.3	28.6	27.7	28.4	27.6	26.8	28.3	27.8	28.48
2004	29.55	29.96	30.37	30.51	29.88	28.88	28.14	28.98	28.35	28.23	29.05	28.65	29.22
1961-1990	27.9	28.7	29.3	29.5	28.4	28.3	27.8	27.9	27.3	27.4	27.5	27.6	28.1
mean temperature (°C) monthly 1998-2004 and 30 year (1961-1990)													

Source: The Meteorological Department (2006)

Table 15 Mean max. temperature (°C) in Muang Phuket

year	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	Annual
1998	34.7	35.3	35.7	36.4	35.2	32.9	32.4	32.2	31.5	30.9	31.1	31.0	33.3
1999	32.3	32.9	33.6	32.3	32.4	32.1	31.9	32.0	32.0	31.3	31.6	31.5	32.2
2000	32.8	-	-	32.1	32.3	31.3	32.2	31.4	32.1	31.3	30.8	31.7	31.8
2001	31.9	32.7	32.2	33.5	32.8	32.1	32.3	32.4	31.2	31.5	31.6	32.1	32.2
2002	32.4	33.7	33.8	33.6	32.8	32.2	32.6	32.0	31.6	32.1	32.0	32.1	32.6
2003	32.7	34.3	34.3	34.1	33.0	32.6	31.5	31.9	30.9	30.2	32.3	31.6	32.45
2004	33.58	33.95	34.25	34.53	33.01	31.88	31.26	31.90	31.91	31.72	32.41	32.07	32.70
1961-1990	31.8	32.9	33.5	33.4	32.0	31.6	31.2	31.2	30.7	30.9	31.0	31.2	31.8
mean max. temperature (°C) monthly 1998-2004 and 30 year (1961-1990)													

Source: The Meteorological Department (2006)

Table 16 Mean min. temperature (°C) in Muang Phuket

year	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	OCT	NOV	DEC	Annual
1998	25.7	26.3	26.2	27.7	27.6	25.9	25.4	25.1	24.8	24.9	24.6	24.5	25.7
1999	24.7	25.1	25.9	25.2	25.4	25.1	25.5	25.0	24.5	24.6	24.6	24.3	25.0
2000	24.9	25.0	25.4	25.4	25.9	25.2	25.7	25.4	25.2	24.9	25.0	25.1	25.3
2001	25.1	25.1	25.4	26.4	26.5	26.0	25.5	26.1	24.9	24.9	25.4	25.0	25.5
2002	25.1	25.4	26.2	26.6	26.3	26.1	26.3	26.4	25.0	24.7	25.3	25.8	25.8
2003	25.5	26.0	26.3	26.3	26.7	25.6	25.0	25.4	24.8	24.5	25.3	25.0	25.53
2004	25.75	25.97	26.49	26.49	26.74	25.87	25.02	26.06	24.79	24.73	25.69	25.22	25.74
1961-1990	23.3	23.7	24.3	24.8	24.5	24.5	24.2	24.4	23.9	23.8	23.8	23.7	24.1
mean min. temperature (°C) monthly 1998-2004 and 30 year (1961-1990)													

Source: The Meteorological Department (2006)

Population

The population census was carried out in 2005 and an effort was made to obtain Thumbon for Phuket province.

Table 17 Population Density

	MALE	FEMALE	TOTAL	HOUSE
Phuket Province	140,703	151,542	292,245	128,110
Amphur Mueang Phuket	50,088	53,473	103,561	53,671
Ko Kaeo	4,273	4,404	8,677	3,967
Ratsada	14,675	15,365	30,040	14,993
Vichit	17,571	19,034	36,605	18,817
Chalong	7,429	8,031	15,460	8,793
Rawai	6,140	6,639	12,779	7,101
Amphur Kathu	2,323	2,503	4,826	2,819
Kamala	2,323	2,503	4,826	2,819
Amphur Thalang	30,110	30,654	60,764	23,705
Thepkrasatri	5,719	5,727	11,446	4,038
Srisunthon	6,227	6,495	12,722	5,734
Choeng Thale	4,664	4,928	9,592	4,507
Pa Khlok	5,621	5,590	11,211	4,076
Mai Khao	5,812	5,779	11,591	3,697
Sakhu	2,067	2,135	4,202	1,653
Thepkrasatri Municipality	2,841	2,968	5,809	2,426
Thepkrasatri	2,841	2,968	5,809	2,426
Choeng Thale Municipality	1,613	1,745	3,358	1,648
Choeng Thale	1,613	1,745	3,358	1,648
Kathu Municipality	8,274	9,334	17,608	9,359
Kathu	8,274	9,334	17,608	9,359
Karon Municipality	3,107	3,283	6,390	4,779
Karon	3,107	3,283	6,390	4,779
Patong Municipality	7,784	7,937	15,721	10,020
Patong	7,784	7,937	15,721	10,020
Phuket Municipality	34,563	39,645	74,208	19,683
Talat Yai	23,919	27,045	50,964	12,424
Talat Nua	10,644	12,600	23,244	7,259

Source: Department of Provincial Administration (2006)

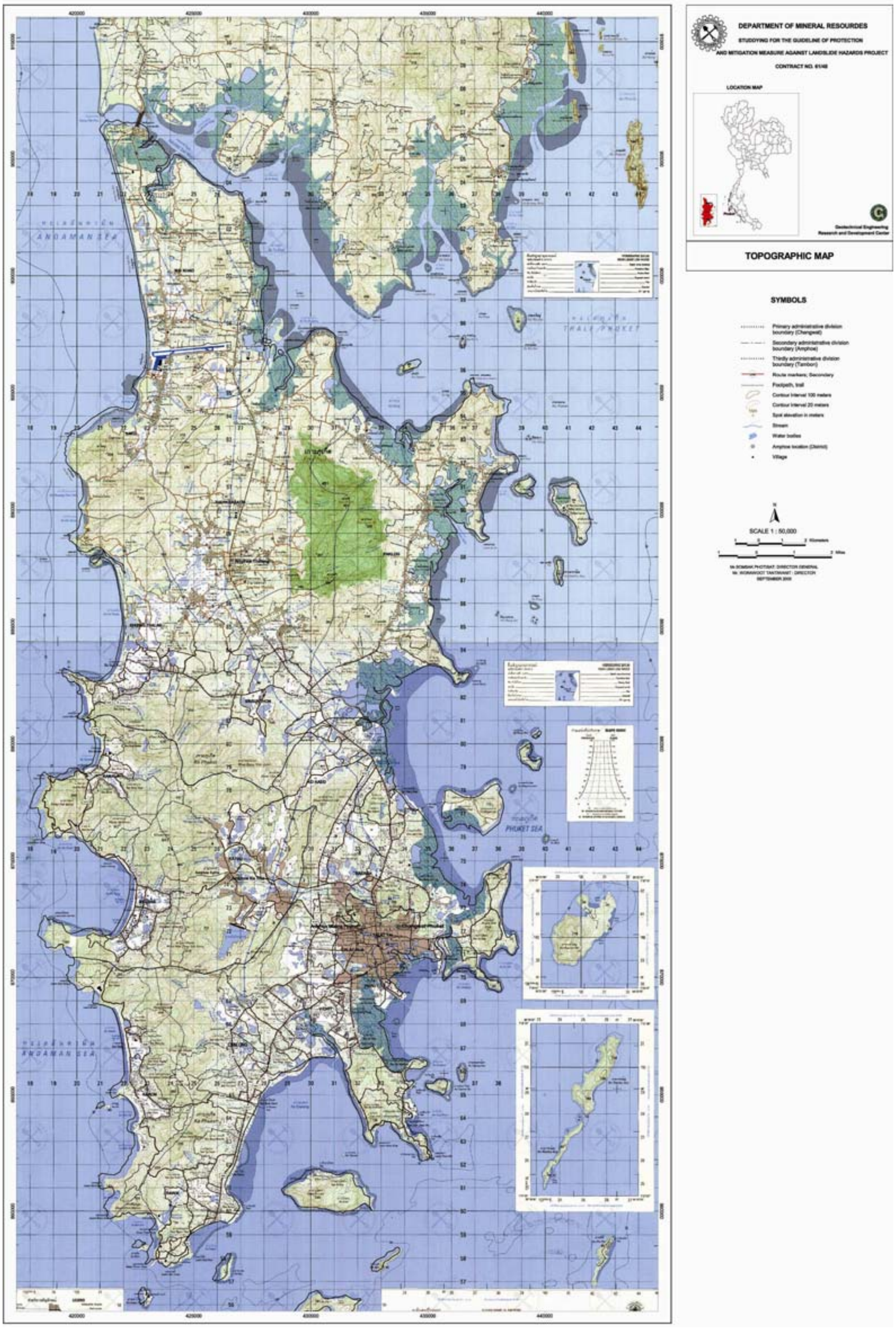

Figure 4 Topographic map of Phuket province
Source: Department of mineral resources (2006)